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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14862.000.000

November 5, 2018

Mr. Michael Hooper
Campus Property Group
P.O Box 564
Larkspur, CA 94977

Subject: 3Rs School Site – 350 Merrydale Road
San Rafael, California

SUBSURFACE EXPLORATION SUMMARY – Geotech

References:
1. ENGEO, Phase I Environmental Site Assessment, 3Rs School Site, San Rafael, California; Project No. 14862.000.000; April 25, 2018.
2. ENGEO, 350 Merrydale Transmittal, Results of Geophysical Subsurface Investigation, San Rafael, California; Project No. 14862.000.000; June 20, 2018.

Dear Mr. Hooper:

We are pleased to submit the findings of our subsurface exploration performed at the subject Property (Property) in San Rafael, California. The purpose of the subsurface exploration was to determine if underground storage tanks (USTs) or UST-related equipment associated with the identified historical underground storage tank (HUST) listing were present within the previously identified two locations.

BACKGROUND

The Property, measuring approximately 2½ acres in area, is located at 3833 Redwood Highway and 350 Merrydale Road in San Rafael, California. The Property, identified with Assessor's Parcel Numbers (APN) 179-041-27 and 179-041-28, is developed with three single-story classroom buildings, associated paved parking, and landscaping. Based on preliminary development concepts provided by Campus Property Group, we understand the development will include 44 three-story townhomes.

ENGEO conducted a phase I environmental site assessment (ESA) for the Property in April 2018 (Reference 1). Based on the findings of this assessment, one Recognized Environmental Condition (REC) identified for the Property was associated with past documentation associated with a potential presence of a former underground storage tank.

Based on this documentation, ENGEO recommended a geophysical survey be performed to determine if underground storage tank(s) remained on the Property.

The geophysical (GPR) survey was performed in June 2018. The report (Reference 2) identified two areas with trench/pit reactions and/or structure reactions at the time of the survey.

SCOPE OF FIELD EXPLORATION

Field sampling activities associated with subsurface exploration were performed on October 13, 2018.

Prior to the exploration, Campus Property Group contacted UST North Service Alert for identification of underground utilities at the Property and contracted the backhoe operator.

The scope of the field exploration included excavation of two locations. Details pertaining to those excavations are presented below.

On October 13, 2018, a backhoe was used to excavate test pits within the footprint of the two locations previously identified by the GPR survey. The excavation at the first location extended to a depth of 4½ feet below ground surface. The lithology consisted of 4 inches of asphalt, which was underlain by 2 inches of base rock, 4 feet of fill, and clay. No odors, staining, equipment, or other evidence of an existing or former UST was observed.

The second excavation near the building was advanced to a depth of 4 feet below ground surface. The lithology consisted of 4 inches of asphalt, which was underlain by 4 inches of base rock, a layer of fabric, 6 inches of recycled rock, and 2½ feet of fill. Clay was encountered at the bottom of the excavation at a depth of approximately 4 feet below ground surface. No odors, staining, equipment, or other evidence of an existing or former UST was observed.

Both test pits were backfilled with the excavated soil cuttings without compactive effort.

DISCUSSION AND CONCLUSION

No existing USTs or evidence of former USTs was observed in the locations that were previously identified during the GPR survey and explored. If any environmental impacts are encountered during demolition, grading, or other construction activities, we recommend an environmental professional be contacted to visit the Property. ENGEO also recommended the preparation of a soil management plan (SMP). An SMP would outline procedures and protocols for handling potentially impacted soil and/or buried structures/equipment that may be encountered during future grading operations and other activities.

If you have any questions regarding this report, please contact us.

Sincerely,

ENGEO Incorporated

Taunee Werts
tw/jaa/jf

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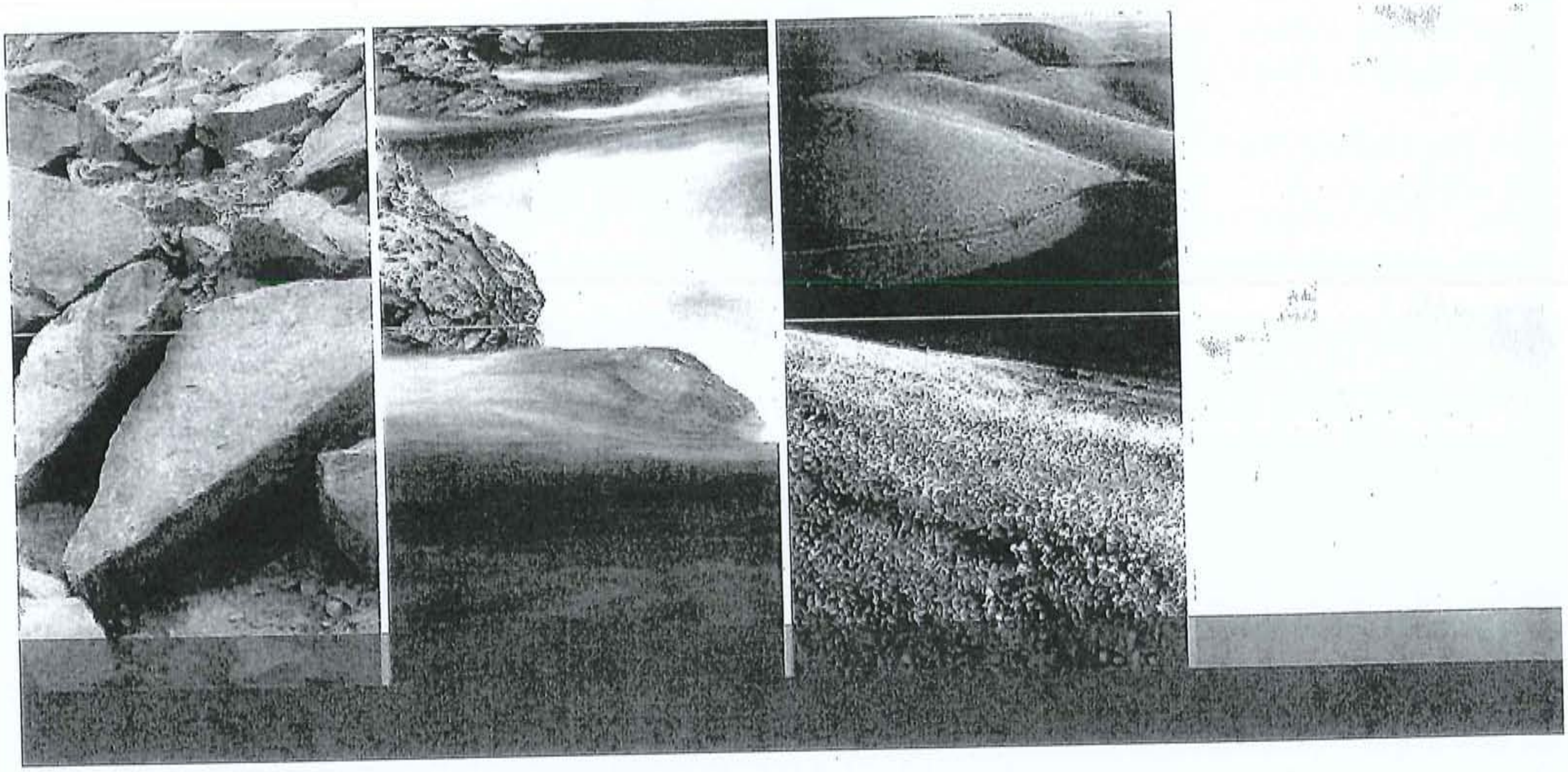

3833 REDWOOD HIGHWAY
SAN RAFAEL, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Michael Hooper
Campus Property Group
P.O. Box 564
Larkspur, CA 94977

PREPARED BY
ENGEO Incorporated

May 8, 2018

PROJECT NO.
14862.000.000

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GEOTECHNICAL
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WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14862.000.000

May 8, 2018

Mr. Michael Hooper
Campus Property Group
P.O. Box 564
Larkspur, CA 94977

Subject: 3833 Redwood Highway
San Rafael, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Hooper:

With your authorization, we performed a preliminary geotechnical assessment for the property located at 3833 Redwood Highway in San Rafael, California. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations. We have also provided preliminary site grading, drainage, and foundation recommendations for use during land planning.

Based upon our initial assessment, it is our opinion that the proposed development is feasible from a geotechnical standpoint. Design-level exploration(s) should be conducted prior to site development once more detailed land plans have been prepared. The main geotechnical considerations include: potential liquefiable soils and compressible soils; slope stability along creek bank areas, presence of expansive near-surface soils and undocumented fills; and shallow groundwater. These concerns along with other planning considerations are further discussed in the following report.

We are pleased to have been of service on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions regarding the contents of this report, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

Maggie Parks, EIT
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical exploration, as described in our agreement dated March 13, 2018, is to provide an assessment of the potential geotechnical concerns associated with the proposed residential development. The scope of our services has included: site reconnaissance, review of published geologic maps and other available geotechnical information at site, advancing six Cone Penetration Tests (CPTs) ranging up to 40 feet deep, and preparation of this report.

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

1.2 PROJECT LOCATION

The project site is located at 3833 Redwood Highway in San Rafael, California (Figure 1). The site is bounded by Merrydale Road on the southwest, commercial and residential buildings on the southeast, Redwood Highway (Highway 101) to the east, and public storage facility to the north-northwest (Figure 2).

A drainage creek lies along the northern edge of the property. The site is approximately 2½ acres in area and comprises Assessor's Parcel Numbers (APN) 179-041-27 and 179-041-28.

1.3 PROJECT DESCRIPTION

Based on preliminary development concept provided by Campus Property Group, we understand the proposed development will include: 44 residential units distributed within several three-story town home buildings.

The existing site elevations are anticipated to remain unchanged for the planned development; minor cuts and fills may be necessary to provide proper drainage for the planned development. The project will also include new streets and surface parking spaces, underground utilities and landscaping.

1.4 SITE BACKGROUND AND SURFACE CONDITIONS

Based on review of historical topographic maps, the site is located close to an original drainage or stream that flowed into the marsh lands to the San Pablo Bay east of the project site. These areas were filled and graded in the early 20th Century. Historical aerial photography, circa 1950s, show that the current drainage channel north of the property had previously traversed through the north of the project. By the 1960's, the original creek alignment had been filled in and diverted to the present-day configuration.

The site is generally flat, at an Elevation of approximately 12 feet above mean sea level (MSL). Currently, the site is occupied by three single-story classroom buildings, paved parking areas, and minor landscaping constructed in the early 1960's. The drainage creek is approximately 6 feet deep lies along the northern edge of the property. The banks of the drainage channel are inclined at a slope of approximately 1½:1 to 2:1 (horizontal: vertical) and the banks are partially covered by rock rip-rap and vegetation.

2.0 FINDINGS

2.1 REGIONAL AND SITE GEOLOGY AND SEISMICITY

The site is located within the Coast Range geomorphic province of California, an area dominated by northwest-trending ridges and valleys. More specifically, the subject site in located on the western edge of San Pablo Bay area of the greater San Francisco Bay. The San Francisco Bay is located in a fault bound, elongated structural trough that has been filled with a sequence of Quaternary age sedimentary deposits derived from the surrounding Coast Ranges. Bedrock geology in this area generally consists of rocks that are a part of the Franciscan Complex, which are an assemblage of deformed and metamorphosed rock units.

Based on mapping by Blake (2000), the deposits underlying the subject site comprise Quaternary-aged alluvium (Figure 3). These deposits consist of sand, gravel, silt, and clay derived from streams and slope-wash. To the north and northeast of the site are mapped artificial fill over marine and marsh deposits. The marine and marsh deposits consist of organic silty clay, silt, and sand and likely were deposited in an estuarine deposit on the margins of San Pablo Bay. Because of the complex nature of an estuarine environment, the alluvium and the marine and marsh deposits may interfinger and interlayer in the area surrounding the site.

2.2 SITE SEISMICITY

The San Francisco Bay Area contains numerous active faults. Figure 3 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. An active fault is defined by the State as one that has had surface displacement within Holocene time (about the last 11,000 years). Based on the 2010 USGS Quaternary Fault and Fold Database (QFFD), the nearest active fault is the Hayward fault located approximately 8.0 miles east of the site. Other active faults in the region are summarized in Table 2.2-1 below.

TABLE 2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

FAULT NAME	DISTANCE FROM SITE (MILES)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE
Hayward-Rogers Creek	8.0	East	7.3
San Andreas	9.9	West	8.0
San Gregorio Connected	16.6	Southwest	7.5
West Napa	19.5	Northeast	6.7
Green Valley	25.0	Northeast	6.8
Calaveras	31.4	Southeast	7.0

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated. As shown on the Regional Geologic Map (Figure 3), faults are mapped in the vicinity of the site, but they are pre-Quaternary faults and are not classified by the state of California as active (Jennings and Bryant, 2010).

As shown on Figure 4, the site is mapped as being highly susceptible to liquefaction by Knudsen et al. (2000). A discussion of the site liquefaction potential follows in Section 3.5.3.

2.3 FIELD EXPLORATION

Our field exploration included advancing six Cone Penetration Test (CPT) soundings at various locations on the site as shown in Figure 2. We performed our field exploration on April 11, 2018. The location and elevations of our explorations are approximate and were estimated using consumer-grade GPS equipment; they should be considered accurate only to the degree implied by the method used.

We retained a CPT rig to push the cone penetrometer to a maximum depth of about 40 feet. The CPT has a 20-ton compression-type cone with a 15-square-centimeter (cm^2) base area, an apex angle of 60-degrees, and a friction sleeve with a surface area of 225 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A.

Soil samples were retrieved using the CPT driven sampler at select locations. Near surface samples were also collected using hand sampling.

2.4 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine Plasticity Index and gradation of selected samples. Individual test results are presented in Appendix B. We also submitted a sample to CERCO Laboratories for corrosivity testing and the results are presented in Appendix C.

2.5 SUBSURFACE CONDITIONS

The CPTs encountered the following generalized subsurface conditions:

- Near surface deposits (up to 4 feet thick) of existing fill deposits comprised of clay and silty sand.
- Underlying existing fills, the CPTs encountered natural alluvial soil deposits. The alluvium includes soft to stiff, clay and silty clay. These deposits were encountered to depths ranging from between about 5 to 15 feet below the ground surface. In general, medium stiff to stiff clays were encountered beneath the upper soft clay softer zones.
- Relatively thin interlayering of medium dense to dense, silty sand layers were encountered at various depths in the CPTs. The sand layers have moderate plasticity and high fines content.

- CPT-4 and CPT- 6 encountered a very dense soil at 24 feet and 25 feet below the ground surface, respectively.
- All of our CPTs encountered practical refusal at depths ranging between 23 and 40 feet below the ground surface.

The CPT logs include the specific subsurface conditions at the location of the probes. We include our exploration logs in Appendix A.

2.6 GROUNDWATER CONDITIONS

During our field exploration, pore pressure dissipation tests were performed at CPT-1, CPT-4, and CPT-6, indicating a depth to groundwater of approximately 3 feet, 4 feet, and 4 feet, respectively. A dynamic pore pressure response was measured in CPT-5 that indicated groundwater of approximately 3½ feet. Fluctuations in the level of groundwater may occur due to variations in rainfall, tidal influences, irrigation practice, and other factors. For preliminary design purposes, a groundwater level of 3 feet was anticipated.

3.0 PRELIMINARY CONCLUSIONS

Based upon this preliminary study, it is our opinion that the proposed development at the site is feasible from a geotechnical standpoint, provided the preliminary recommendations contained in this report and future design-level geotechnical studies are incorporated into the development plans.

The main geotechnical considerations for the proposed development include: potential liquefiable soils and compressible soils; slope stability along creek bank areas, presence of expansive near-surface soils and undocumented fills; and shallow groundwater. These concerns along with other planning considerations are further discussed in subsequent sections of this report.

Site-specific design-level exploration should be undertaken once project plans and details are determined to characterize subsurface conditions and develop design recommendations. The site specific exploration shall include borings and laboratory soil testing to provide specific recommendations for site preparation and grading, foundation design, drainage and other recommendations for the proposed development.

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, soil liquefaction, lateral spreading, and flooding. These hazards are discussed in the following sections. Based on topographic and lithologic data, regional subsidence or uplift and landslides hazards are considered low at the site.

3.1.1 Ground Rupture

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

3.1.2 Ground Shaking

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

3.1.3 Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site, as in other geologically similar locations in the Bay Area, but the offset or strain is expected to be low to negligible.

3.1.4 Liquefaction/Clay Soil Softening

The site is located within an area mapped by Knudsen et al. (2000) as highly susceptible to liquefaction (Figure 4). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

As described previously, the project site is primarily underlain by clays with interlayers of sandy soil below the groundwater table. We performed a detailed liquefaction potential analysis of the CPT soundings to estimate liquefaction potential using the computer software CLiq Version 2.0 developed by GeoLogismiki. The analysis used in the software is based on the procedure by Boulanger and Idriss (2014). The Cyclic Stress Ratio (CSR) was estimated for a Peak Ground Acceleration (PGA) value of 0.5 g as outlined in the ASCE 7-10 and moment magnitude of 7.3. We evaluated the liquefaction potential for the soils encountered below the assumed water table. The analysis showed that discontinuous layers of silty to sandy material found in the CPT soundings were liquefiable. The discontinuous layers were generally approximately 2 to 5 feet thick and extended to approximately 30 feet below the ground surface.

3.1.5 Seismic-Induced Settlement Analyses

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface.

Densification occurs as excess pore pressures dissipate, resulting as vertical settlement at the ground surface.

Clay-like (cohesive) soils can also develop pore pressures during cyclic loading, but generally do not reach zero effective stress and are typically considered non-liquefiable (Robertson 2009). However, clay-like soils can deform during cyclic earthquake loading and experience volumetric strains and post-earthquake reconsolidation. The volumetric strains for clay-like soils are generally small compared to cohesionless soils (sand-like), since clay-like soils often retain some original soil structure. Clay reconsolidation was estimated using the program Cliq.

We calculated potential liquefaction-induced settlement estimates using the program Cliq. The procedures used in Cliq are based on the methods published by Zhang, G., Robertson, P.K., and Brachman, R. (2002). Since some of the granular materials were characterized as medium dense and potentially liquefiable and some fine-grained soil is susceptible to soil softening, we estimate the total liquefaction-induced settlements across the site to be less than 2 inches. Differential settlement during a liquefaction event is expected to be less than 1 inch over a horizontal distance of 30 feet (SCEC, 1999).

As discussed by Ishihara (1985) and Youd and Garris (1995), liquefiable soil that is not overlain by a sufficiently thick non-liquefiable layer area at increased risk of ground surface disruptions, such as fissures and sand boils. The thickness of non-liquefiable soil necessary to reduce this risk is a function of the thickness of the liquefiable soil layer below. Based on the liquefaction analysis and the subsurface stratigraphy, it is our opinion that there is an increased risk of surface expression of liquefied material at the site. Therefore, we provide recommendations in Section 4.1 to help reduce this hazard for the proposed development.

3.1.6 Lateral Spreading and Earthquake-Induced Landsliding

Lateral spreading and earthquake-induced landsliding involve lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sands or weak soils. Due to drainage channel to the north creating a free-face and potentially liquefiable material, there is a potential for lateral stability along the northern portion of the site. Because the bank face is relatively low (approximately 6 feet tall), we recommend a preliminary setback as discussed in Section 4.2 be established for future development. Additional analysis can be performed during design-level study to determine the potential for lateral displacement and impacts to the project.

3.1.7 Tsunami, Seiche, and Flooding

The site is near the edge of San Pablo Bay, though the site is not within an area mapped by the State of California within a tsunami inundation zone (CGS, 2009). A seiche is a type of seismically-induced wave formed within an enclosed body of water. Because the site is not adjacent to enclosed body of water, a seiche is unlikely at the site.

However, based on site elevation and proximate distance from water sources, flooding may be expected at the subject site. The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if recommended.

3.2 SLOPE STABILITY ALONG CREEK

As discussed above, the creek bank along the northern edge of the property is relatively steep. However, because the creek bank is relatively low (approximately 6 feet), we recommend a preliminary setback in Section 4.2. If the proposed development is planned adjacent to the creek, we recommend that slope stability and slope stabilization be addressed in the design-level report.

3.3 COMPRESSIBLE SOILS

We encountered clayey soils in all our CPT soundings. The upper 15 feet of clay soil range from soft to medium stiff and may be susceptible to consolidation settlement induced from building loads. The amount of settlement depends on the weight of future buildings and the compressibility of the soil. The potential for static settlement from the clayey soil should be addressed in the design-level report with additional soil sampling and laboratory testing once building loads are determined. If settlement from building loads cannot be tolerated by the design, mitigation measures including surcharge fill placement and ground improvement can be performed prior to building construction.

3.4 EXISTING UNDOCUMENTED FILL

Based on the CPT logs, up to four feet of fill is anticipated across to the site associated with previous site development. Due to the type of exploration utilized in this study, the thickness of fill should be confirmed with exploratory borings during design-level study. The presence of existing fill can lead to variable foundation movement due to the unknown density of the fill and material properties for structures. Mitigation of existing fill is provided in Section 4.1.2 in this report.

3.5 EXPANSIVE SOIL

A near surface soil sample was collected and laboratory Attenburg Limit testing of the surface material have a Plasticity Index (PI) of 17 which is indicative of a moderately expansive clay material. Successful performance of structures on expansive soils requires special attention during construction. Expansive soils change in volume with changes in moisture. These soils can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction.

We provide preliminary grading recommendations for compaction of clay soil at the site. The purpose of these preliminary recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. Preliminary earthwork recommendations are presented in Section 4.1.5 of this report.

3.6 SOIL CORROSION POTENTIAL

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

TABLE 3.6-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-CPT2	1.0-1.5 feet	6.63	2,300	None Detected	22

Corrosion recommendations associated with these test results are provided in Appendix C.

3.7 SHALLOW GROUNDWATER

In our CPT soundings, we encountered relatively shallow groundwater, at 3 feet below the ground surface. Shallow groundwater may impact excavations for foundations and buried utilities, as well as impact earthwork plans. Excavation anticipated to extending below the groundwater will likely require dewatering during construction. Subjected to the design elevation of the site, additional geotechnical recommendations will be provided in the design-level report.

4.0 PRELIMINARY RECOMMENDATIONS

4.1 LIQUEFACTION SUSCEPTIBILITY

Based on preliminary CPTs, zones of soft and medium dense alluvium are considered susceptible to seismic induced settlement (liquefaction) of up to 2 inches. We estimate differential settlement of 1 inch over a horizontal distance of approximately 30 feet. The foundation for buildings should be design to accommodate this level of settlement. If this seismic settlement potential is excessive and not desirable, it is possible that ground improvement measures could be employed to reduce potential seismic induced settlement.

Design level exploration should characterize liquefaction potential, and also determined that there is sufficient non-liquefiable cap to preclude risk of secondary ground effects (i.e. sand boils, etc.). Other measures such as overexcavation the upper soils and replacement with geogrid reinforced fills may be employed if necessary; such as overexcavation of approximately 3 to 5 feet placement of a layer of geogrid (Tensar TriAx TX160 or approved equivalent) at the bottom of the overexcavation prior to placement and compaction of engineered backfill. Such measures should be evaluated in design level exploration as necessary.

4.2 SLOPE SETBACK RECOMMENDATIONS

From a preliminary planning level, we recommend establishing a minimum structural setback of a minimum 15 feet from the top of the drainage channel bank. The purpose of these setbacks is to address potential for instability and erosion of the drainage channel banks. While erosion of the banks can be mitigated by lining the slopes of the channel with rip-rap or concrete, or regular maintenance. Seismic stability that may extend into the property may impact site development. A slope stability analysis should be performed during design level study to evaluate the impacts to the planned development.

4.3 PRELIMINARY FOUNDATION DESIGN

In order to reduce the effects of the potentially expansive soils and/or liquefaction settlement, the foundations should be sufficiently stiff to move as rigid units with minimum differential movements. This can be accomplished with a post-tensioned mat foundation. A minimum mat thickness of 10 inches should be anticipated for preliminary purposes. The treatment of expansive soil provided in Section 4.3.5 will minimize swell/shrink potential of the soil, but minor movement is anticipated as a result of seasonal moisture fluctuation and irrigation; therefore, structural mats may require stiffening to reduce differential movements due to swelling/shrinkage to a value compatible with the type of structure that will be constructed. In addition, the foundations should be designed for 1 inches differential seismic induced settlement over a distance of 30 feet if liquefaction settlement is not mitigated.

4.4 BUILDING CODE SEISMIC DESIGN

We provide the 2016 California Building Code (CBC) seismic parameters in Table 4.4-1 below.

TABLE 4.4-1: 2016 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	D
Mapped MCE$_R$ Spectral Response Acceleration at Short Periods, S_S (g)	1.50
Mapped MCE$_R$ Spectral Response Acceleration at 1-second Period, S_1 (g)	0.60
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE$_R$ Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.50
MCE$_R$ Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.90
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.00
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.60
Mapped MCE Geometric Mean (MCE$_G$) Peak Ground Acceleration, PGA (g)	0.50
Site Coefficient, F_{PGA}	1.00
MCE$_G$ Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.50
Long period transition-period, T_L	12 sec

4.5 GRADING CONSIDERATIONS

4.5.1 Demolition and Stripping

Site development should commence with the removal of buried structures, including footing elements, abandoned utilities, and septic tanks and their leach fields, if any exist. All debris should be removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation and pavements (asphalt concrete/concrete and underlying aggregate base) should be removed from areas to receive fill, or structures, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. The actual depth of tree root removal should be determined by the Geotechnical Engineer's representative in the field. Subject to approval by the Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soils should be removed from the project site. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

4.5.2 Existing Fill and Disturbed Soil

All existing fill and soft material should be excavated to firm native soils. Excavated material may be used as fill material if it meets the requirements of Section 4.3.3. For planning purposes, the upper 3 feet of soil across the project site should be anticipated to be overexcavated.

4.5.3 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the site soils are suitable for use as engineered fill provided they are broken down to 6 inches or less in size. Other materials and debris, including trees with their root balls, should be removed from the project site.

Imported fill materials should meet the above requirements and have a plasticity index less than the on-site soils. ENGEO should sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

4.5.4 Differential Fill Thickness

Cuts associated with removal of buried structures, foundations, tanks, or undocumented fills could result in differential fill thickness conditions. For overexcavation activities that create a differential fill thickness across a building footprint, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a differential fill thickness of up to 10 feet is acceptable across a building footprint. For a differential fill thickness exceeding 10 feet across a footprint, we recommend performing subexcavation activities to bring this vertical distance to within the 10-foot tolerance and that the material be replaced as engineered fill. As a minimum, the overexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

4.5.5 Fill Placement and Conditioning of Onsite Expansive Material

The near surface onsite material contains moderately expansive soil. To mitigate for shrink and swell potential to impact the planned improvements, overexcavation of the upper 18 inches of near surface soil and recompacting per the compaction control requirements in Table 4.5.5-1.

TABLE 4.5.5-1: Fill Compaction Requirements (ASTM D-1557)

Fill Type	Minimum Relative Compaction	Required Moisture Content
Near surface onsite material (PI>12)	87 to 90 Percent	3 Percentage Point above Optimum
Low Expansive Import (PI ≤ 12)	90 Percent	Above Optimum

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

Additional compaction requirements may be required for deeper fills and retaining wall backfill. These additional requirements will be developed during our detailed exploration.

4.6 TEMPORARY DEWATERING

Utility trench and footing excavation extending below the shallow groundwater condition at the site may require temporary dewatering during construction to keep the excavation and working areas reasonably dry. We anticipate that dewatering for underground utility construction will be accomplished by pumping from sumps. Extended dewatering of utility trench excavations may cause settlement of newly installed pipelines and adjacent improvements. In addition, post-construction long-term dewatering may occur due to the movement of water along utility trenches. We recommend that utility trenches include low permeability cutoffs to reduce the risk of inadvertent groundwater flow along permeable bedding or backfill. When the utility plans are finalized, we will work with the Civil Engineer to determine the placement of the low permeability cutoffs.

In addition, seepage into utility joints may effectively cause dewatering and lead to settlement. We recommend that trench depth be limited as much as practical for the development and that utilities be watertight.

4.7 PRELIMINARY PAVEMENT DESIGN

The following preliminary pavement section has been determined for an assumed R-value of 5 and in accordance to the design methods contained in Chapter 630 of Caltrans Highway Design Manual.

TABLE 4.7-1: Preliminary Pavement Section

TRAFFIC INDEX	AC (INCHES)	AB (INCHES)
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement sections are provided for estimating only. We recommend the actual subgrade material should be tested for R-value and the Traffic Index and minimum pavement section(s) should be confirmed by the Civil Engineer.

4.8 DRAINAGE

The building pads must be positively graded at all times to provide for rapid removal of surface water runoff from the foundation systems and to prevent ponding of water under floors or seepage toward the foundation systems at any time during or after construction. Ponding of stormwater must not be permitted on the building pads during prolonged periods of inclement weather. All surface water should be collected and discharged into the storm drain system. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should be directed to a solid pipe that discharges to the street or to an approved outlet or onto an impervious surface, such as pavement that will drain at a 2 percent slope gradient.

Due to the generally high fines content anticipated in the near-surface site materials, the site soils encountered are not expected to have adequate permeability values to handle stormwater infiltration in grassy swales or permeable pavers. Therefore, best management practices should assume that stormwater infiltration is limited at the site.

4.9 STORMWATER BIORETENTION AREAS

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

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

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

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.
2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

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

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

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

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

5.0 FUTURE STUDIES

As previously discussed, a site-specific design-level geotechnical exploration should be performed once details of the project have been definited. Preliminary conclusions and recommendations presented herein are based on limited site and laboratory data. Based on our preliminary findings in this study, we recommend the design-level geotechnical exploration will include supplemental borings, and laboratory soil testing to provide to future refine the following geotechnical concerns:

- Depth of fill overexcavation and grading criteria
- Stability of drainage channel slope
- Static consolidation settlement risks and mitigation measures as necessary
- Seismic settlement risks and design considerations
- Detail foundation design criteria based on building types

The goal of the exploration is to allow for more detailed evaluations of the geotechnical issues discussed in this report and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 3833 Redwood Highway project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site.

Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

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

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

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

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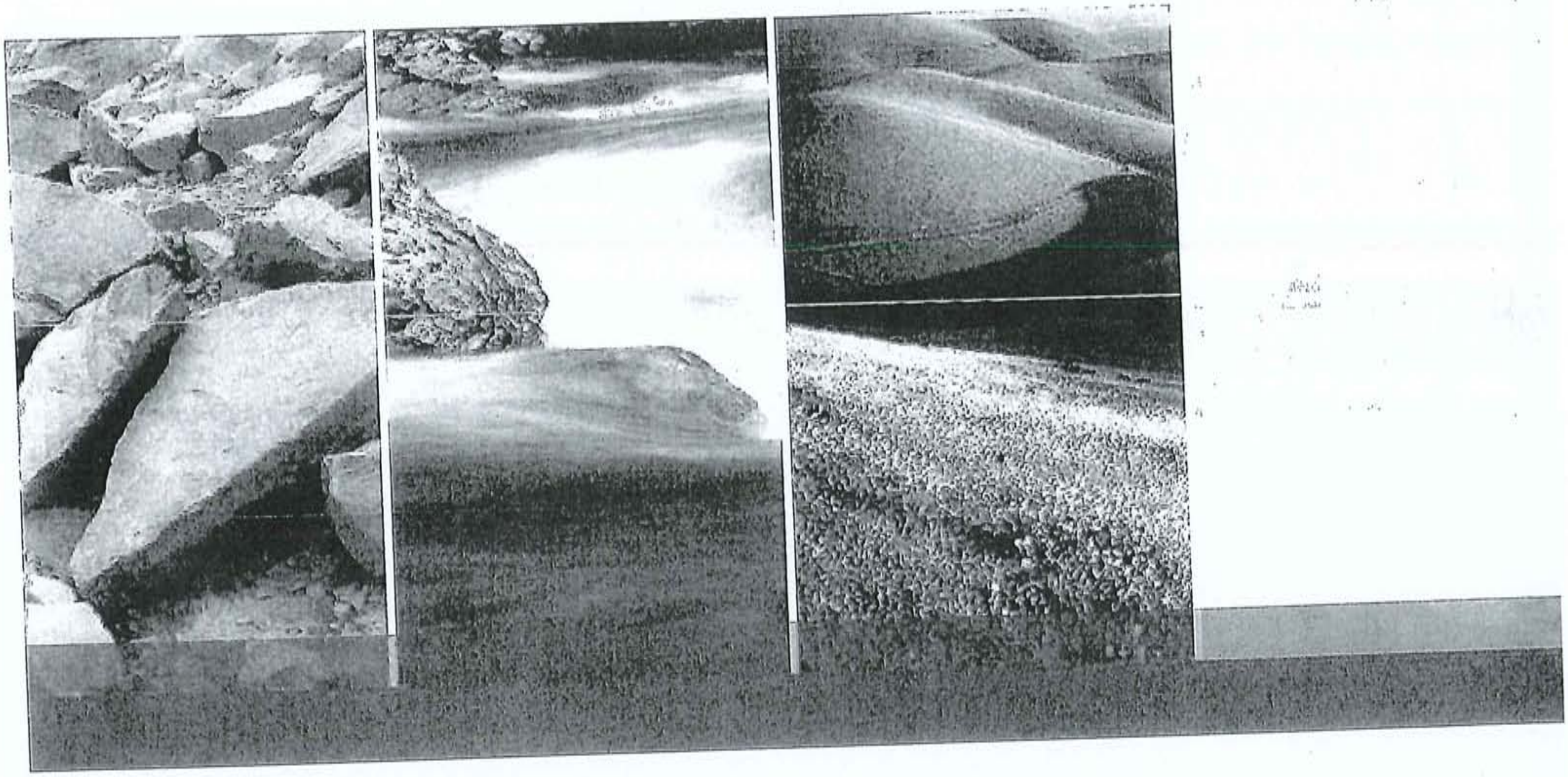

FIGURES

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO — Expect Excellence —

VICINITY MAP
3833 REDWOOD HIGHWAY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 14862.000.000
SCALE: AS SHOWN
DRAWN BY: GLJ
CHECKED BY: LC
FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

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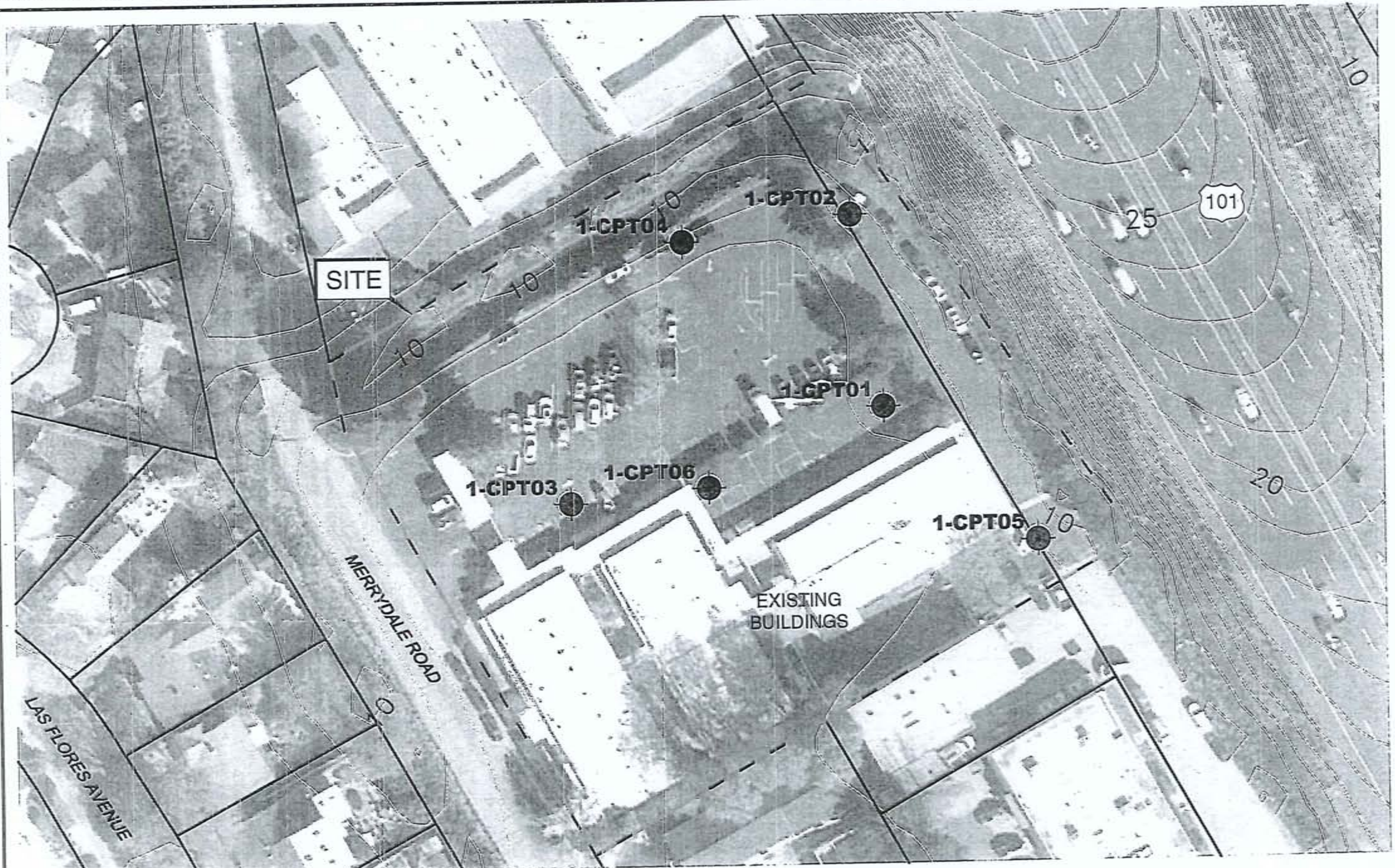


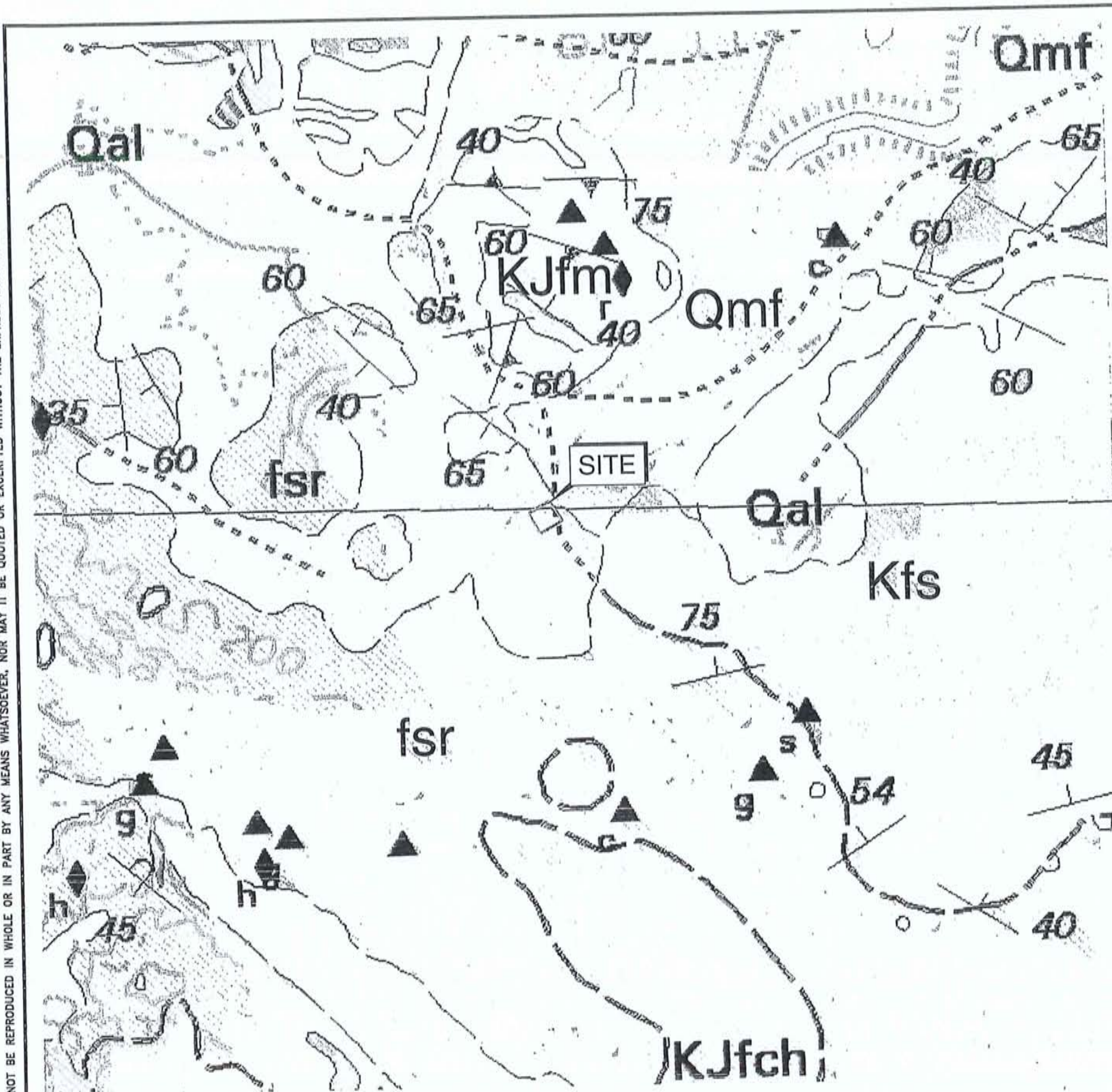
EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT06 CONE PENETRATION TEST

ENGEO — Expect Excellence	SITE PLAN 3833 REDWOOD HIGHWAY SAN RAFAEL, CALIFORNIA	PROJECT NO.: 14862.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: LC	FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

———— – – – –	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED
———— – – – · ·	FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, x INDICATES ACTIVE FAULT

STRIKE AND DIP OF STRATA

INCLINED · VERTICAL · OVERTURNED

Qmf	ARTIFICIAL FILL OVER MARINE AND MARCH DEPOSITS
Qal	ALLUVIUM
Kfs	SANDSTONE AND SHALE
KJfm	METAMORPHIC ROCK
KJfch	CHERT
fsr	MELANGE

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: BLAKE, 2000

ENGEO — Expect Excellence —

REGIONAL GEOLOGIC MAP
3833 REDWOOD HIGHWAY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 14862.000.000
SCALE: AS SHOWN
DRAWN BY: GLJ | CHECKED BY: LC
FIGURE NO. 3

ORIGINAL FIGURE PRINTED IN COLOR

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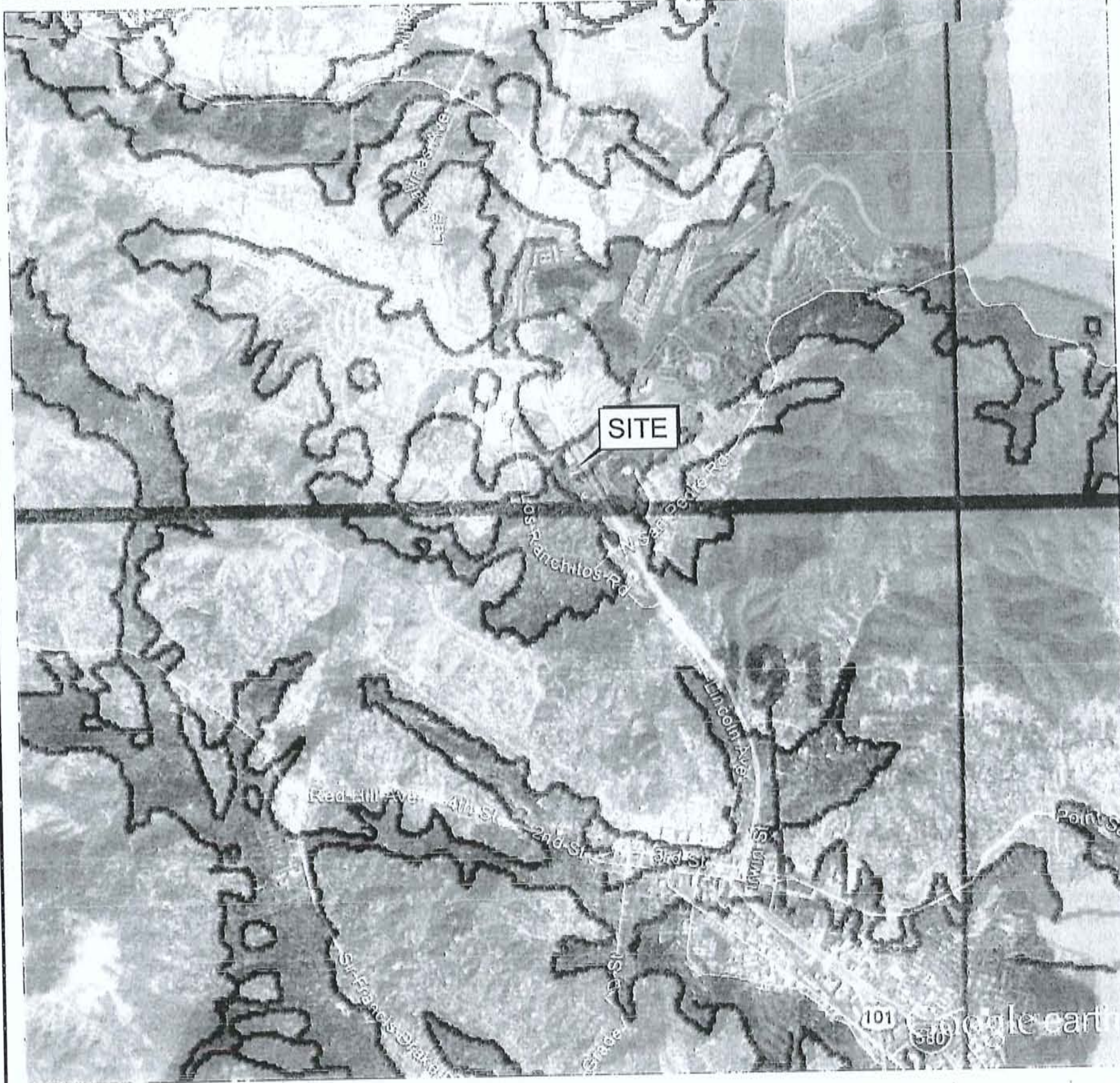

EXPLANATION

VERY HIGH SUSCEPTIBILITY		LOW	
HIGH		VERY LOW	
MODERATE			

N

0 FEET 4000
0 METERS 2000

BASE MAP SOURCE: KNUDSEN, et al., 2000 AND GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	REGIONAL LIQUEFACTION MAP 3833 REDWOOD HIGHWAY SAN RAFAEL, CALIFORNIA	PROJECT NO.: 14862.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: LC	FIGURE NO. 4

ORIGINAL FIGURE PRINTED IN COLOR

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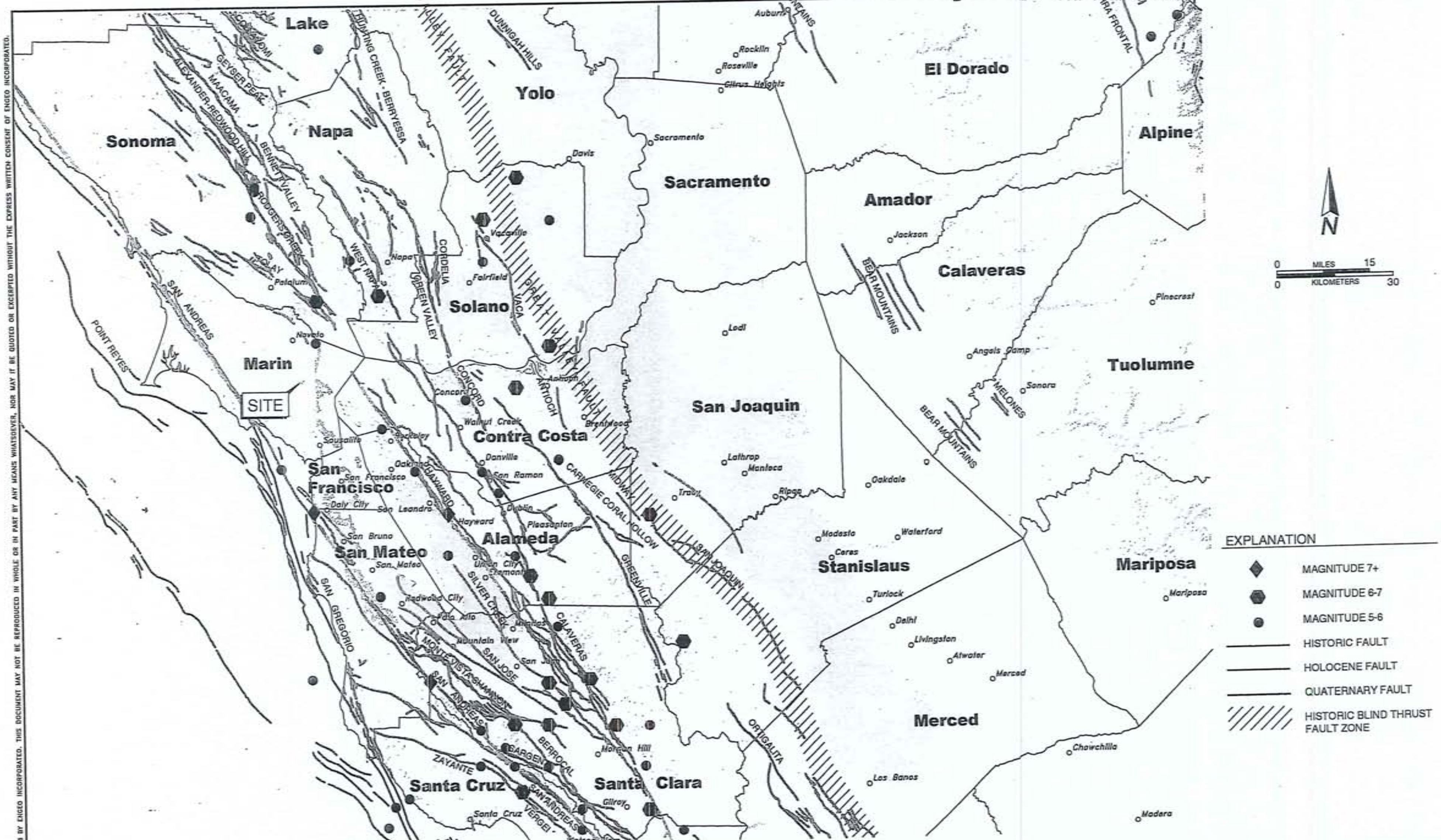

BASE MAP SOURCE:
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U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO — Expect Excellence —

REGIONAL FAULTING AND SEISMICITY
3833 REDWOOD HIGHWAY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 14862.000.000		FIGURE NO.
SCALE: AS SHOWN		5
DRAWN BY: GLJ	CHECKED BY: LC	

ORIGINAL FIGURE PRINTED IN COLOR

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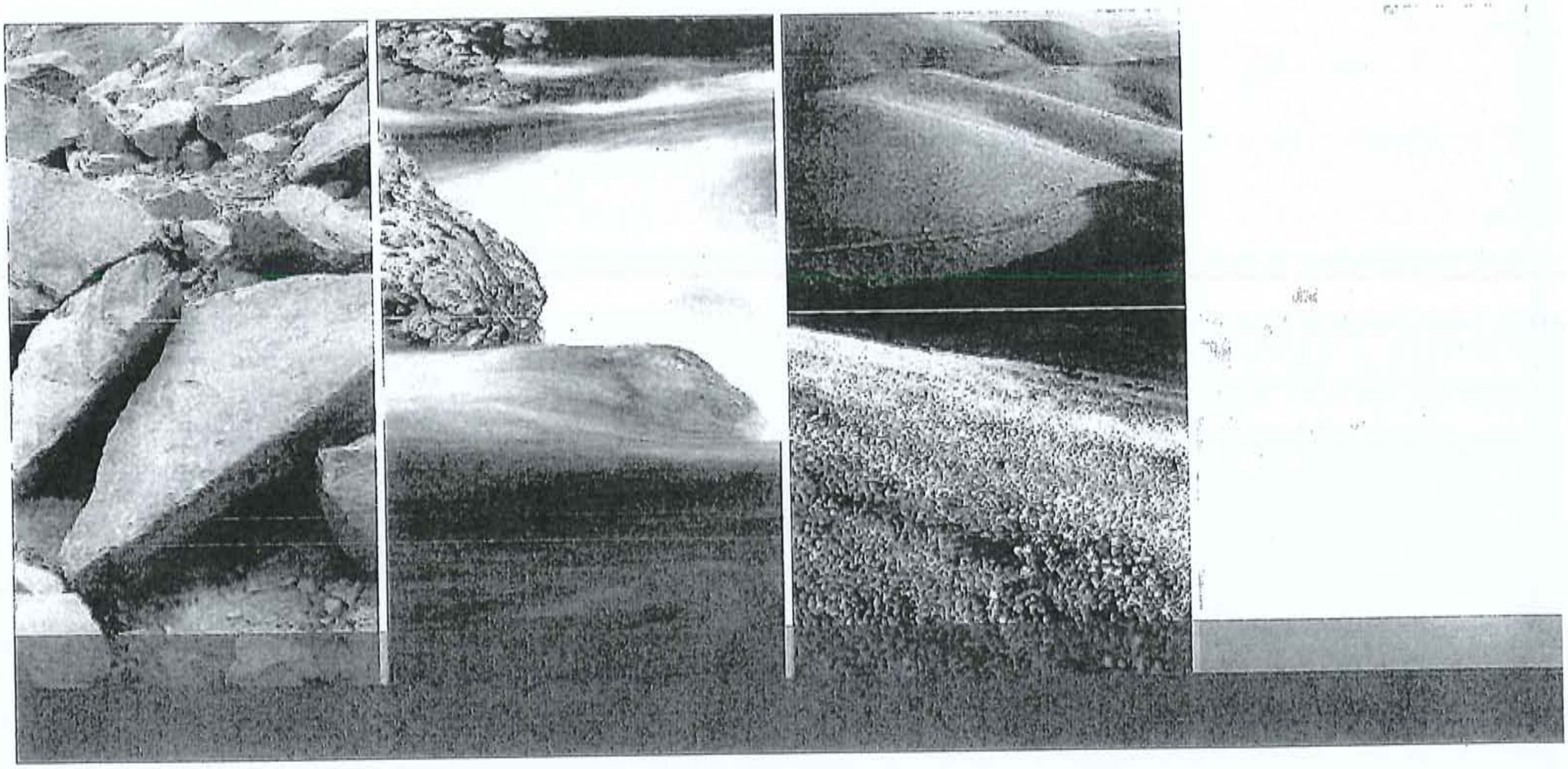


APPENDIX A

CONE PENETRATION TESTS (CPTS)

PRESENTATION OF SITE INVESTIGATION RESULTS

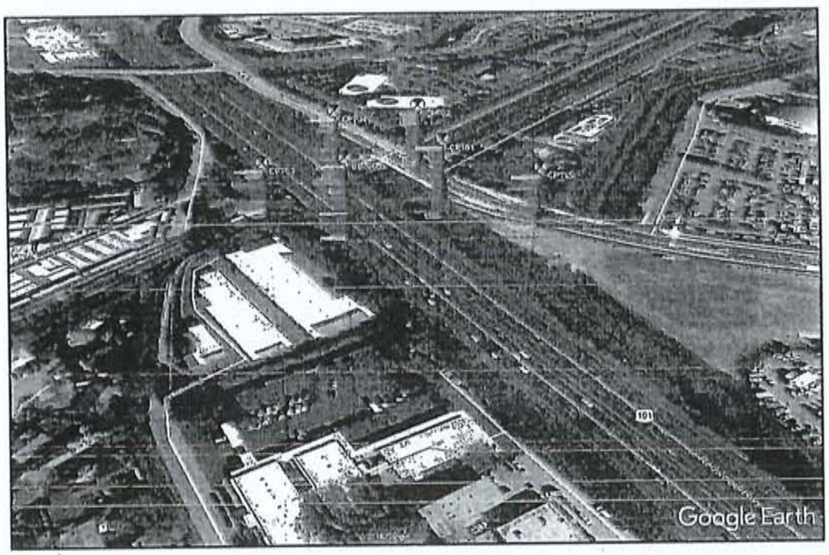
350 Merrydale Road

Prepared for:

ENGEO Inc.

CPT Inc. Job No: 18-56045

Project Start Date: 11-Apr-2018
Project End Date: 11-Apr-2018
Report Date: 12-Apr-2018



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CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

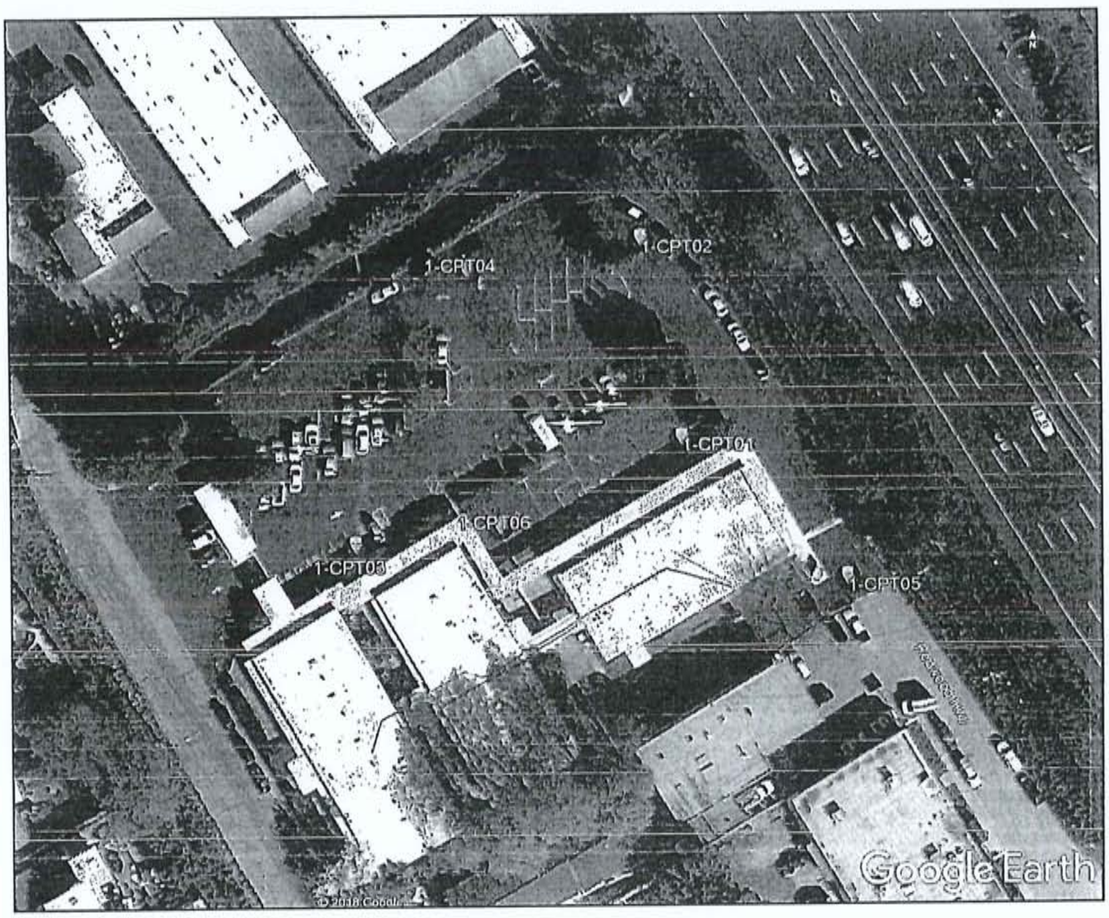
Introduction

The enclosed report presents the results of the site investigation program conducted by CPT Inc. for ENGEO Inc. at 350 Merrydale Road, San Rafael, CA. The program consisted of six cone penetration tests (CPT).

Project Information

Project	
Client	ENGEO Inc.
Project	350 Merrydale Road
CPT Inc. project number	18-56045

A map from Google earth including the CPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30 ton rig cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Reference
CPT	Consumer Grade GPS	32610

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

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
483:T1500F15U500	483	15	225	1500	15	500
Cone AD483 was used for all the CPT soundings.						

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

Limitations

This report has been prepared for the exclusive use of ENGEO Inc. (Client) for the project titled "350 Merrydale Road". The report's contents may not be relied upon by any other party without the express written permission of CPT Inc. CPT Inc. has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

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

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

CPT Inc.'s piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

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

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

All piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

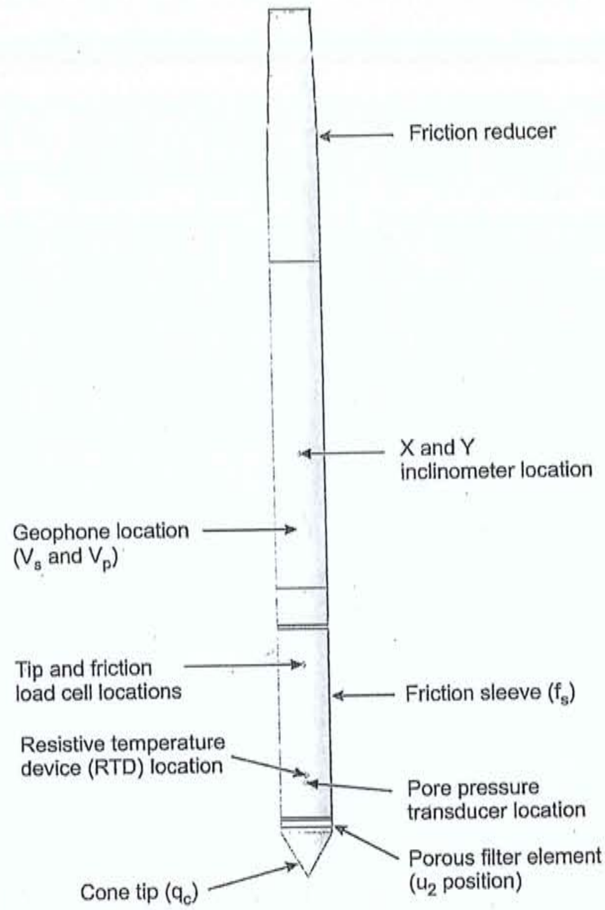


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

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

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

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

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to CPT Inc.'s cone penetration testing procedures:

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

The interpretation of the piezocone data and associated calculated parameters for this report are based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

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

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

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

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

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

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

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

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

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

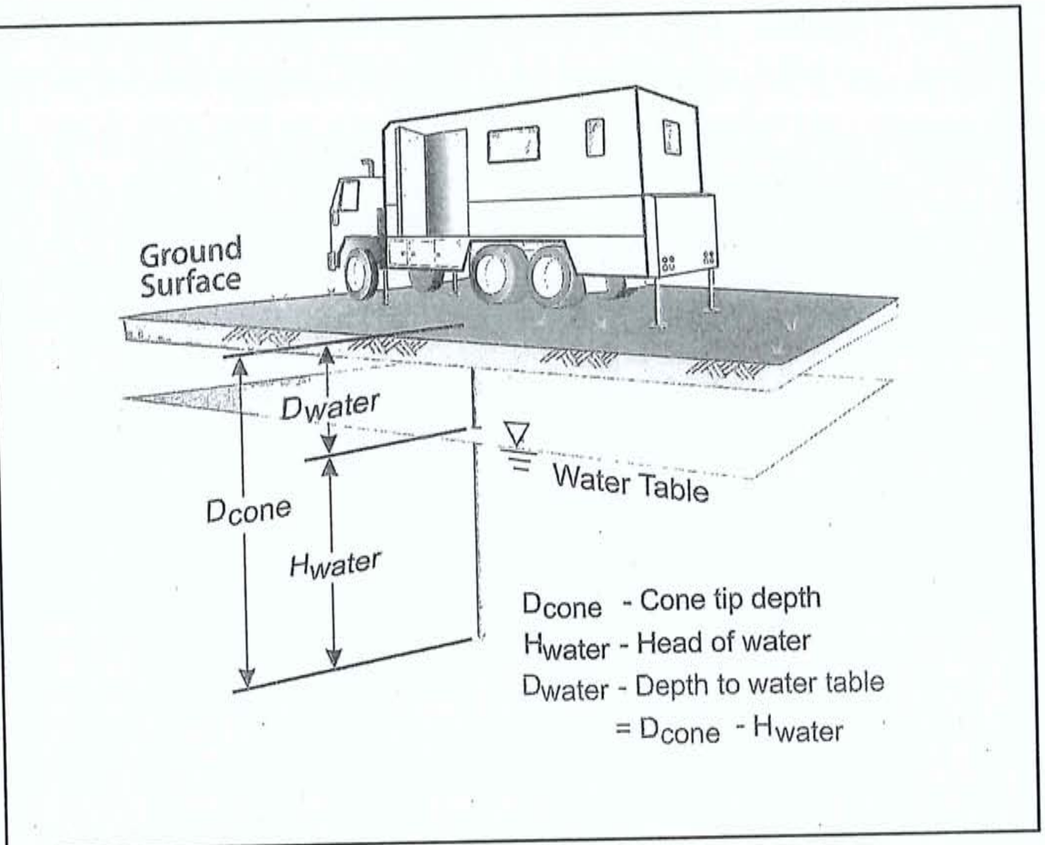


Figure PPD-1. Pore pressure dissipation test setup

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

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

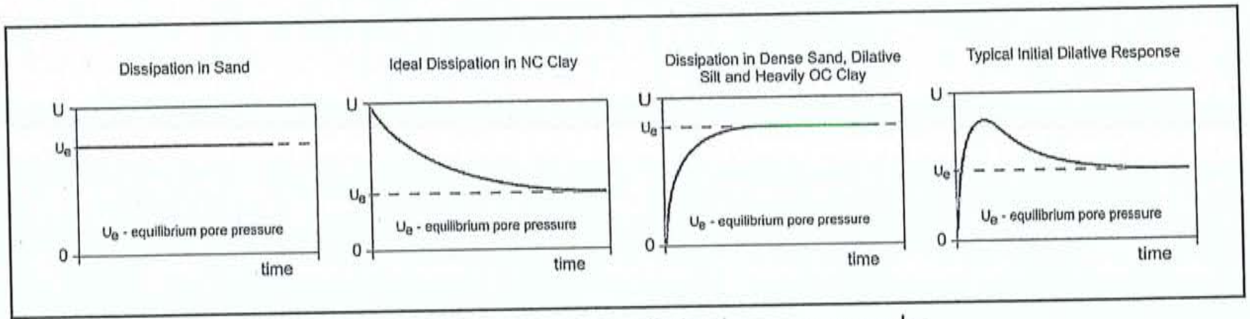


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDICES

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Plots
- Cone Penetration Test Standard Plots – Expanded Range
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt) and N1(60)
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Job No: 18-56045
Client: ENGEO Inc.
Project: 350 Merrydale Road
Start Date: 11-Apr-2018
End Date: 11-Apr-2018

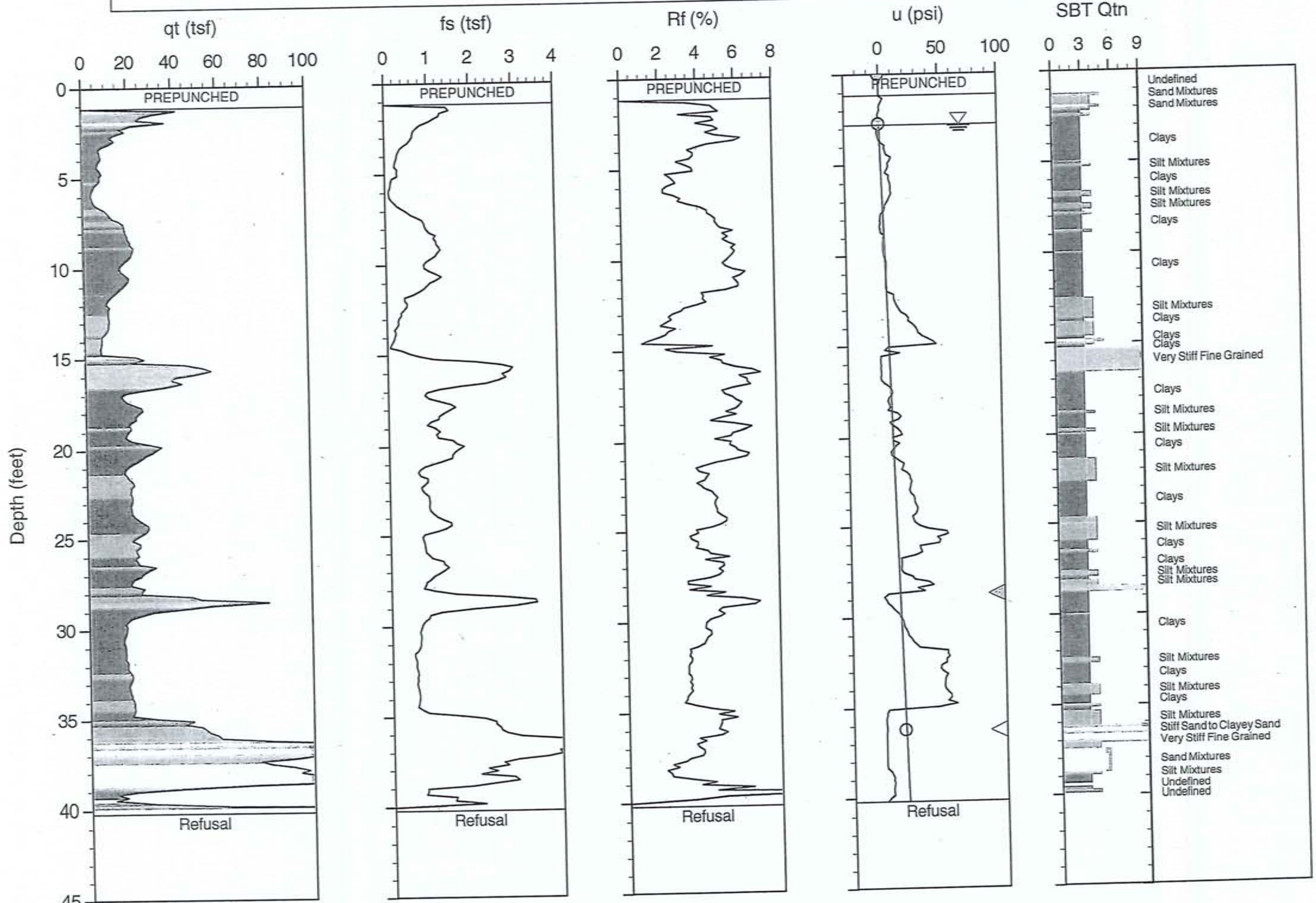
CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-CPT01	18-56045_CP01	11-Apr-2018	483:T1500F15U500	2.8	40.190	4205871	540660	
1-CPT02	18-56045_CP02	11-Apr-2018	483:T1500F15U500	3.7	32.152	4205903	540654	4
1-CPT03	18-56045_CP03	11-Apr-2018	483:T1500F15U500	4.0	22.966	4205854	540608	4
1-CPT04	18-56045_CP04	11-Apr-2018	483:T1500F15U500	3.7	36.581	4205898	540626	
1-CPT05	18-56045_CP05	11-Apr-2018	483:T1500F15U500	3.5	27.559	4205849	540686	3
1-CPT06	18-56045_CP06	11-Apr-2018	483:T1500F15U500	4.0	39.862	4205857	540631	

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

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 2018-04-11 09:48 Site: 350 Merrydale Road	Sounding: 1-CPT01 Cone: 483:T1500F15U500



Max Depth: 12.250 m / 40.19 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 18-56045_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205871m E: 540660m
Page No: 1 of 1

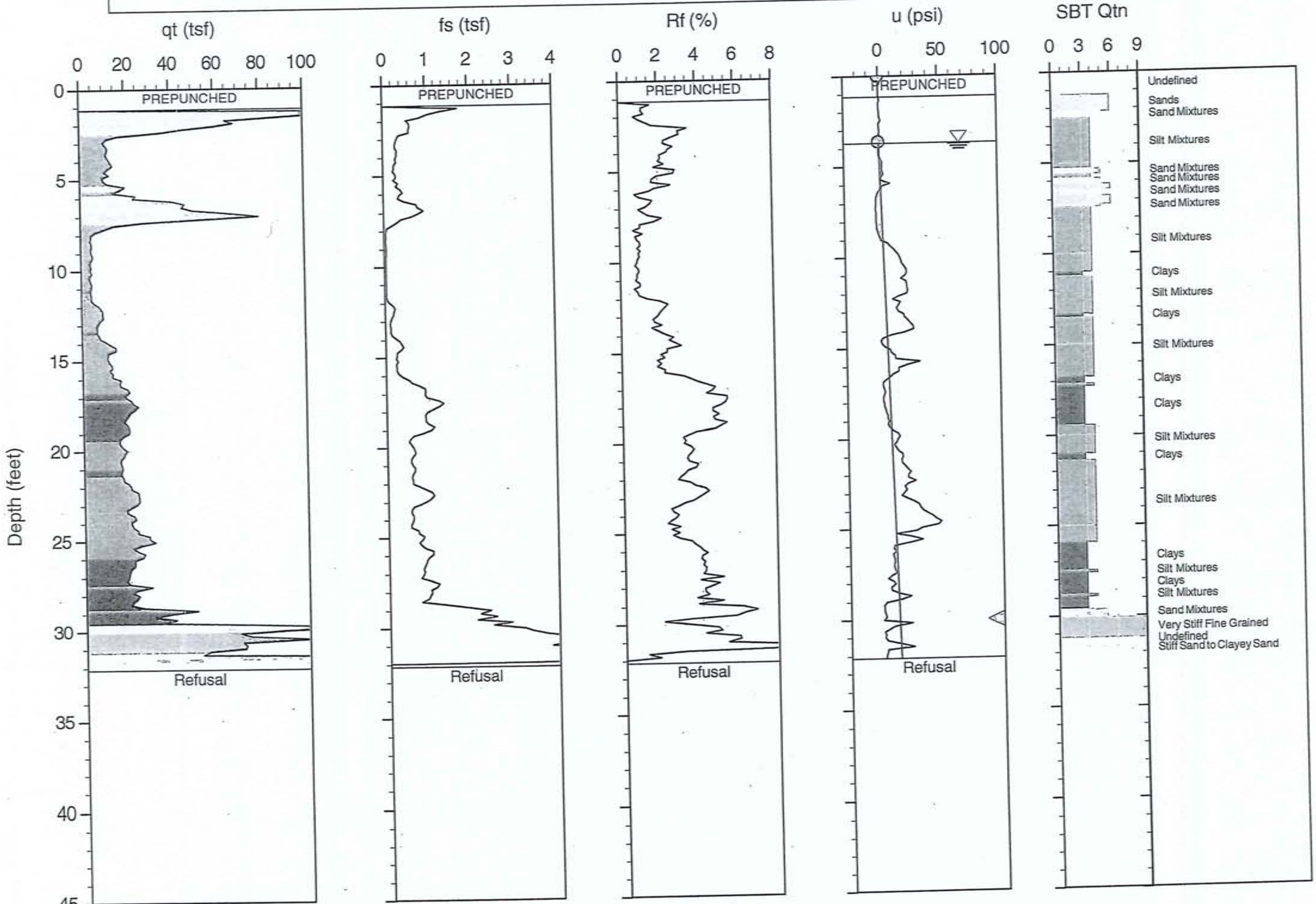
Overplot Item: ○ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 09:05
Site: 350 Merrydale Road

Sounding: 1-CPT02
Cone: 483:T1500F15U500



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Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ◎ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP02.COR
Unit Wt: SBTQtn (PKR2009)

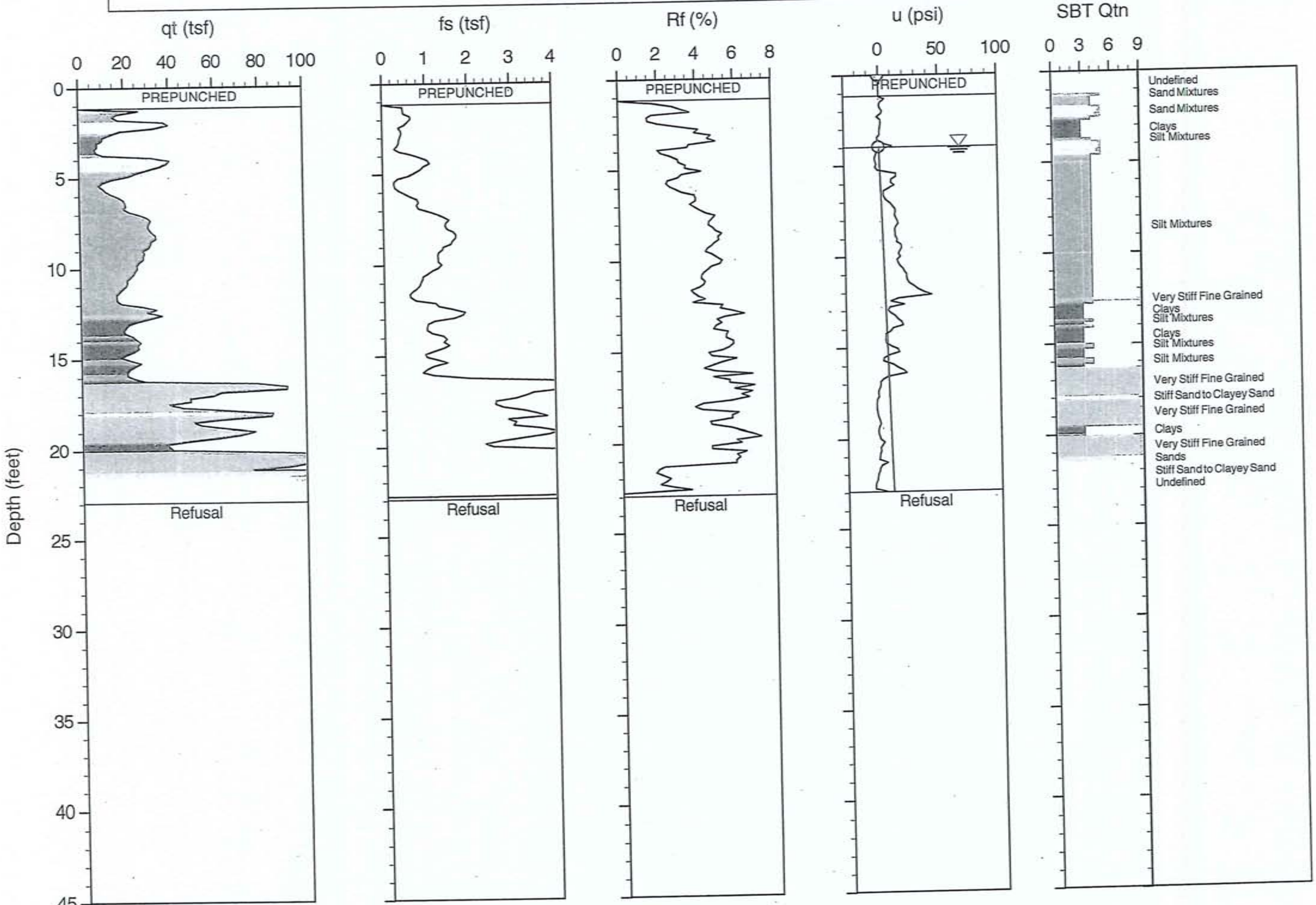
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205903m E: 540654m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 15:50
Site: 350 Merrydale Road

Sounding: 1-CPT03
Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item: ⊙ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP03.COR
Unit Wt: SBTQtn (PKR2009)

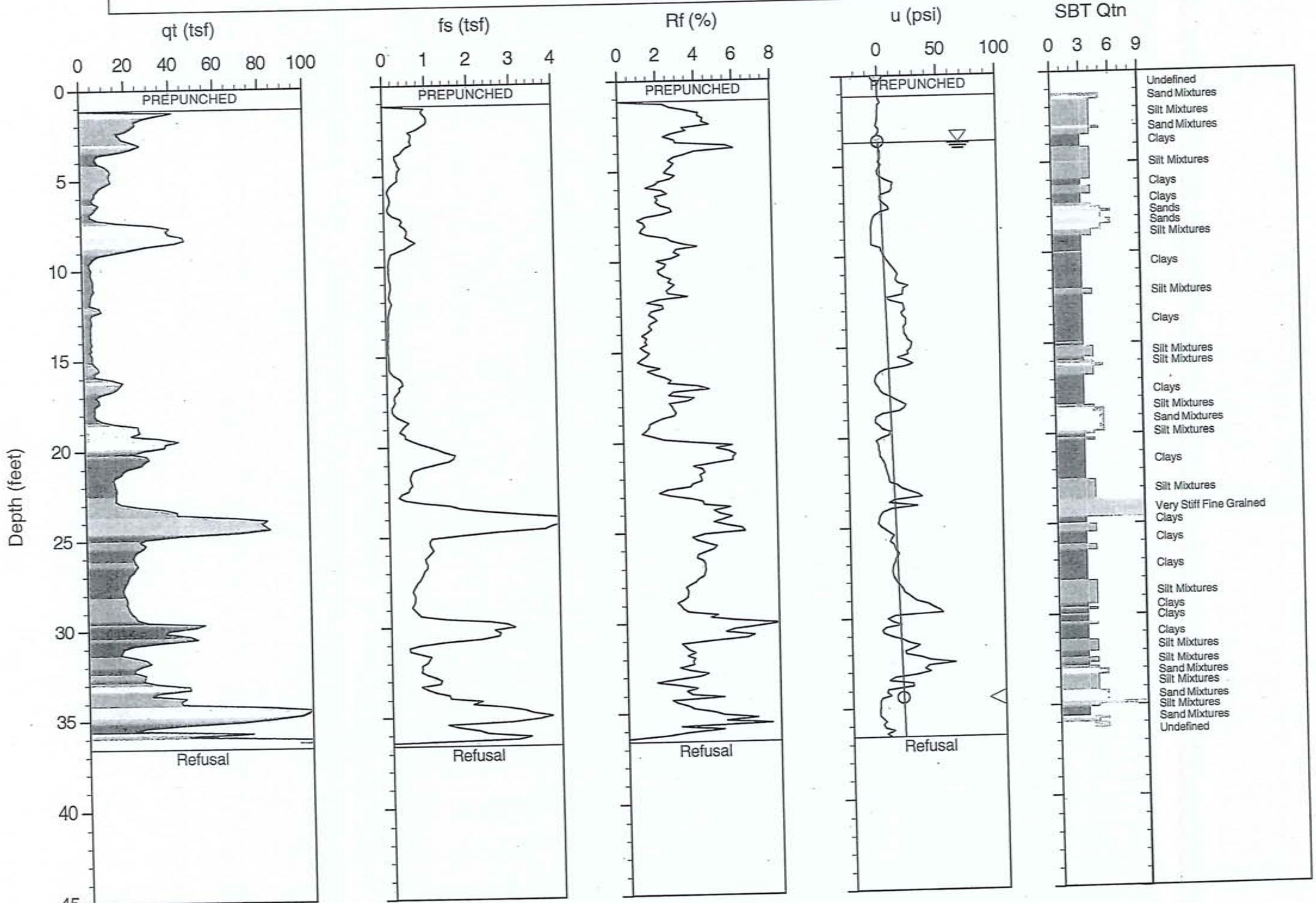
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205854m E: 540608m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 11:35
Site: 350 Merrydale Road

Sounding: 1-CPT04
Cone: 483:T1500F15U500



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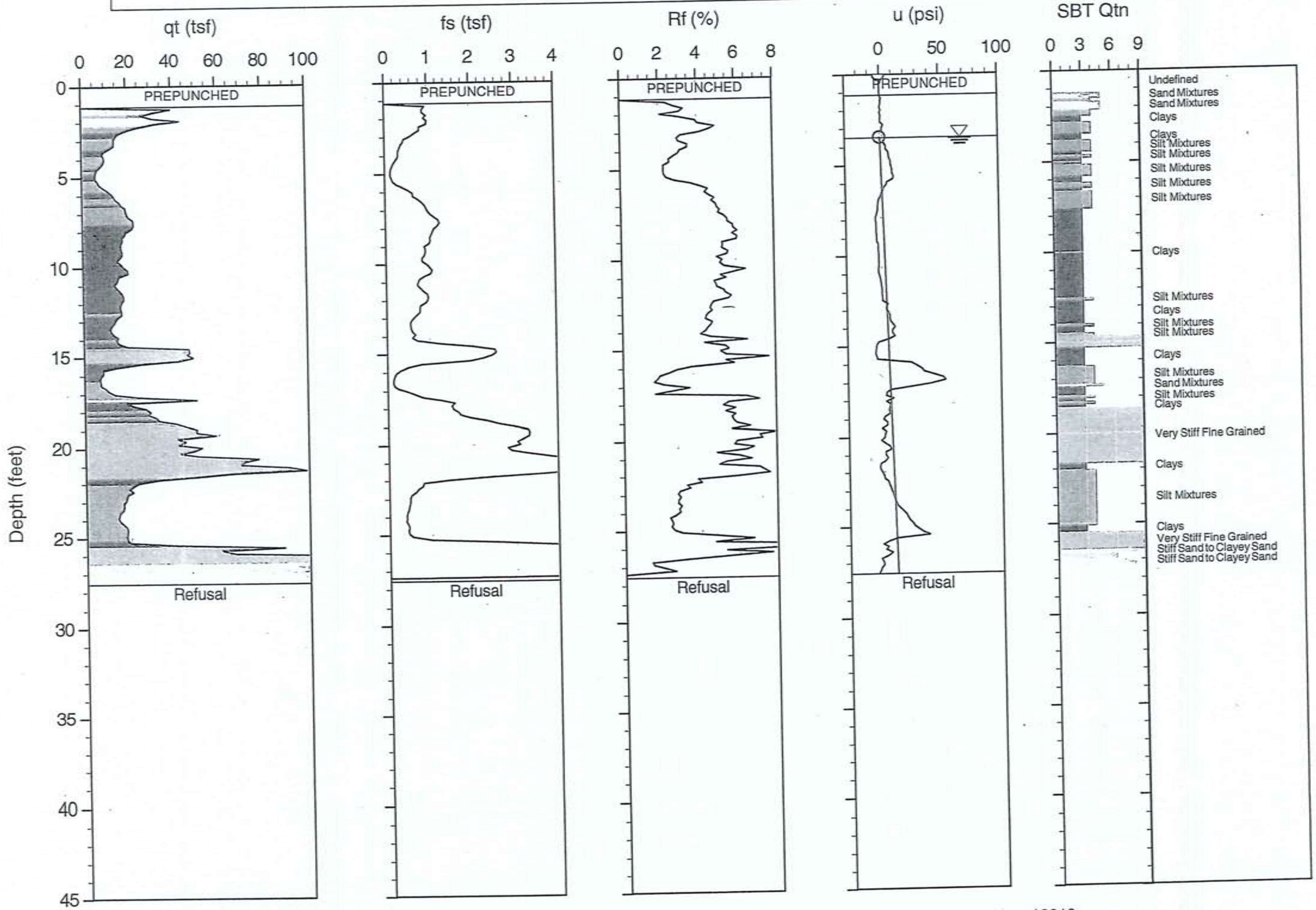
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ⊖ Assumed Ueq

File: 18-56045_CP04.COR
Unit Wt: SBTQtn (PKR2009)
◁ Dissipation, equilibrium achieved

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205898m E: 540626m
Page No: 1 of 1
—— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 2018-04-11 12:46 Site: 350 Merrydale Road	Sounding: 1-CPT05 Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item:

File: 18-56045_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205849m E: 540686m
Page No: 1 of 1

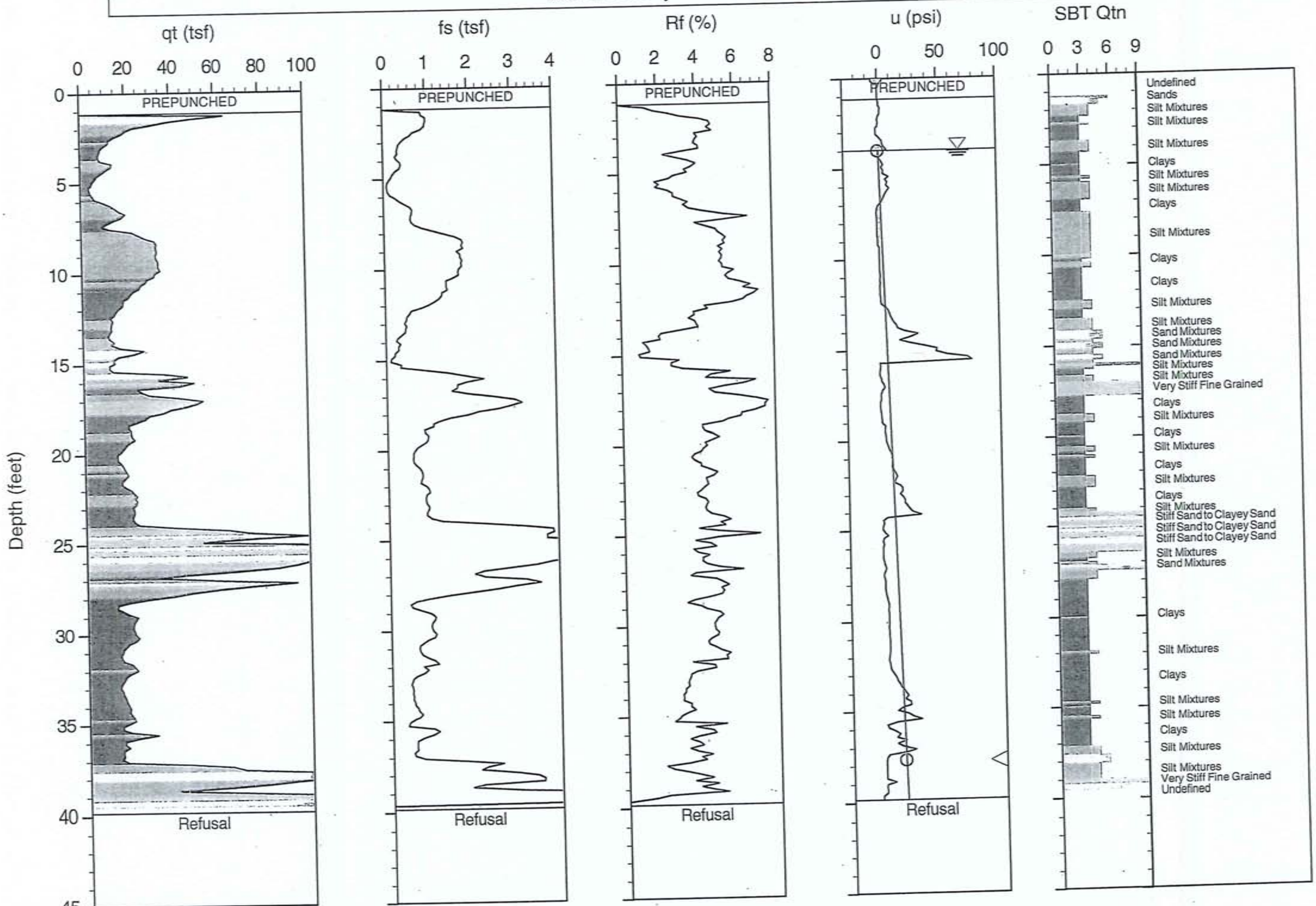
⊚ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 10:50
Site: 350 Merrydale Road

Sounding: 1-CPT06
Cone: 483:T1500F15U500



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Avg Int: Every Point

File: 18-56045_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205857m E: 540631m
Page No: 1 of 1

Overplot Item: ◎ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

Cone Penetration Test Standard Plots – Expanded Range

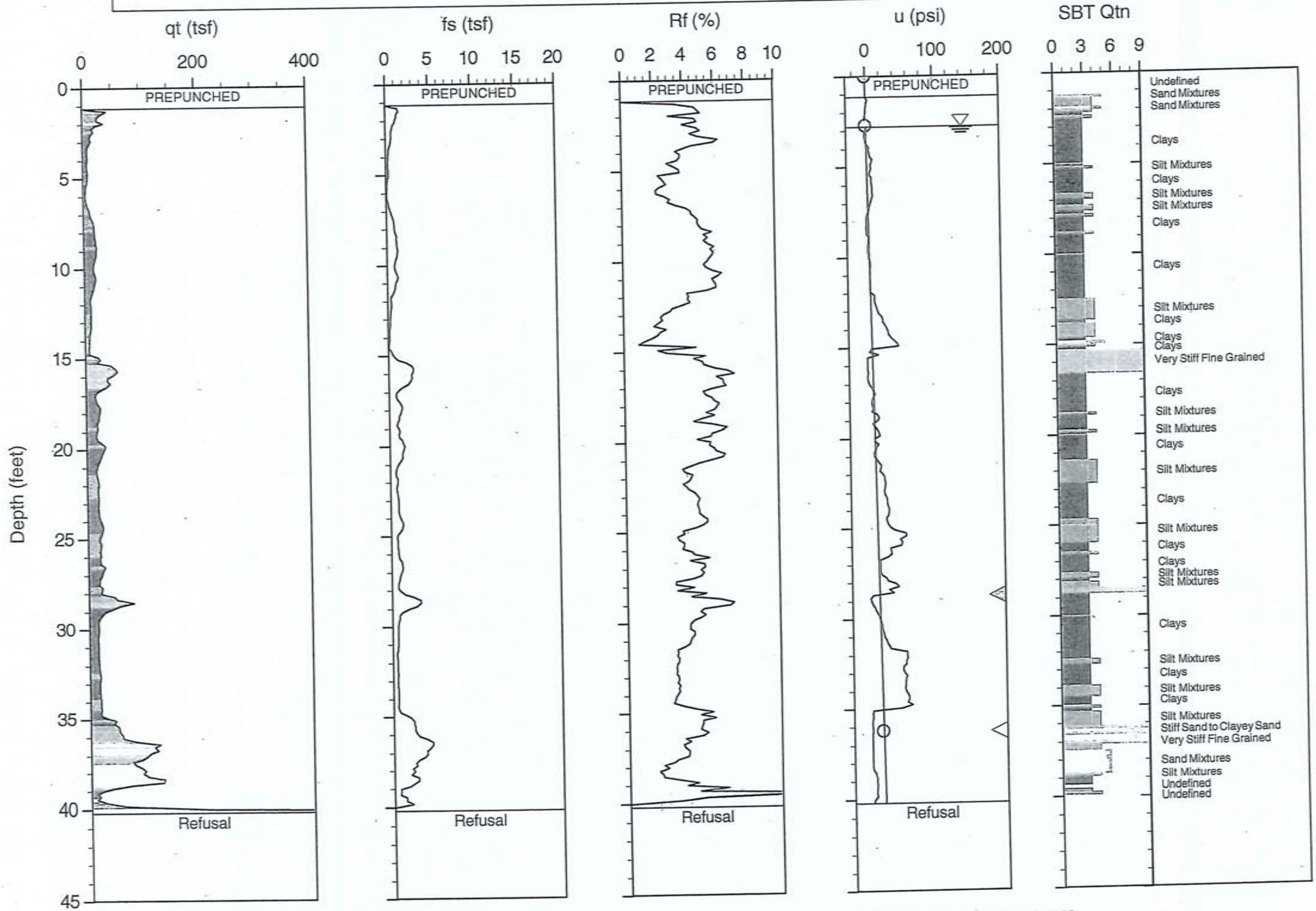
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 09:48
Site: 350 Merrydale Road

Sounding: 1-CPT01
Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item:

File: 18-56045_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205871m E: 540660m
Page No: 1 of 1

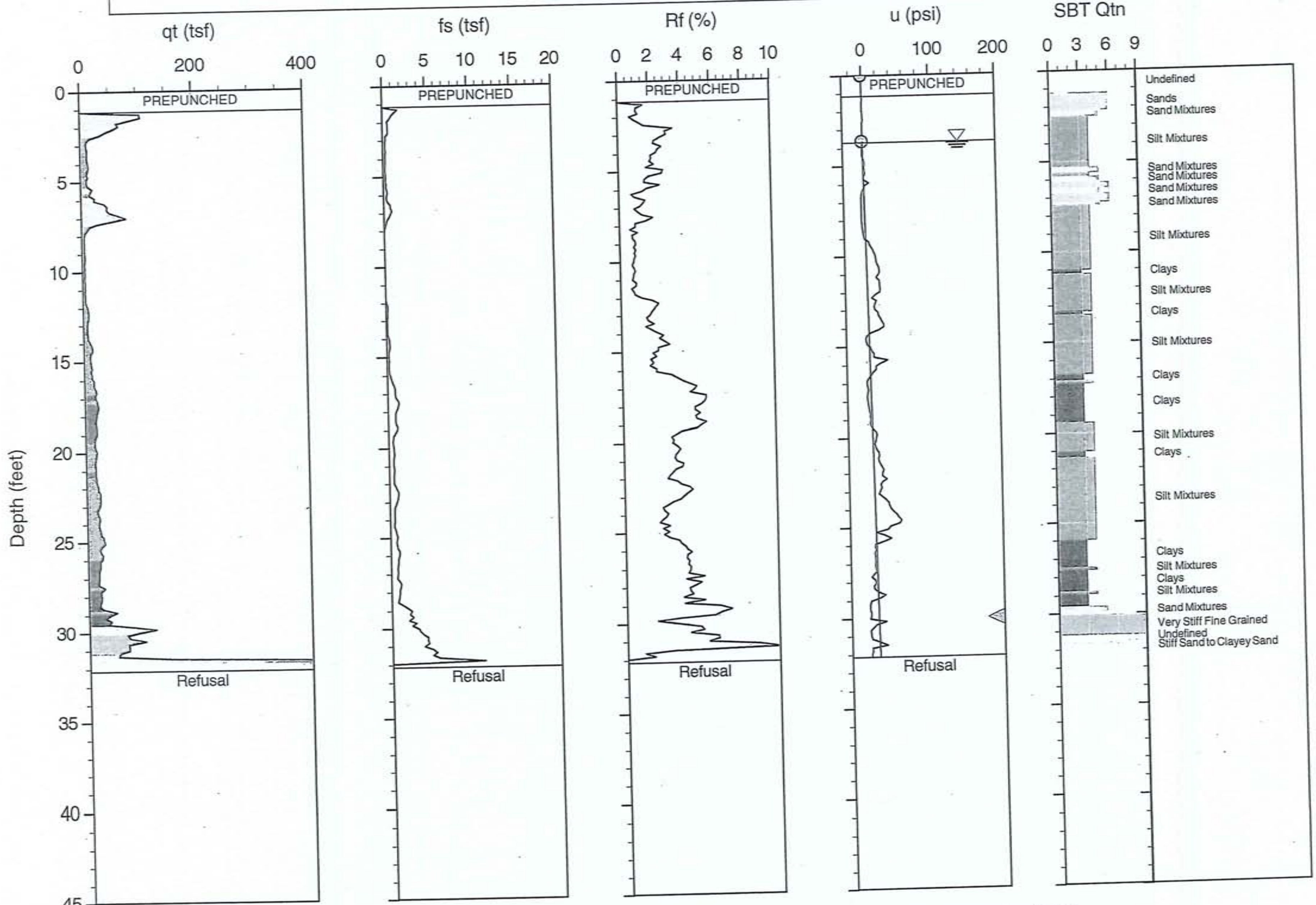
Assumed Ueq Dissipation equilibrium achieved Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 09:05
Site: 350 Merrydale Road

Sounding: 1-CPT02
Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item:

File: 18-56045_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205903m E: 540654m
Page No: 1 of 1

⊗ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 15:50
Site: 350 Merrydale Road

Sounding: 1-CPT03
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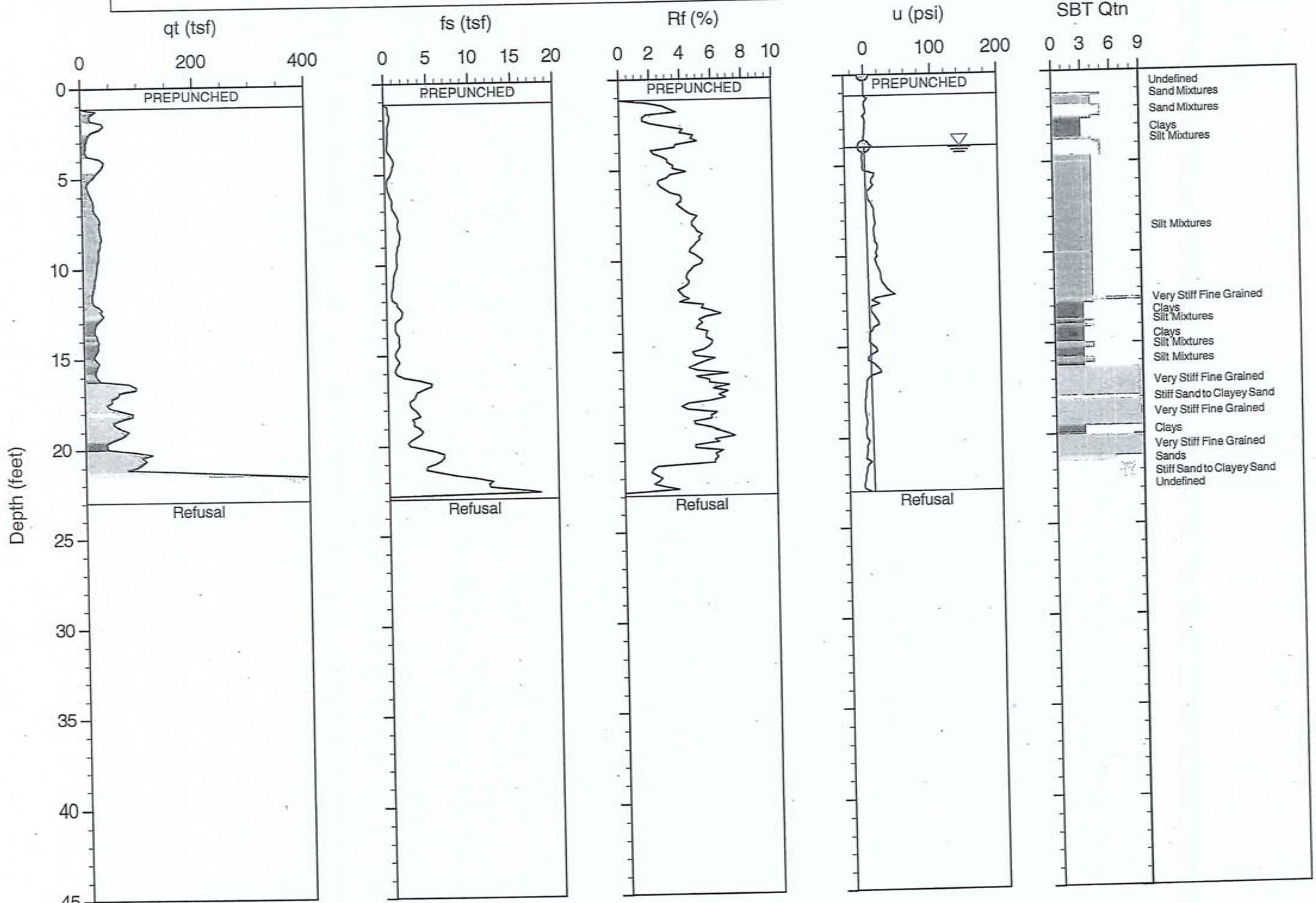

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Avg Int: Every Point
Overplot Item: ◎ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205854m E: 540608m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 11:35
Site: 350 Merrydale Road

Sounding: 1-CPT04
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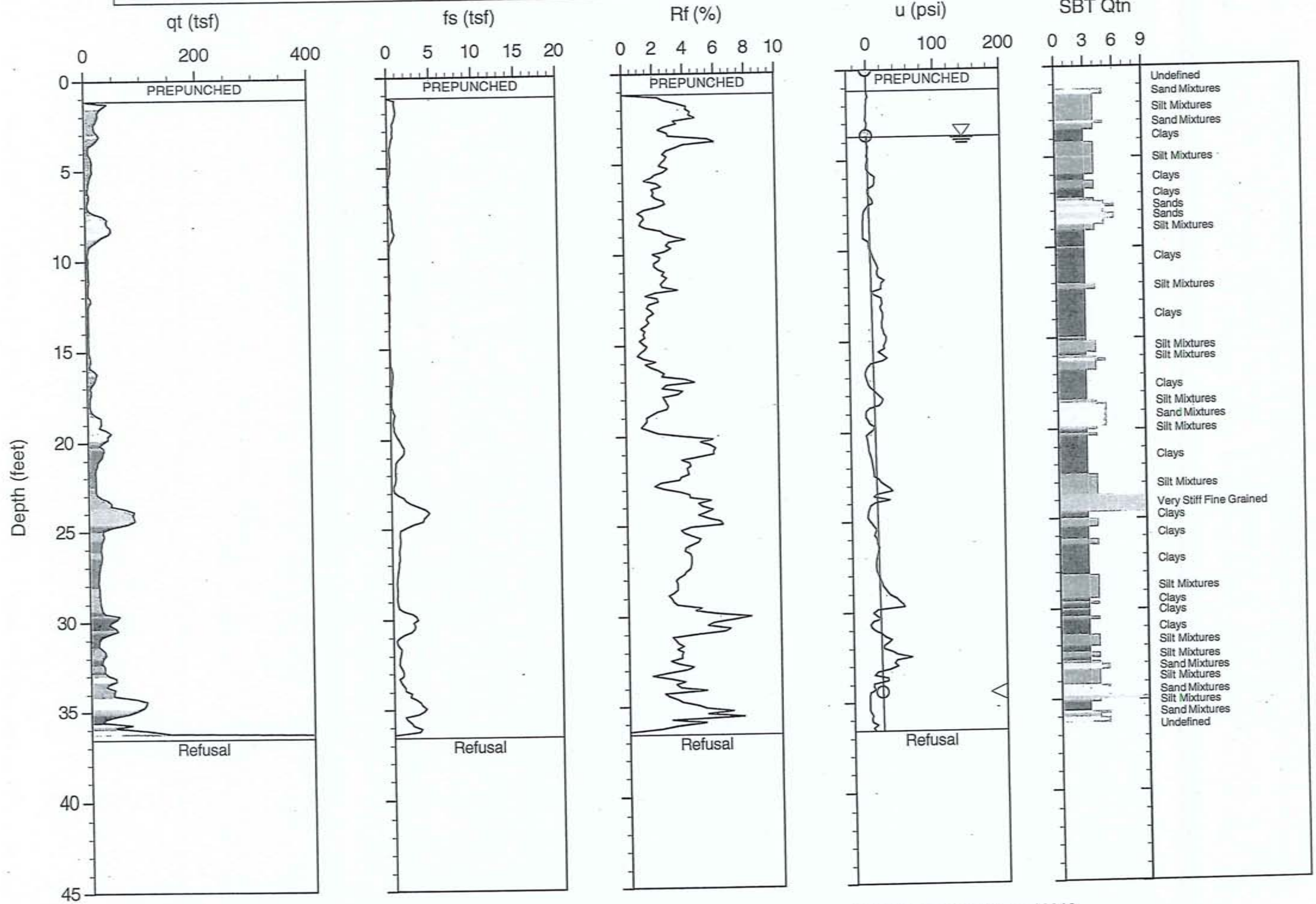

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Avg Int: Every Point

File: 18-56045_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205898m E: 540626m
Page No: 1 of 1

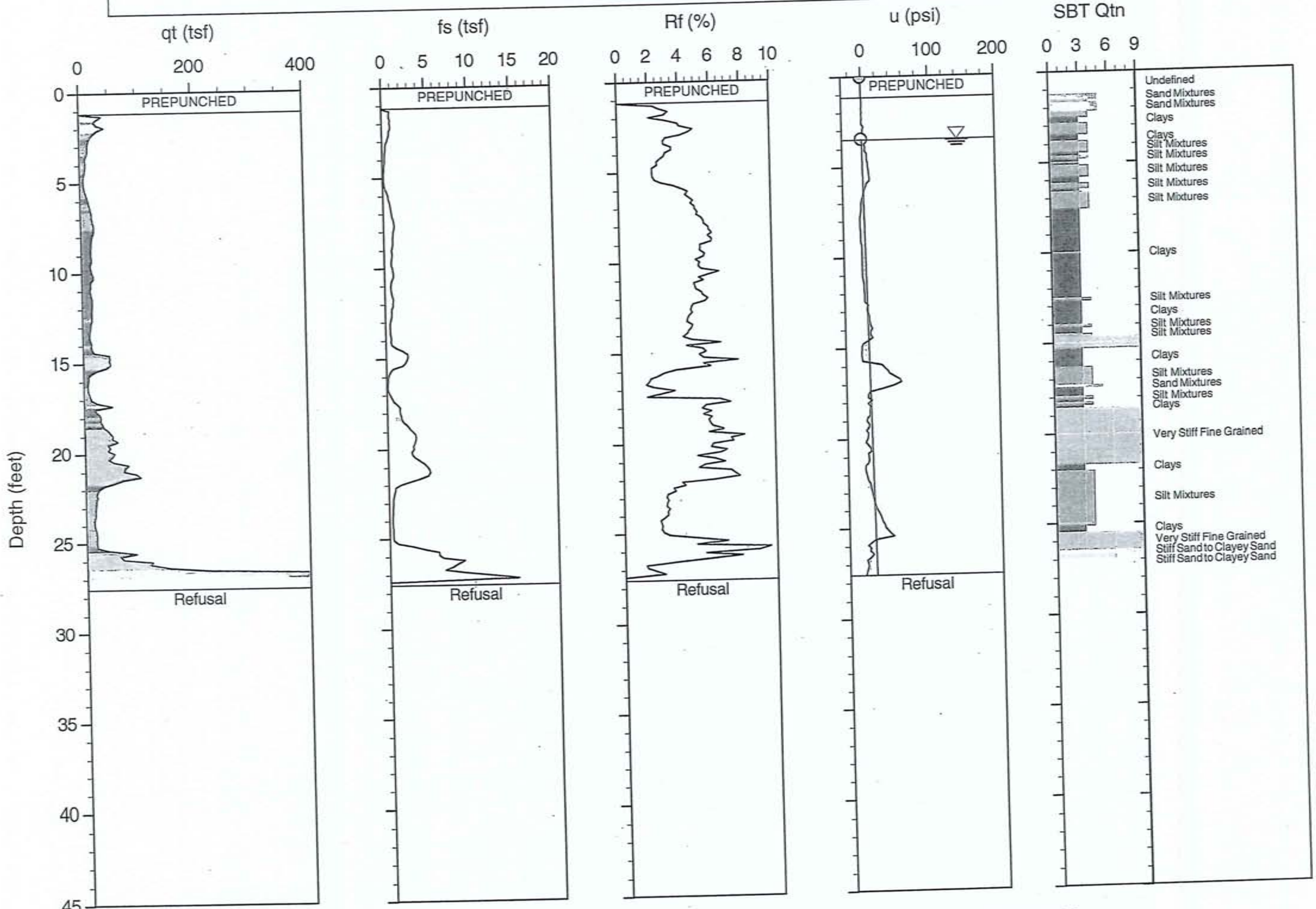
Overplot Item: ⊙ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 12:46
Site: 350 Merrydale Road

Sounding: 1-CPT05
Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item: ○ Assumed Ueq ◁ Dissipation, equilibrium achieved — Hydrostatic Line

File: 18-56045_CP05.COR
Unit Wt: SBTQtn (PKR2009)

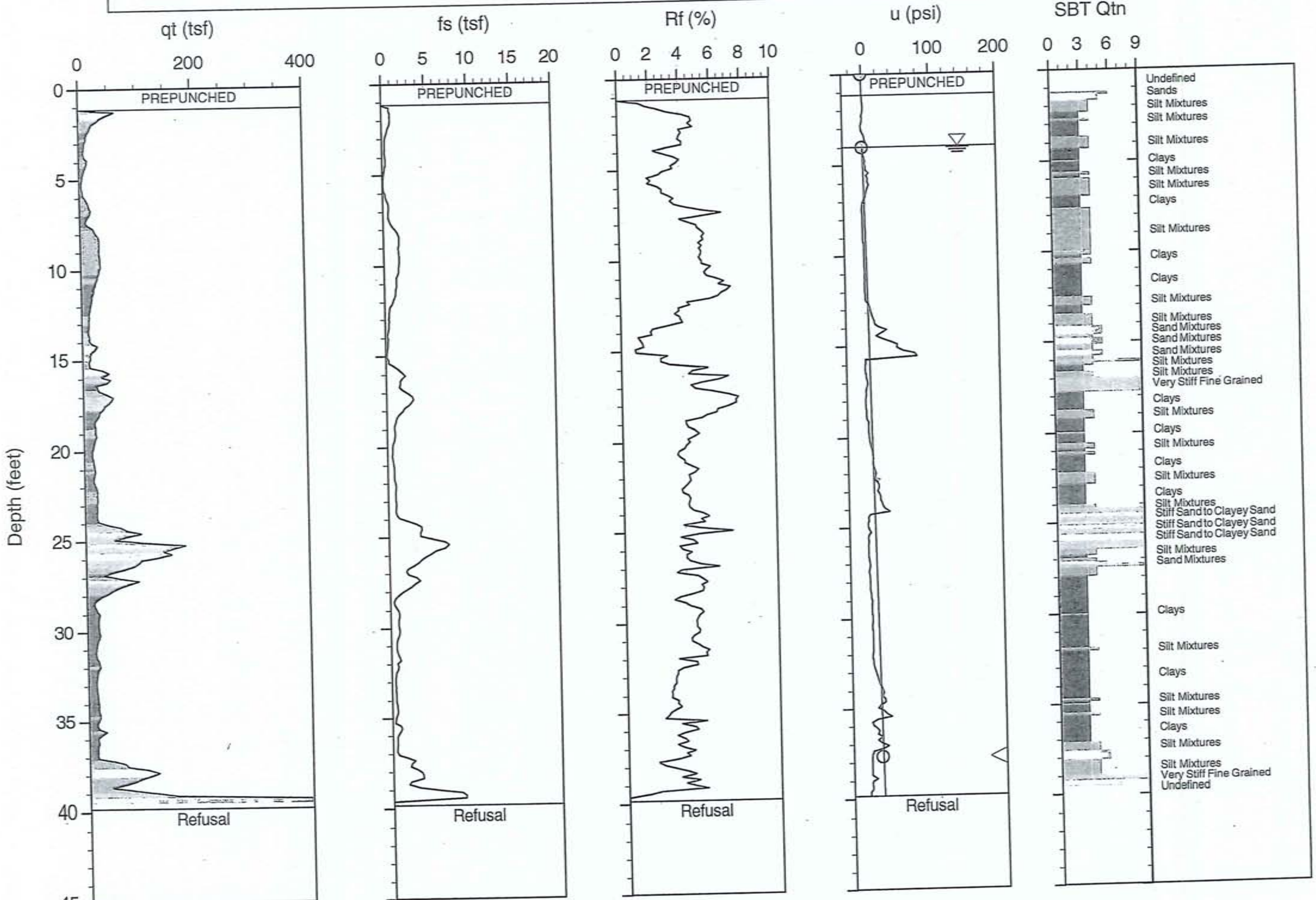
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205849m E: 540686m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 10:50
Site: 350 Merrydale Road

Sounding: 1-CPT06
Cone: 483:T1500F15U500



Max Depth: 12.150 m / 39.86 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ○ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205857m E: 540631m
Page No: 1 of 1

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

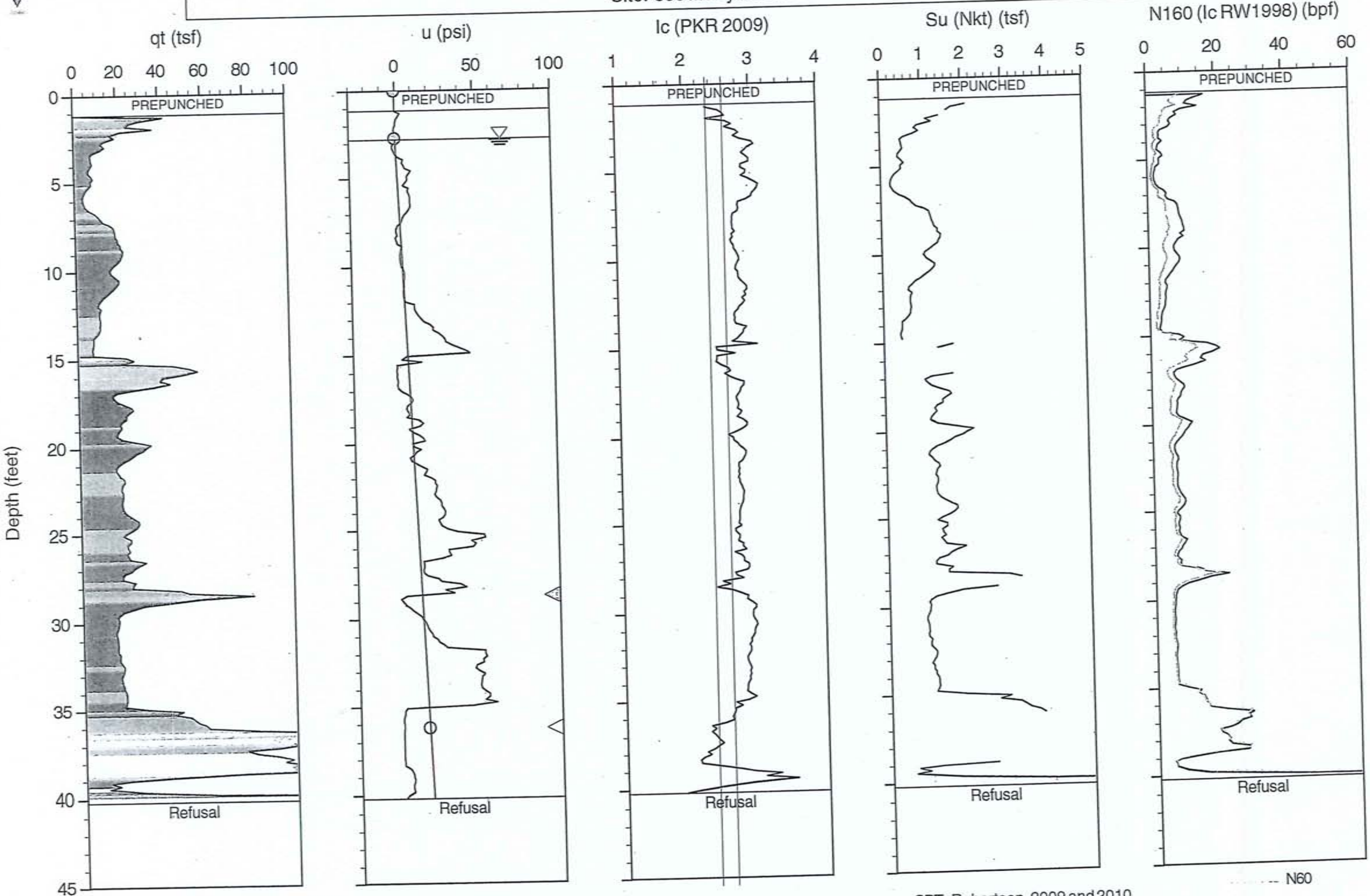
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 09:48
Site: 350 Merrydale Road

Sounding: 1-CPT01
Cone: 483:T1500F15U500



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Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item:

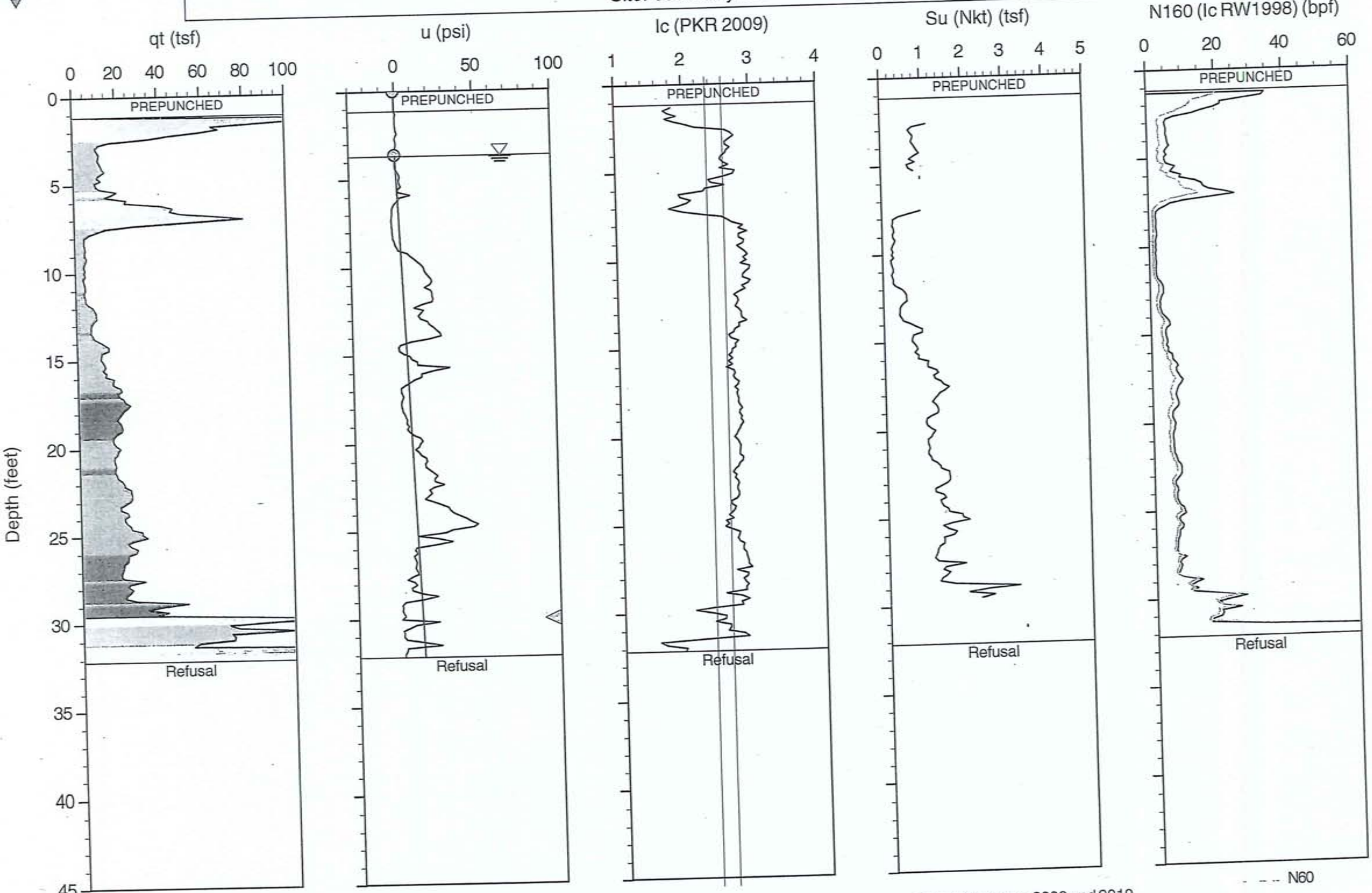
File: 18-56045_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205871m E: 540660m
Page No: 1 of 1

⊖ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 2018-04-11 09:05 Site: 350 Merrydale Road	Sounding: 1-CPT02 Cone: 483:T1500F15U500



Max Depth: 9.800 m / 32.15 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ⊙ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

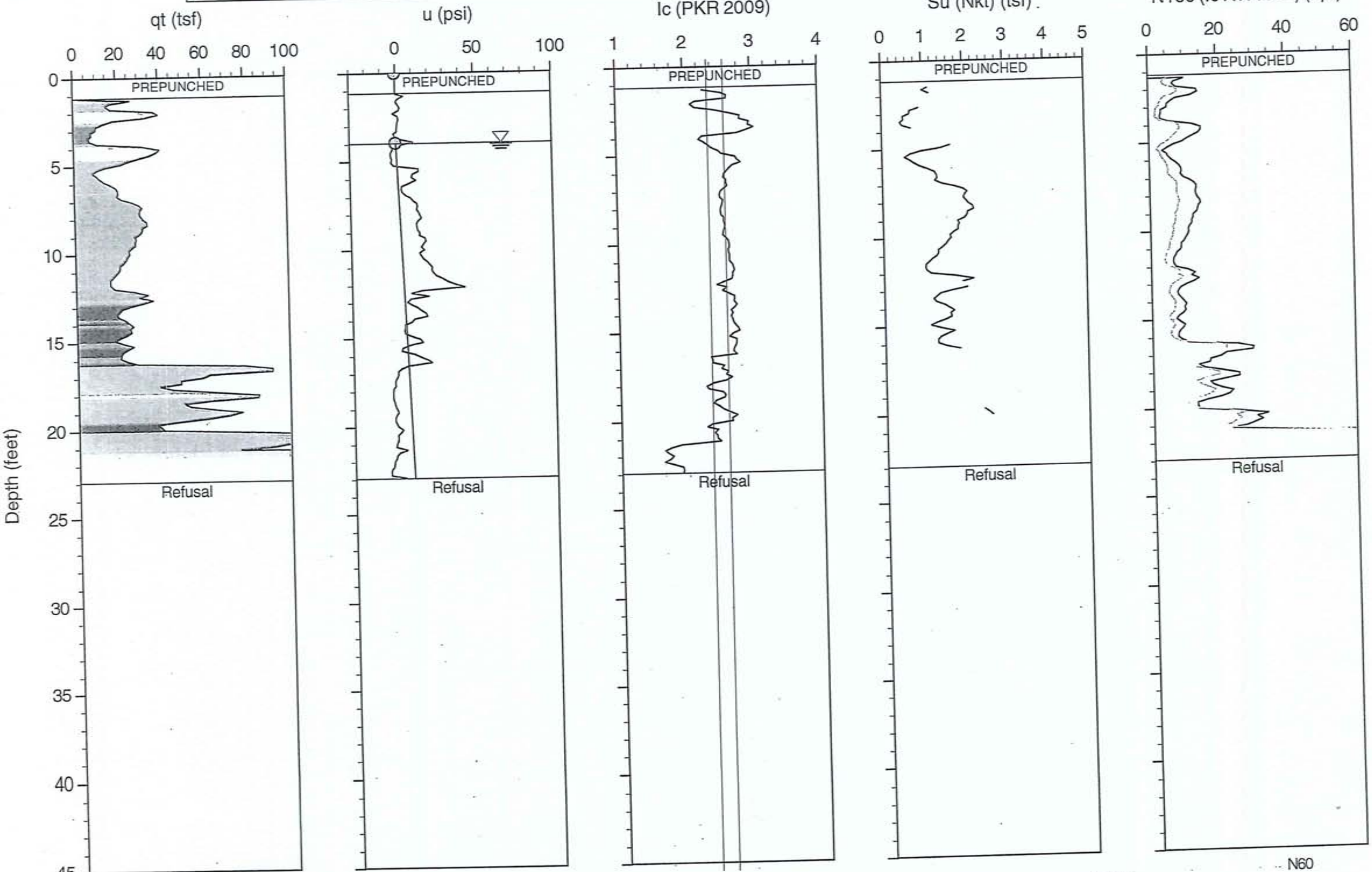
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205903m E: 540654m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 15:50
Site: 350 Merrydale Road

Sounding: 1-CPT03
Cone: 483:T1500F15U500



Max Depth: 7.000 m / 22.97 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ○ Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

File: 18-56045_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

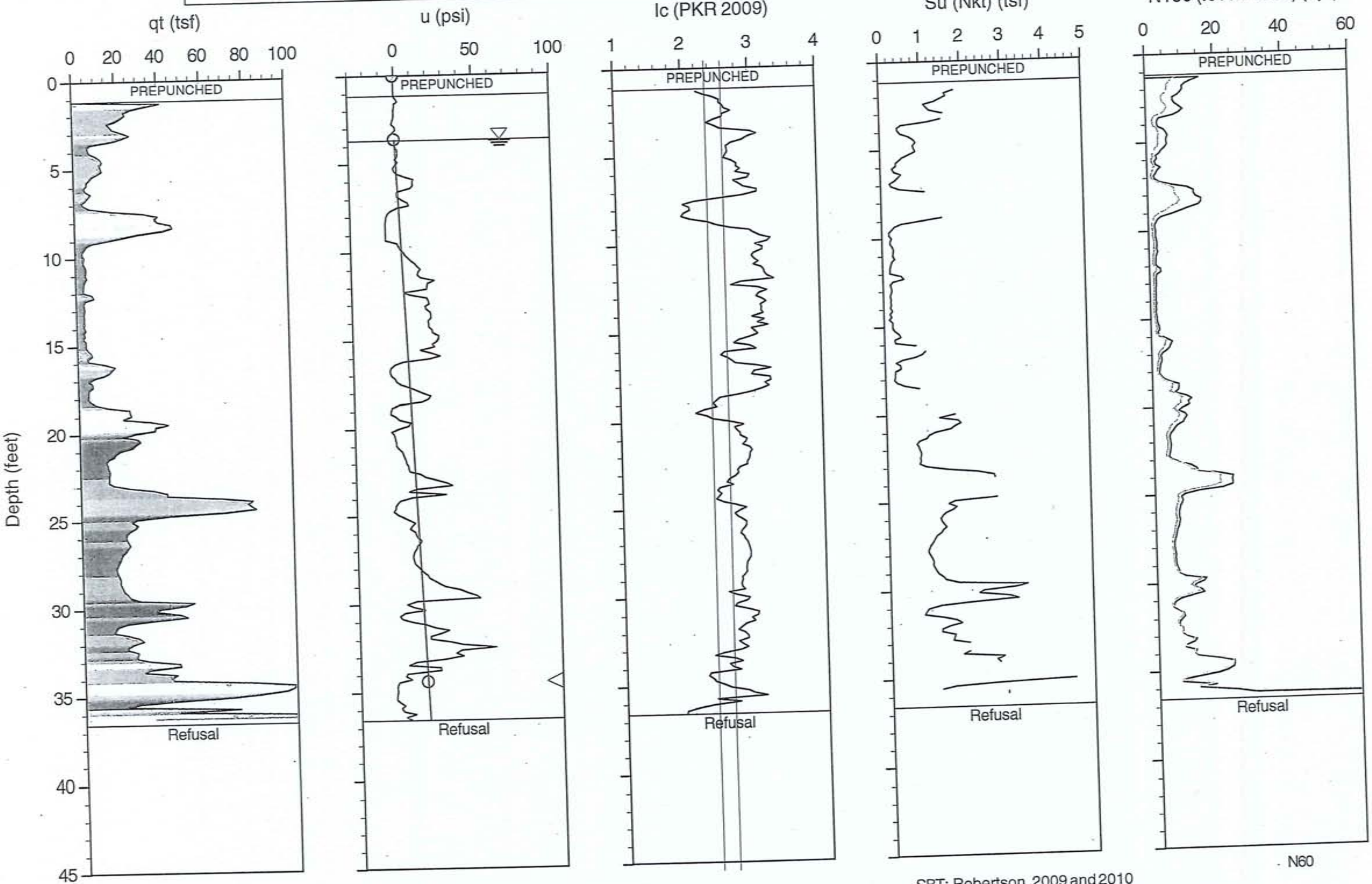
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205854m E: 540608m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 11:35
Site: 350 Merrydale Road

Sounding: 1-CPT04
Cone: 483:T1500F15U500



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Avg Int: Every Point
Overplot Item:

File: 18-56045_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205898m E: 540626m
Page No: 1 of 1

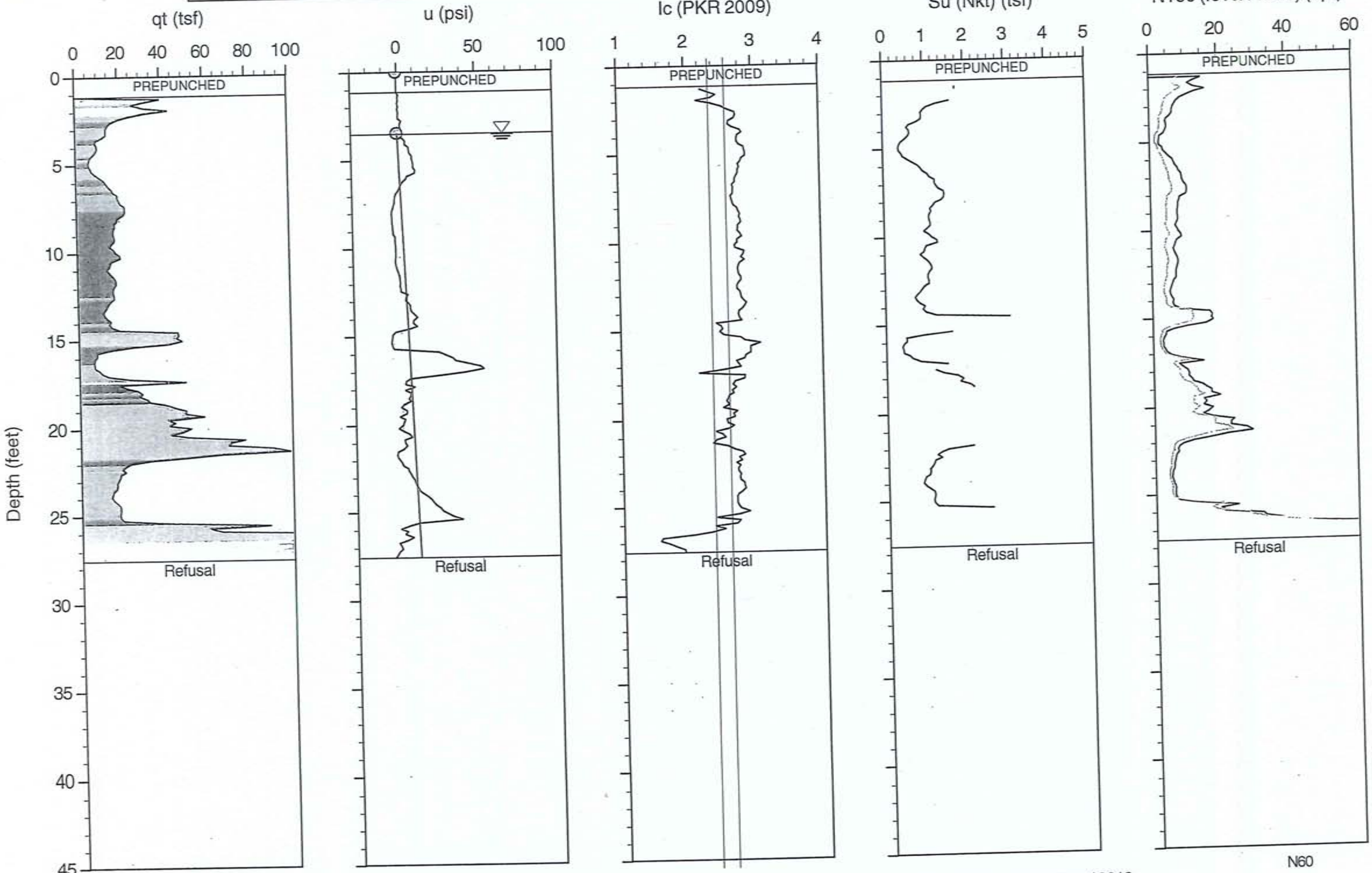
⊖ Assumed Ueq ◁ Dissipation equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 12:46
Site: 350 Merrydale Road

Sounding: 1-CPT05
Cone: 483:T1500F15U500



Max Depth: 8.400 m / 27.56 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ⊜ Assumed Ueq

File: 18-56045_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
◁ Dissipation, equilibrium achieved

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205849m E: 540686m
Page No: 1 of 1

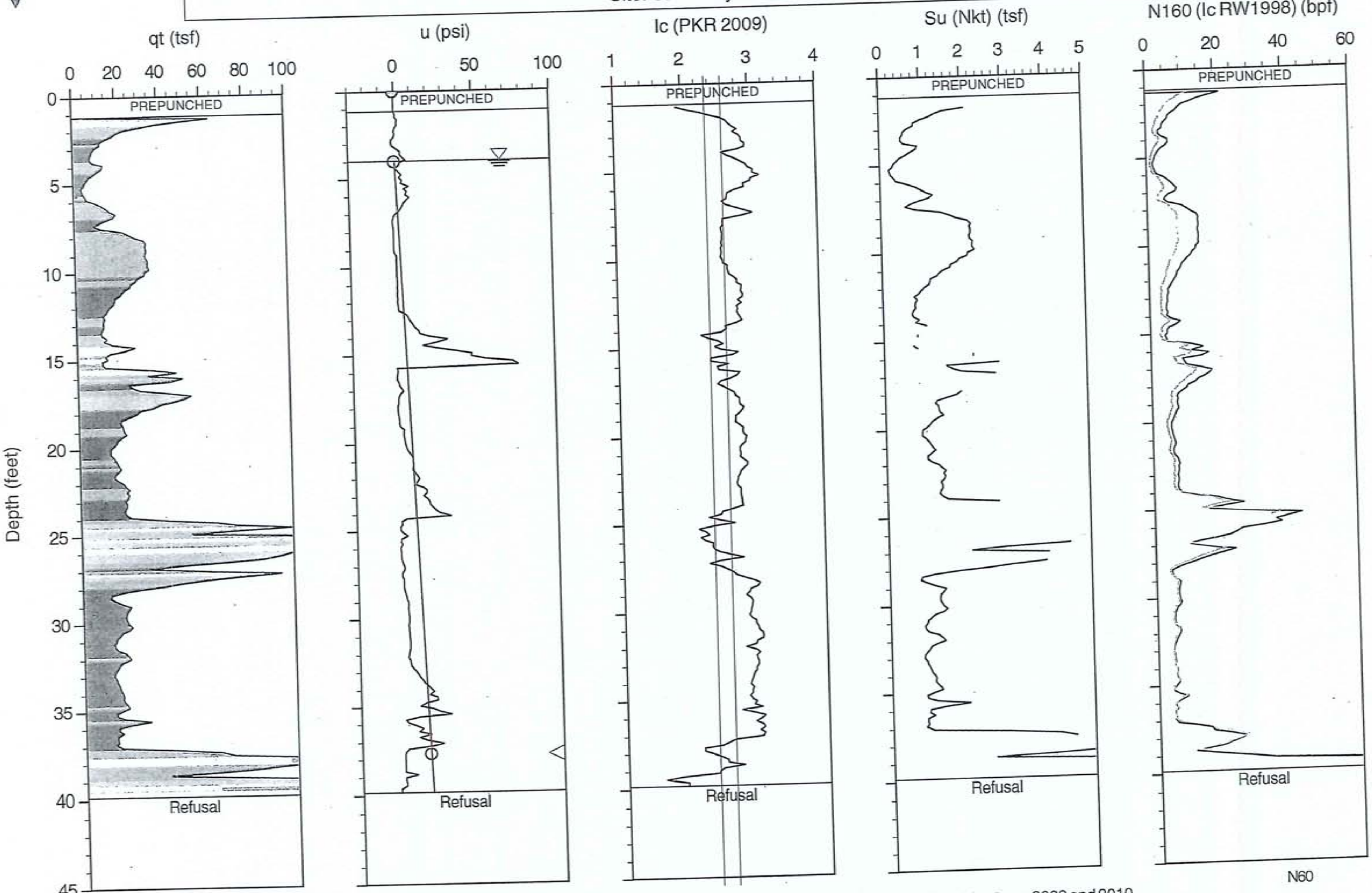
—— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 10:50
Site: 350 Merrydale Road

Sounding: 1-CPT06
Cone: 483:T1500F15U500



Max Depth: 12.150 m / 39.86 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item:

File: 18-56045_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

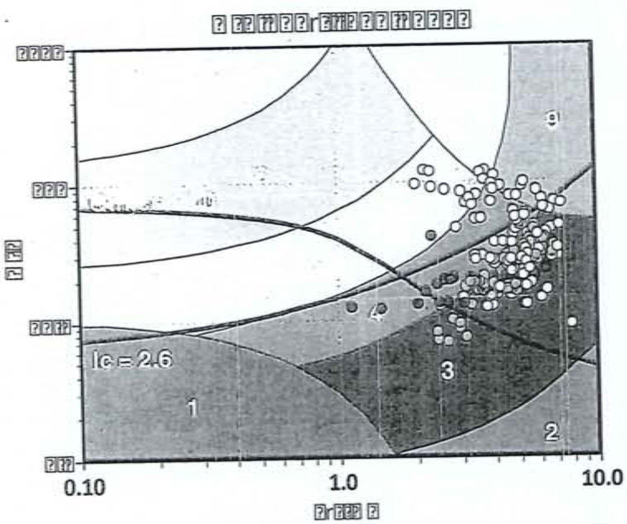
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4205857m E: 540631m
Page No: 1 of 1

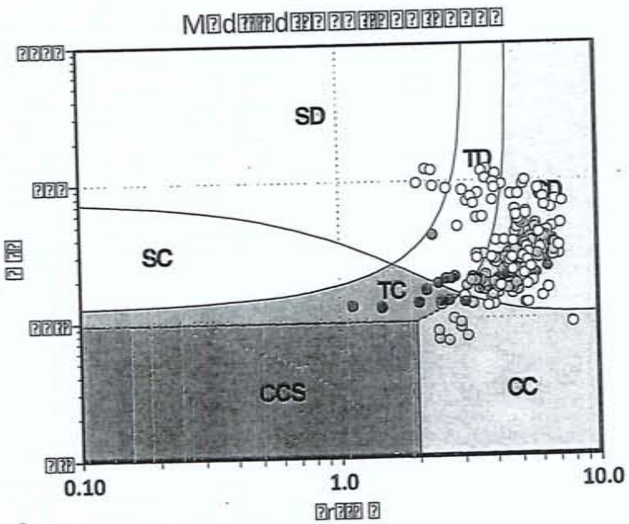
○ Assumed Ueq
◁ Dissipation equilibrium achieved
—— Hydrostatic Line

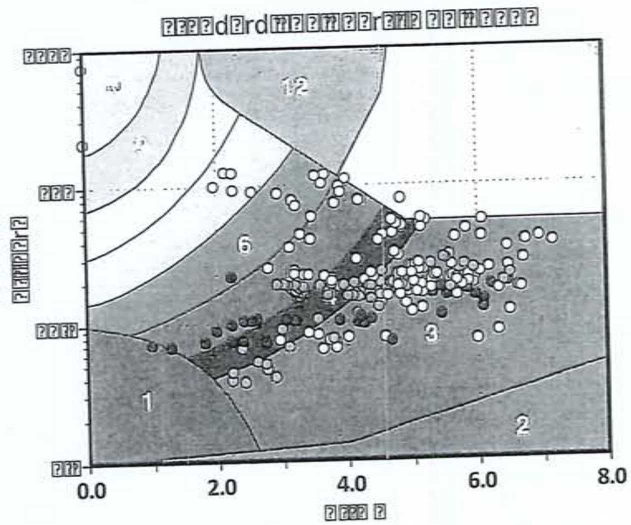
Soil Behavior Type (SBT) Scatter Plots

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 2018-04-11 09:48 Site: 350 Merrydale Road	Sounding: 1-CPT01 Cone: 483:T1500F15U500







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

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

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

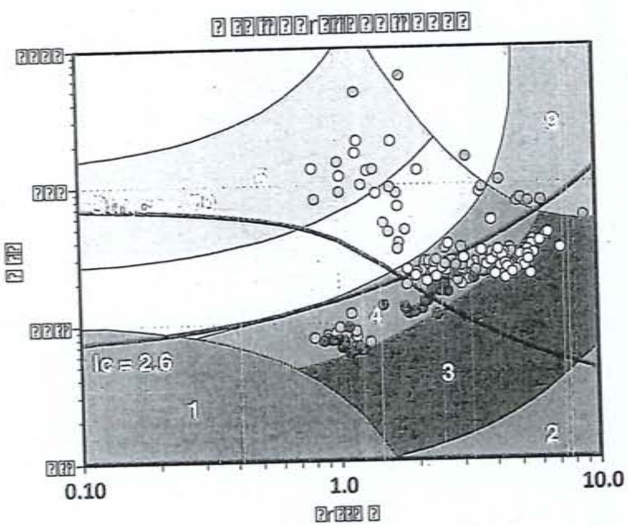
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

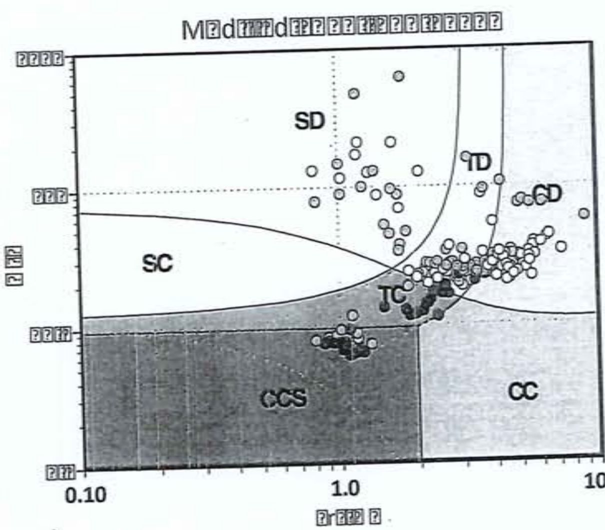
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

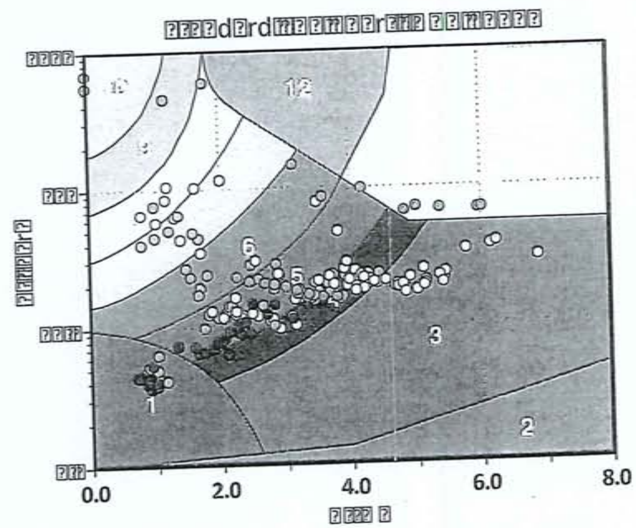
ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 09:05
Site: 350 Merrydale Road

Sounding: 1-CPT02
Cone: 483:T1500F15U500







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

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

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

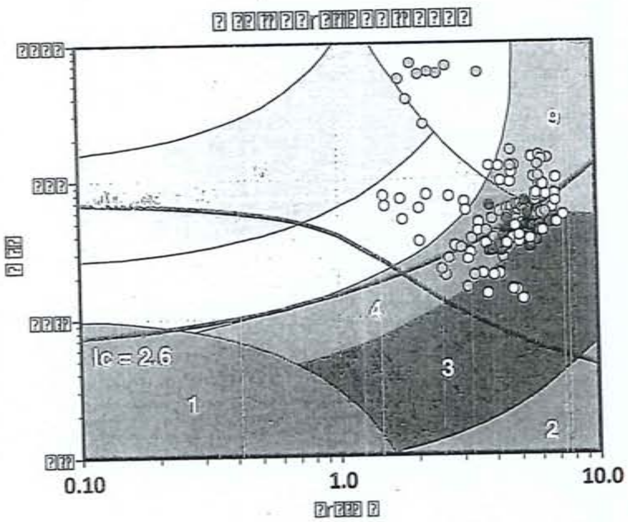
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

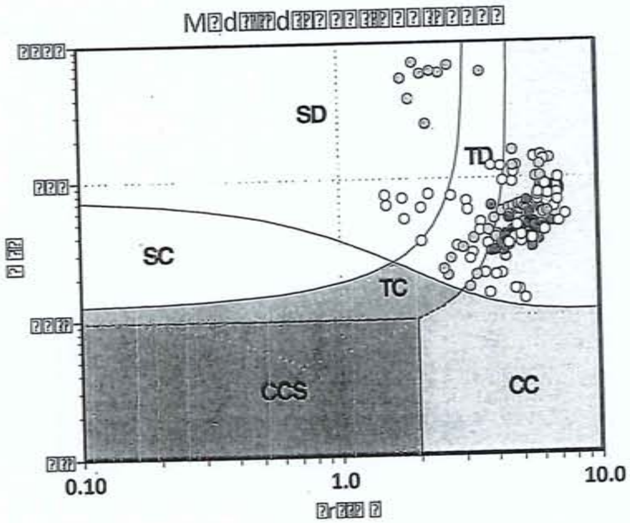
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

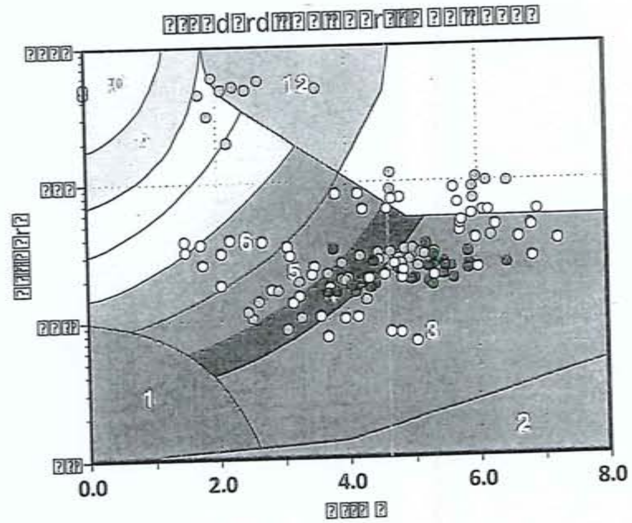
ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 15:50
Site: 350 Merrydale Road

Sounding: 1-CPT03
Cone: 483:T1500F15U500







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

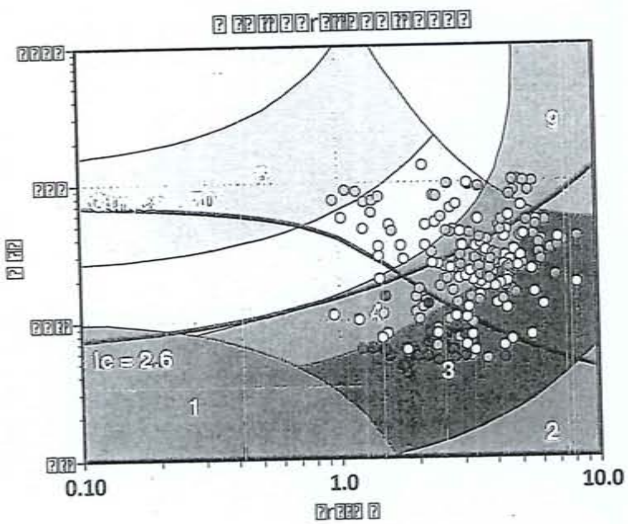
Legend
- Sensitive, Fined Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

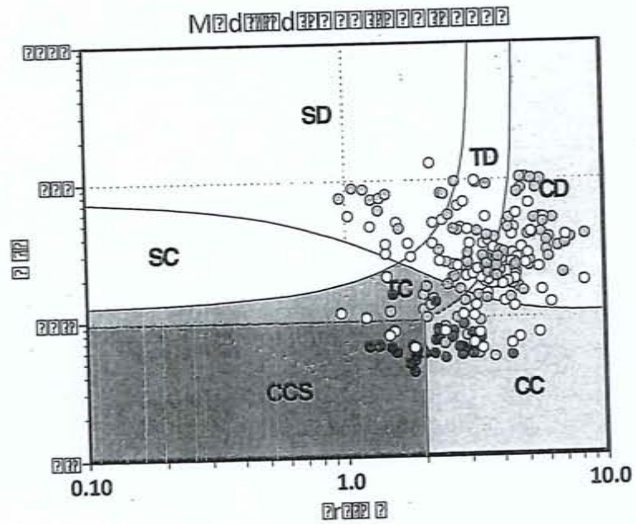
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

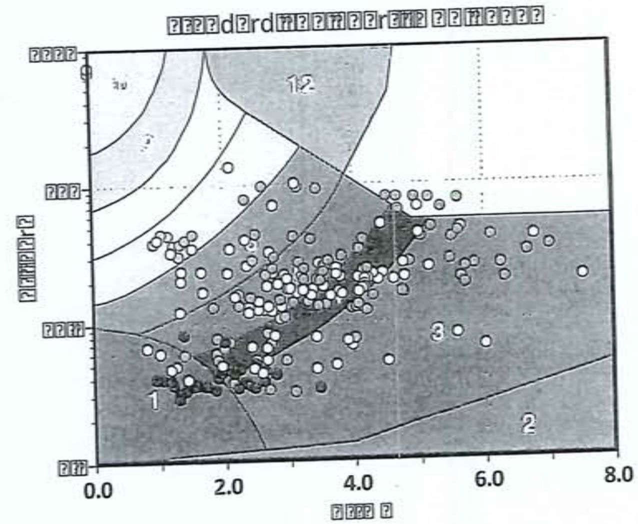
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 2018-04-11 11:35 Site: 350 Merrydale Road	Sounding: 1-CPT04 Cone: 483:T1500F15U500







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

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

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

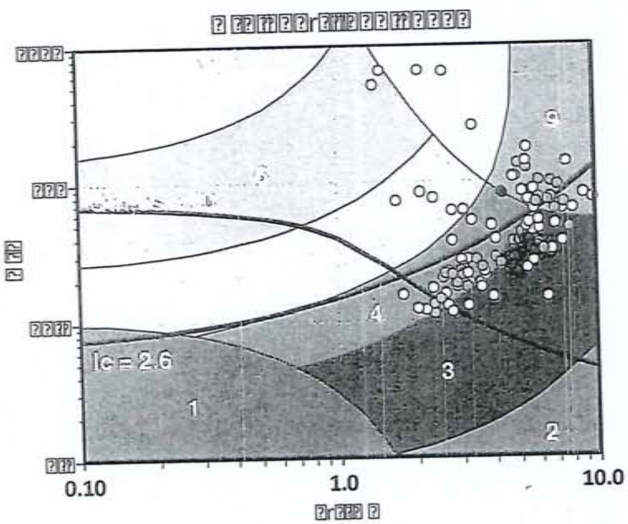
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

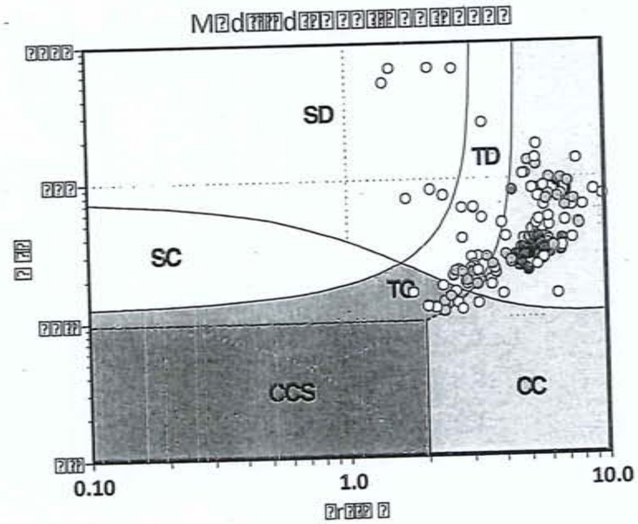
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

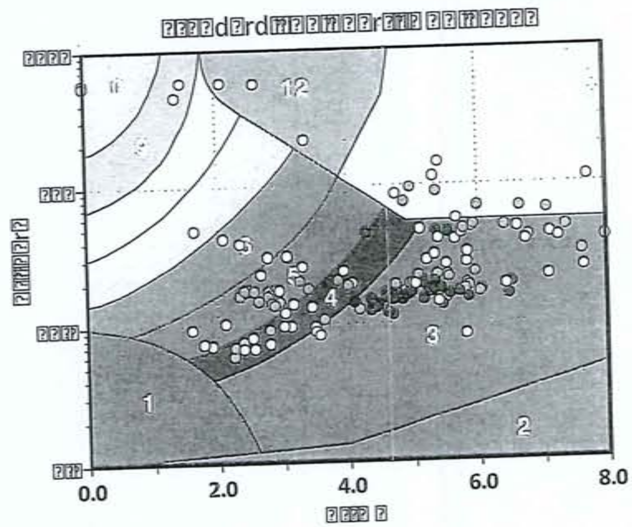
ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 12:46
Site: 350 Merrydale Road

Sounding: 1-CPT05
Cone: 483:T1500F15U500







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

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

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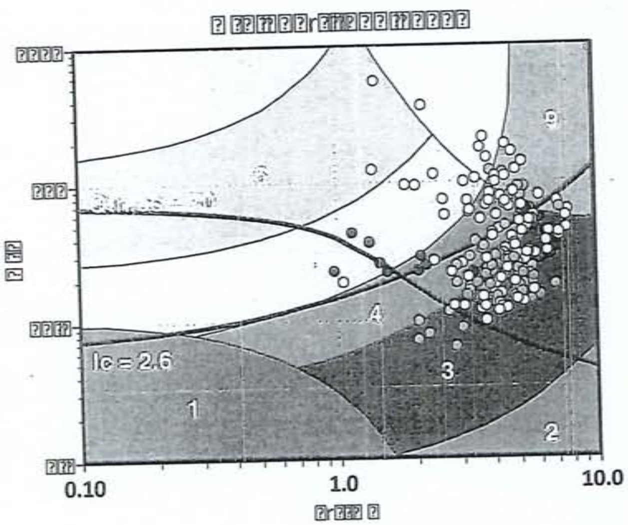
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

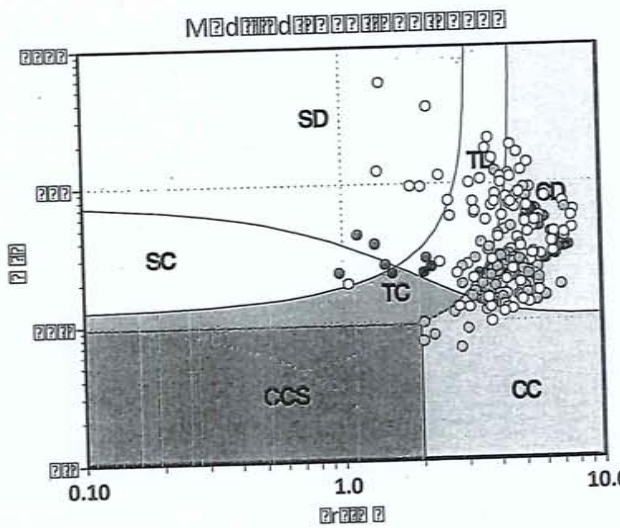
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

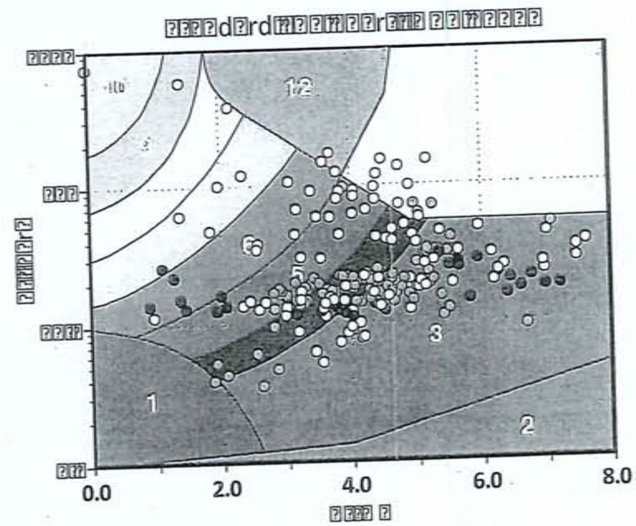
ENGEO Inc.

Job No: 18-56045
Date: 2018-04-11 10:50
Site: 350 Merrydale Road

Sounding: 1-CPT06
Cone: 483:T1500F15U500







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

Legend
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- Silt Mixtures
- Sand Mixtures
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- Very Stiff Fine Grained

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- Clay
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- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Job No: 18-56045
Client: ENGEO Inc.
Project: 350 Merrydale Road
Start Date: 11-Apr-2018
End Date: 11-Apr-2018

CPTu PORE PRESSURE DISSIPATION SUMMARY

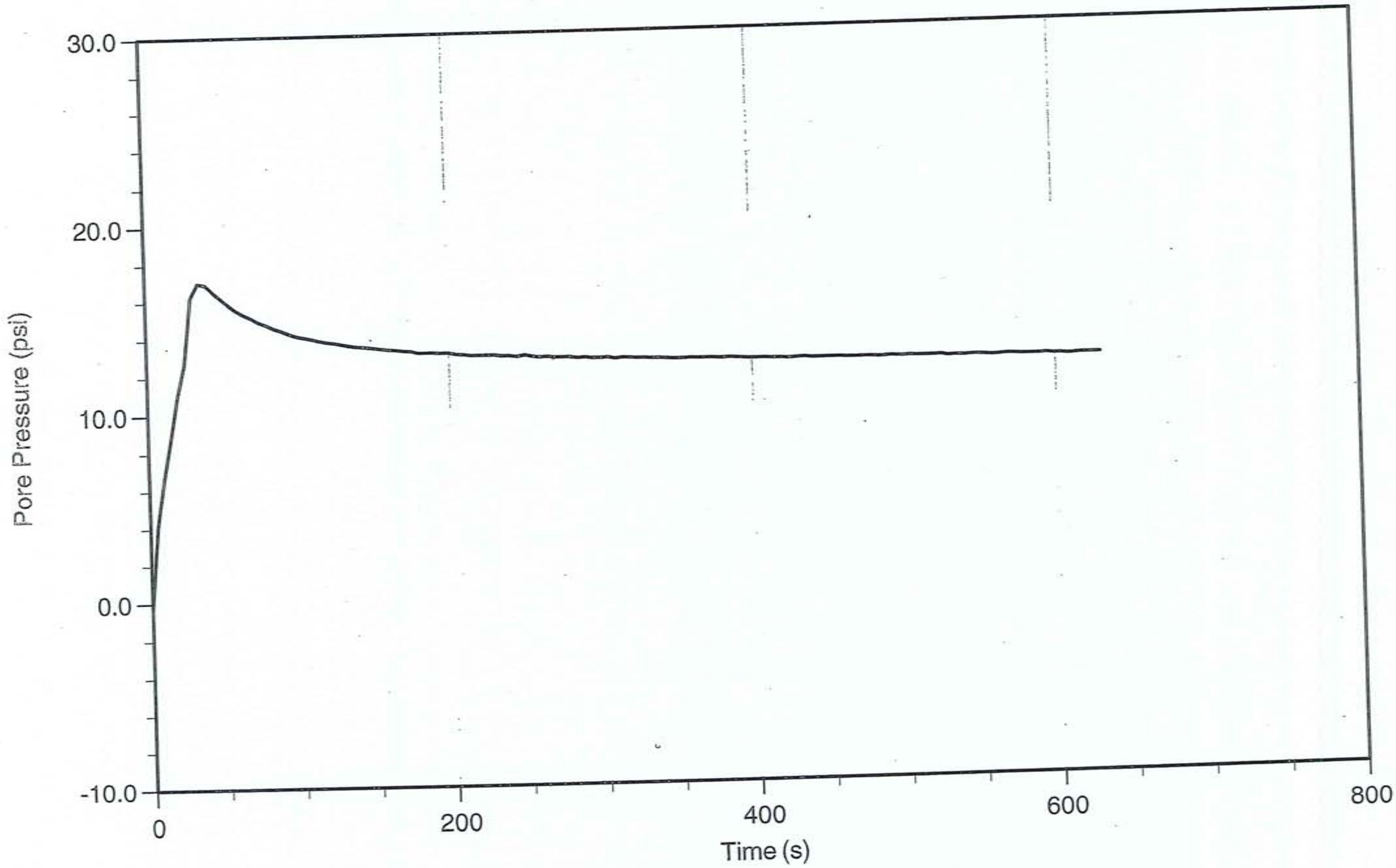
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (psi)	Calculated Phreatic Surface (ft)
1-CPT01	18-56045_CP01	15	630	28.707	Not Achieved	
1-CPT01	18-56045_CP01	15	405	36.253	14.5	2.8
1-CPT02	18-56045_CP02	15	100	30.019	Not Achieved	
1-CPT04	18-56045_CP04	15	400	34.448	13.3	3.7
1-CPT06	18-56045_CP06	15	420	37.729	14.6	4.0

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 04/11/2018 09:48
Site: 350 Merrydale Road

Sounding: 1-CPT01
Cone: 483:T1500F15U500 Area=15 cm²



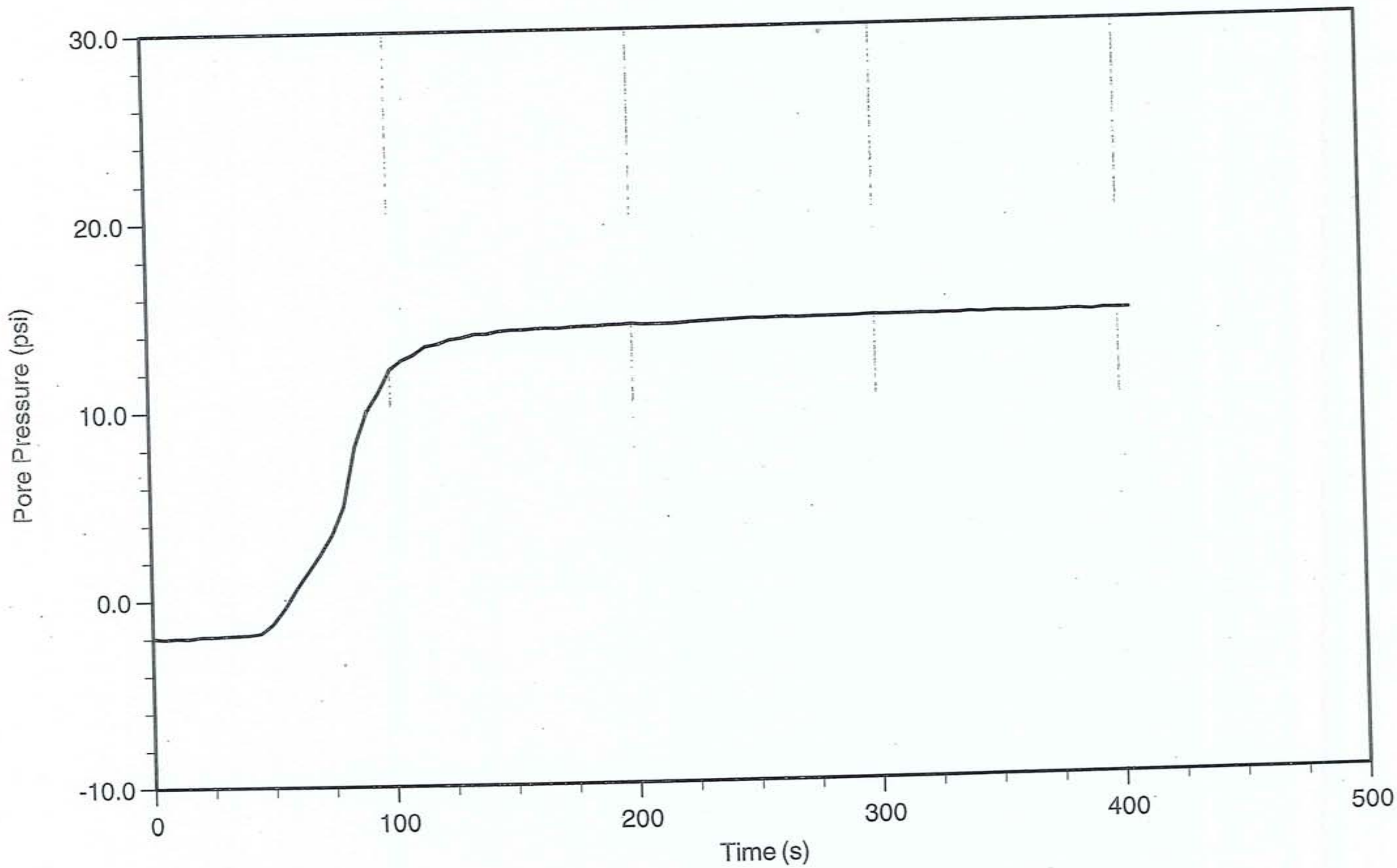
Trace Summary:

Filename: 18-56045_CP01.PPF
Depth: 8.750 m / 28.707 ft
Duration: 630.0 s

UMin: -0.3 psi
UMax: 17.0 psi

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 04/11/2018 09:48 Site: 350 Merrydale Road	Sounding: 1-CPT01 Cone: 483:T1500F15U500 Area=15 cm²

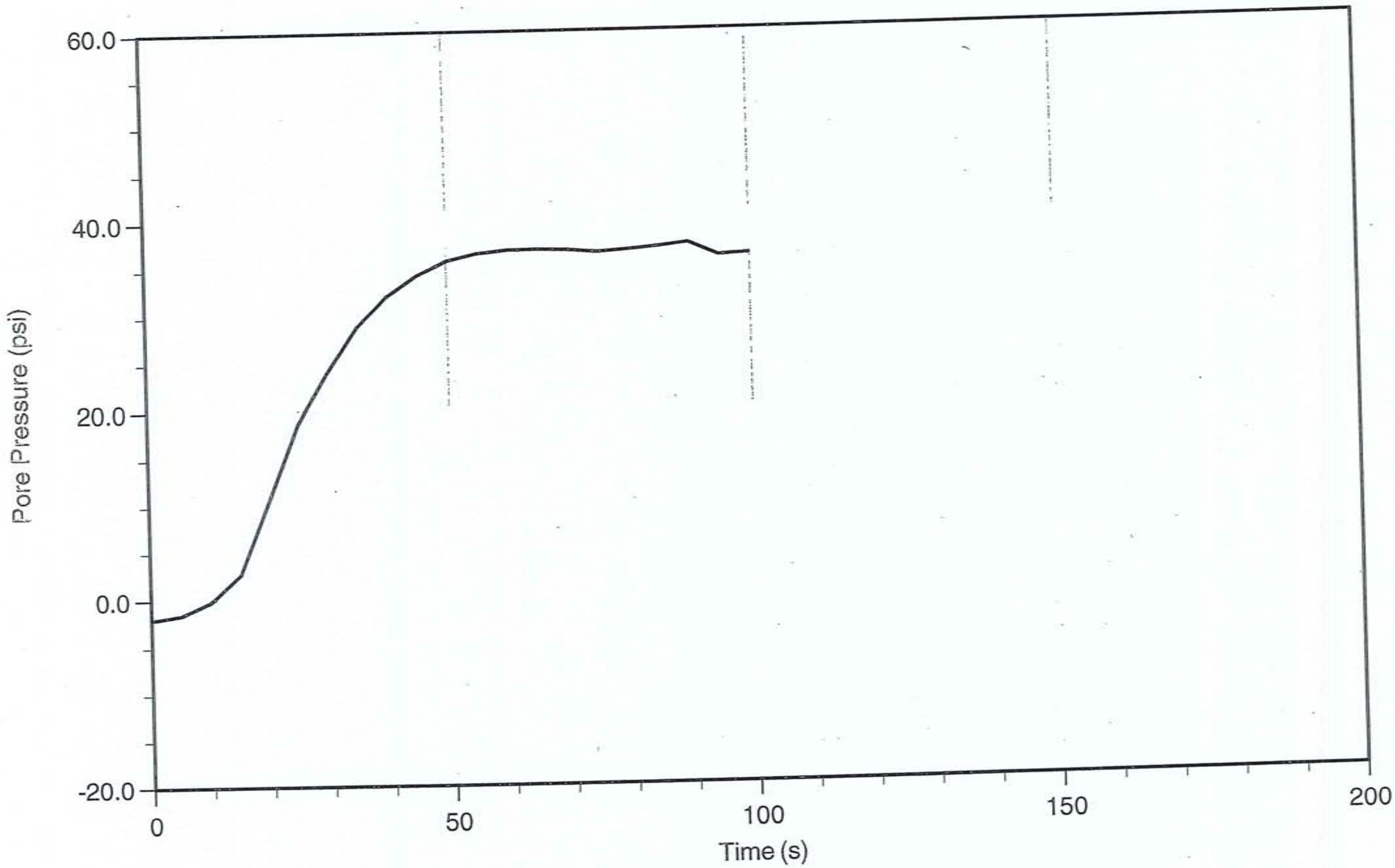


Trace Summary:

Filename: 18-56045_CP01.PPF	UMin: -2.0 psi	WT: 0.842 m / 2.762 ft
Depth: 11.050 m / 36.253 ft	UMax: 14.6 psi	Ueq: 14.5 psi
Duration: 405.0 s		

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.	Job No: 18-56045 Date: 04/11/2018 09:05 Site: 350 Merrydale Road	Sounding: 1-CPT02 Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56045_CP02.PPF
Depth: 9.150 m / 30.019 ft
Duration: 100.0 s

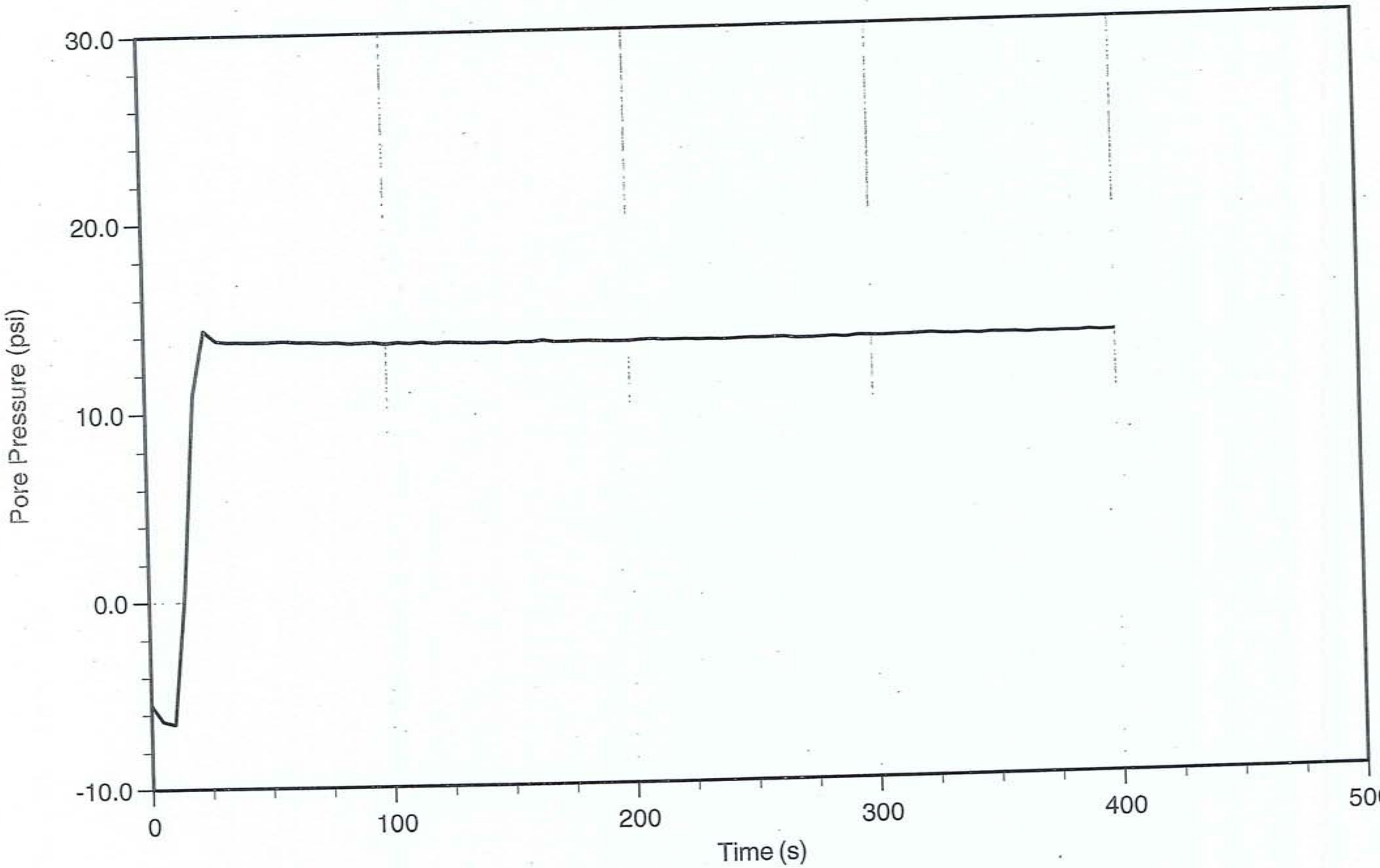
U Min: -2.0 psi
U Max: 37.3 psi

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 04/11/2018 11:35
Site: 350 Merrydale Road

Sounding: 1-CPT04
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56045_CP04.PPF
Depth: 10.500 m / 34.448 ft
Duration: 400.0 s

UMin: -6.5 psi
UMax: 14.3 psi

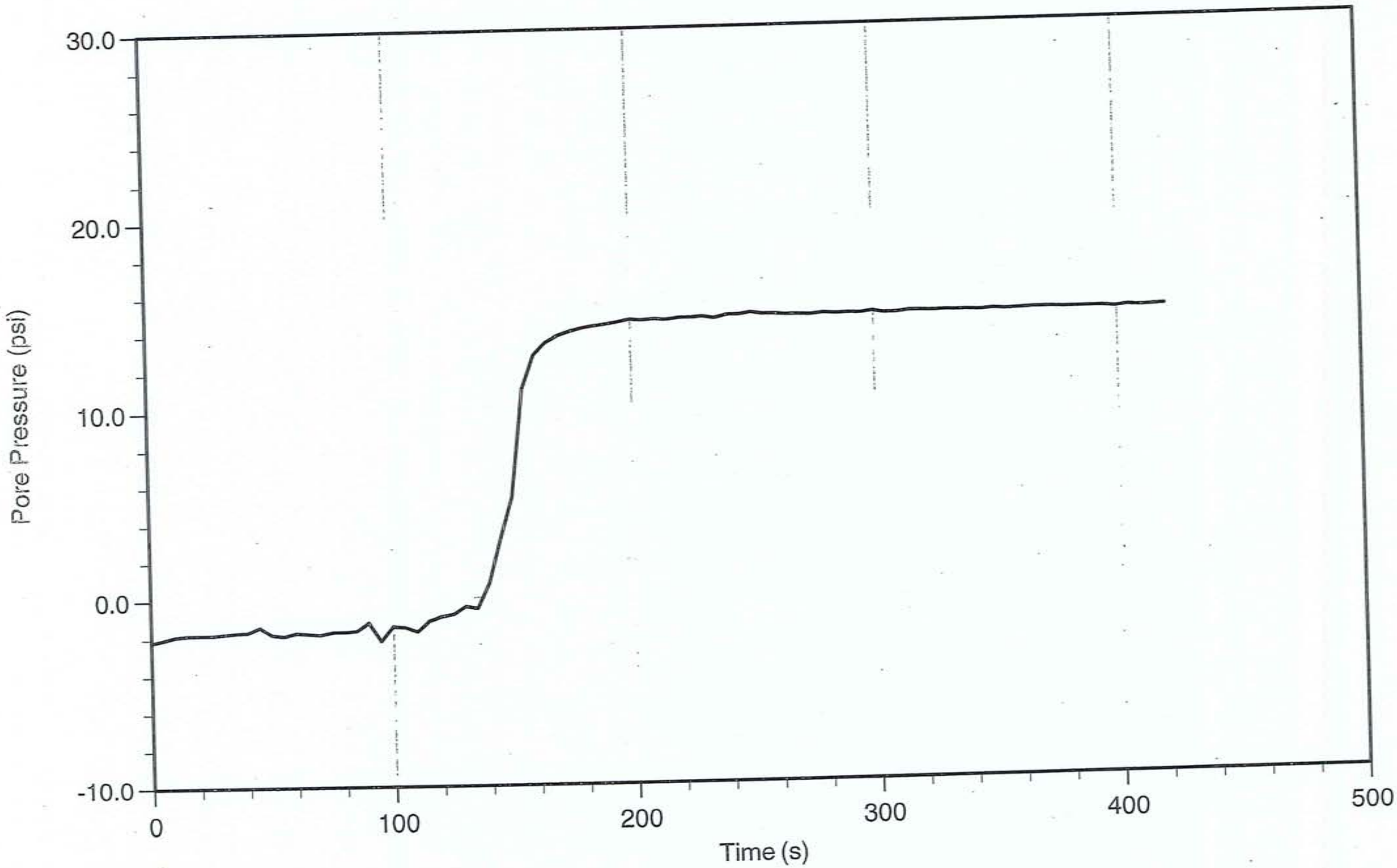
WT: 1.116 m / 3.661 ft
Ueq: 13.3 psi

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

ENGEO Inc.

Job No: 18-56045
Date: 04/11/2018 10:50
Site: 350 Merrydale Road

Sounding: 1-CPT06
Cone: 483:T1500F15U500 Area=15 cm²

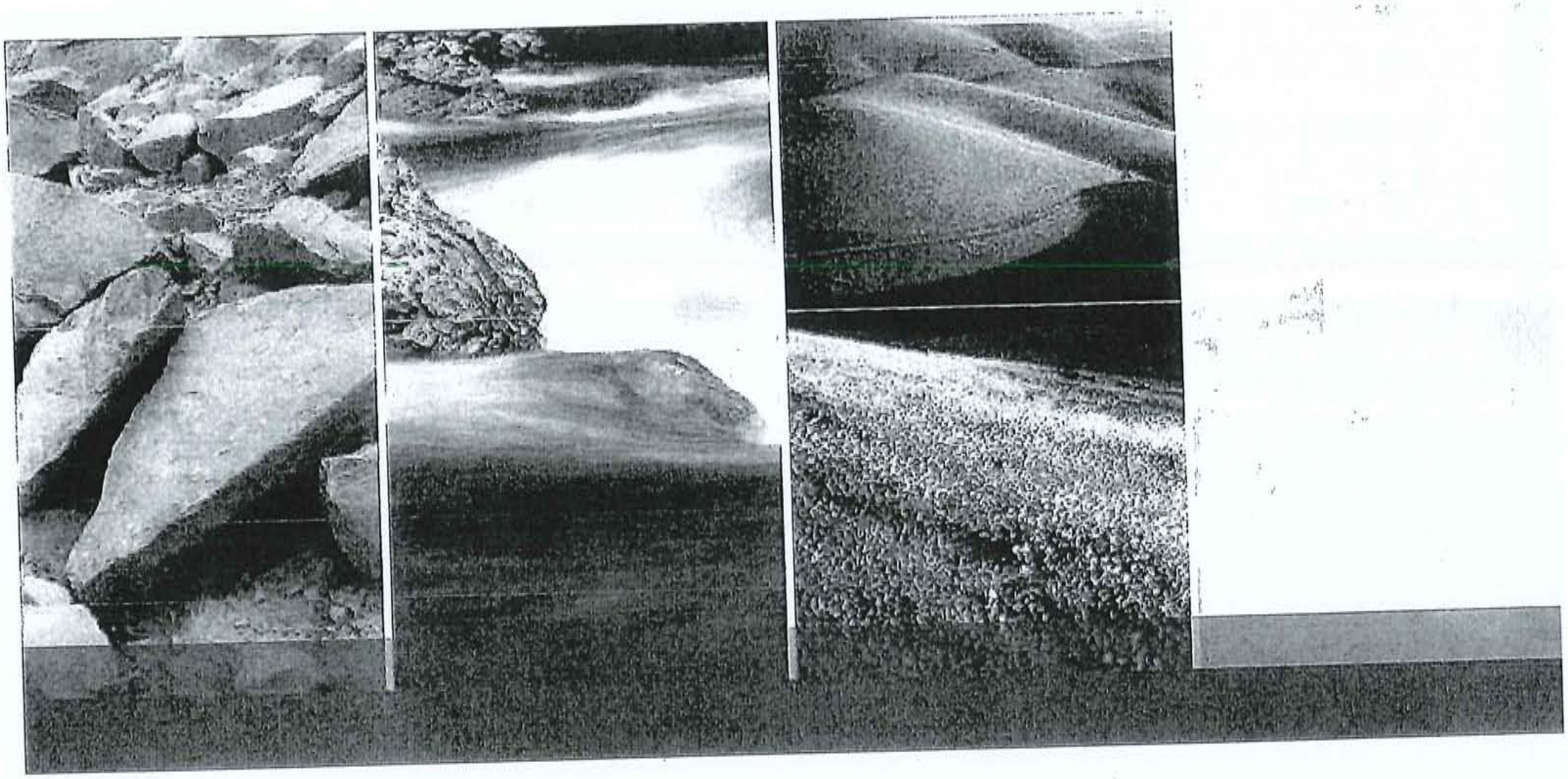


Trace Summary:

Filename: 18-56045_CP06.PPF
Depth: 11.500 m / 37.729 ft
Duration: 420.0 s

U Min: -2.2 psi
U Max: 14.8 psi

WT: 1.233 m / 4.045 ft
Ueq: 14.6 psi

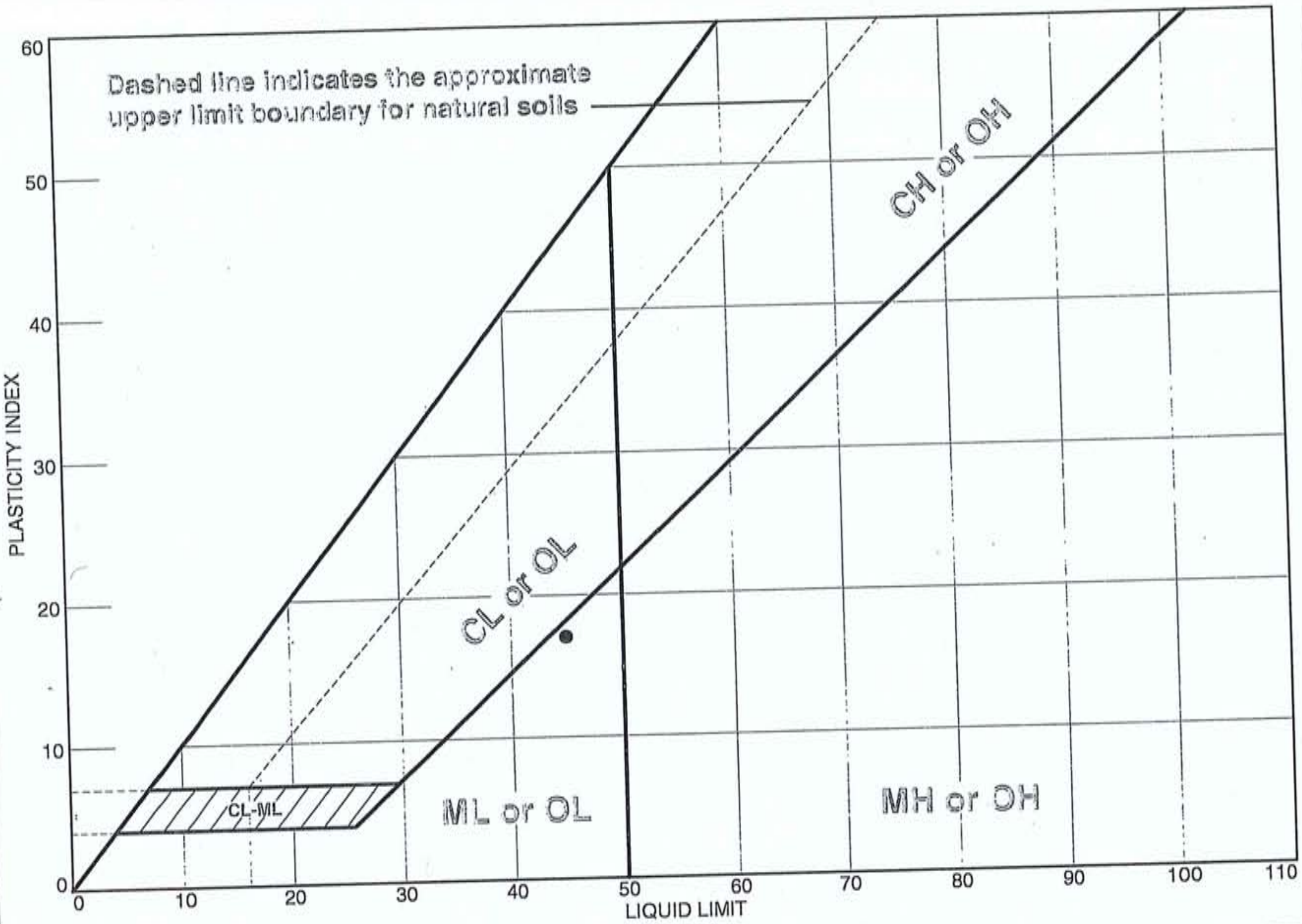


APPENDIX B

LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Particle Size Distribution Report

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	45	28	17		44.7	

Project No. 14862.000.000 **Client:** Campus Property Group

Project: 3833 Redwood Highway San Rafael, CA

●**Sample Number:** 1-CPT6 @ 2.5-3

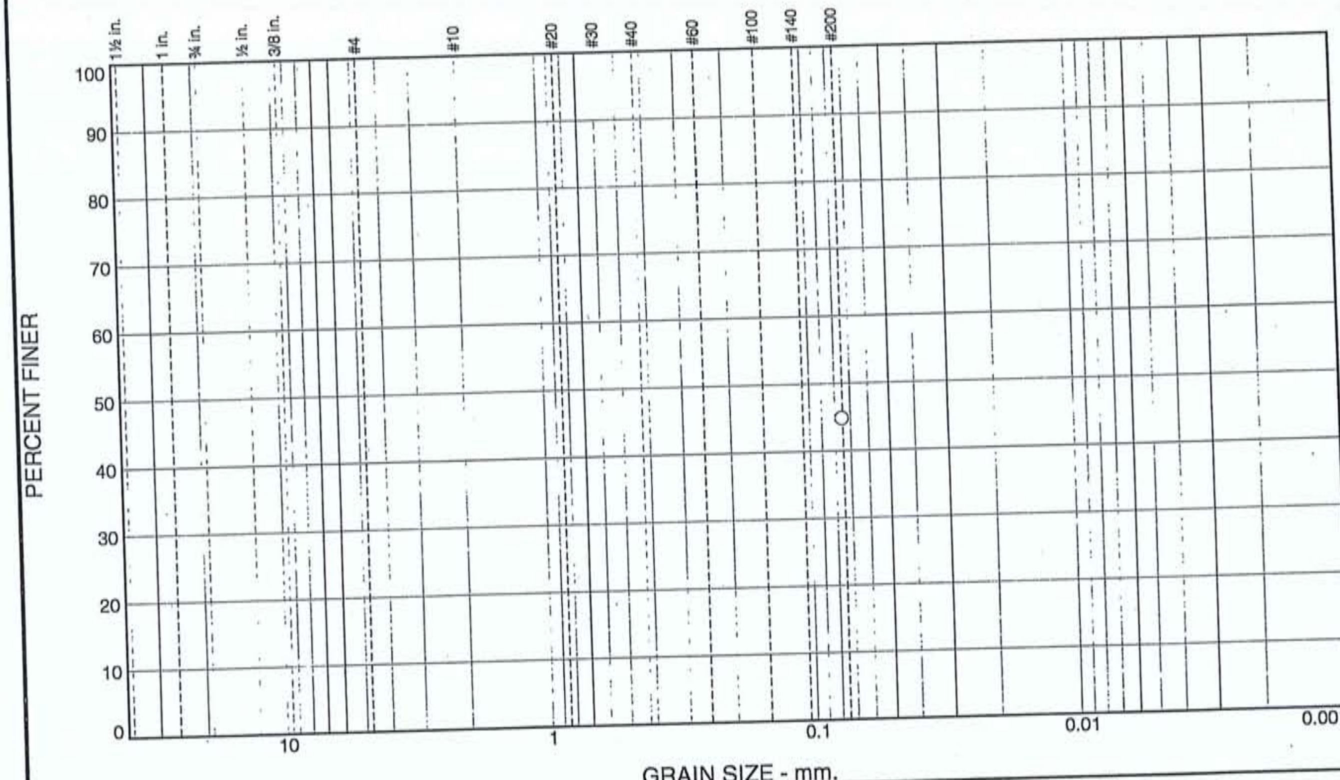
Remarks:

●PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
Dry Sample Weight = 227.52; Soak Time = 4 hrs 10 min

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						44.7	

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

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 28 LL= 45 PI= 17

Coefficients

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

Classification

USCS= AASHTO=

Remarks

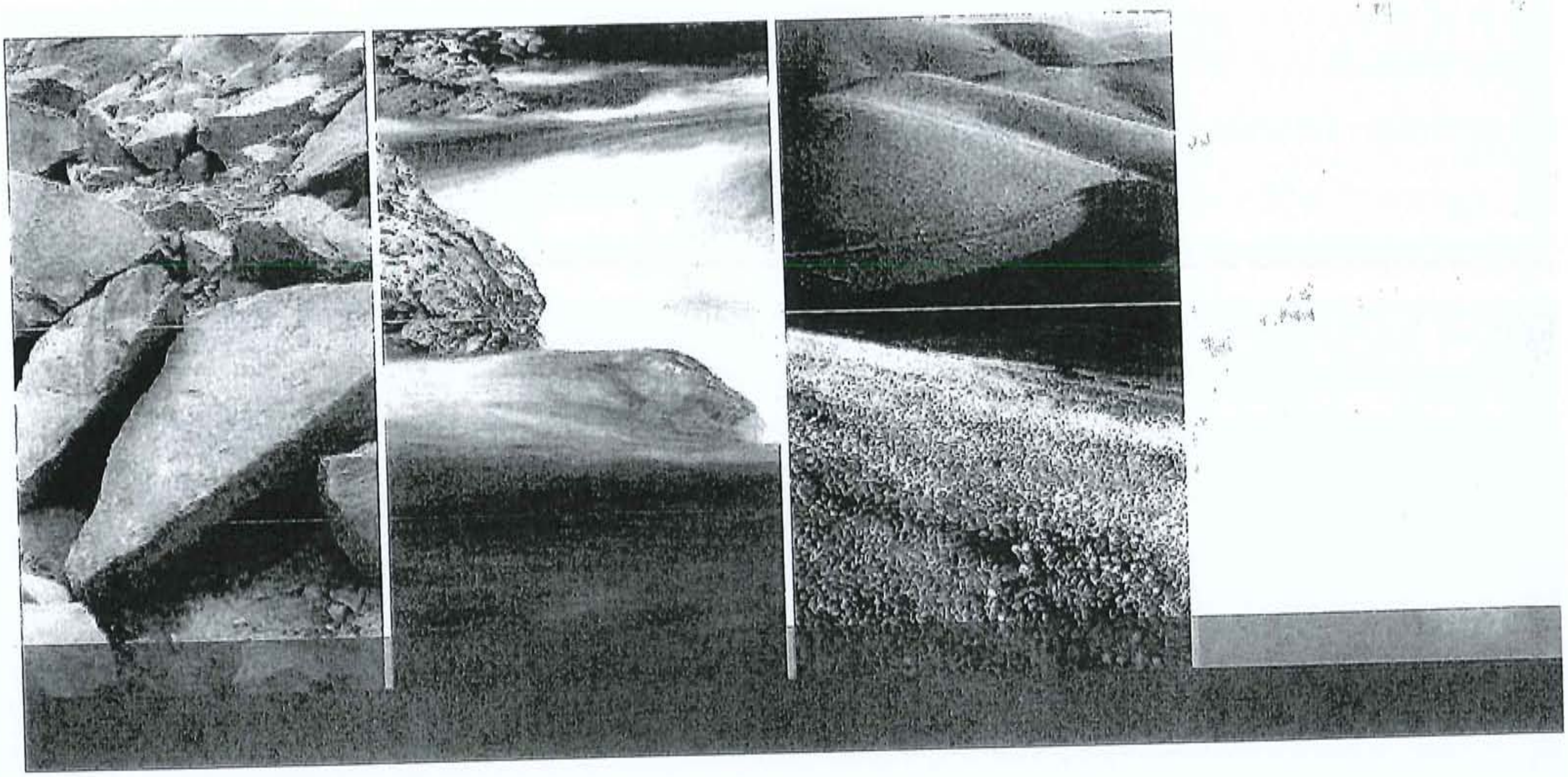
PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
Dry Sample Weight = 227.52; Soak Time = 4 hrs 10 min

Sample Number: 1-CPT6 @ 2.5-3 **Date:** 4/23/2018

GEO INCORPORATED

Client: Campus Property Group
Project: 3833 Redwood Highway San Rafael, CA
Project No: 14862.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield



APPENDIX C

Corrosivity Analysis (Cerco)

California State Certified Laboratory No. 2153

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

25 April, 2018

Job No. 1804120
Cust. No. 10169

Mr. Nick Serra
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No.: 14862.000.000
Project Name: 1-CPT2
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Serra:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on April 17, 2018. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 22 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil 6.63, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 310-mV, which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

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

JDH/jdl
Enclosure

alifornia State Certified Laboratory No. 2153

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

lient: ENGEO Incorporated
lient's Project No.: 14862.000.000
lient's Project Name: 1-CPT2
ate Sampled: 11-Apr-18
ate Received: 17-Apr-18
latrix: Soil
uthorization: Signed Chain of Custody

Date of Report: 25-Apr-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1804120-001	1-CPT2 @ 1'-1.5'	310	6.63	-	2,300	-	N.D.	22
Method:		ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:		-	-	10	-	50	15	15
		23-Apr-2018	23-Apr-2018	-	24-Apr-2018	-	23-Apr-2018	23-Apr-2018

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

Chain of Custody

Page 1 of 1

CERCO analytical
1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775

Job No.	CU#	Client Project I.D.	Schedule / Analyte	Date Sampled	Date Due
1804120	1069				

Full Name: Nick Serra
Phone 925-395 x2570
Fax
Cell ☒

Company and/or Mailing Address: ENGEO Incorporated

Sample Source: 1-CPT2 Project No. 14862.000.000

ANALYSIS / ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation
	1-CPT2@1-1.5'	4/11/18	3pm	S		1qt			X	X	X	X	X			X

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

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

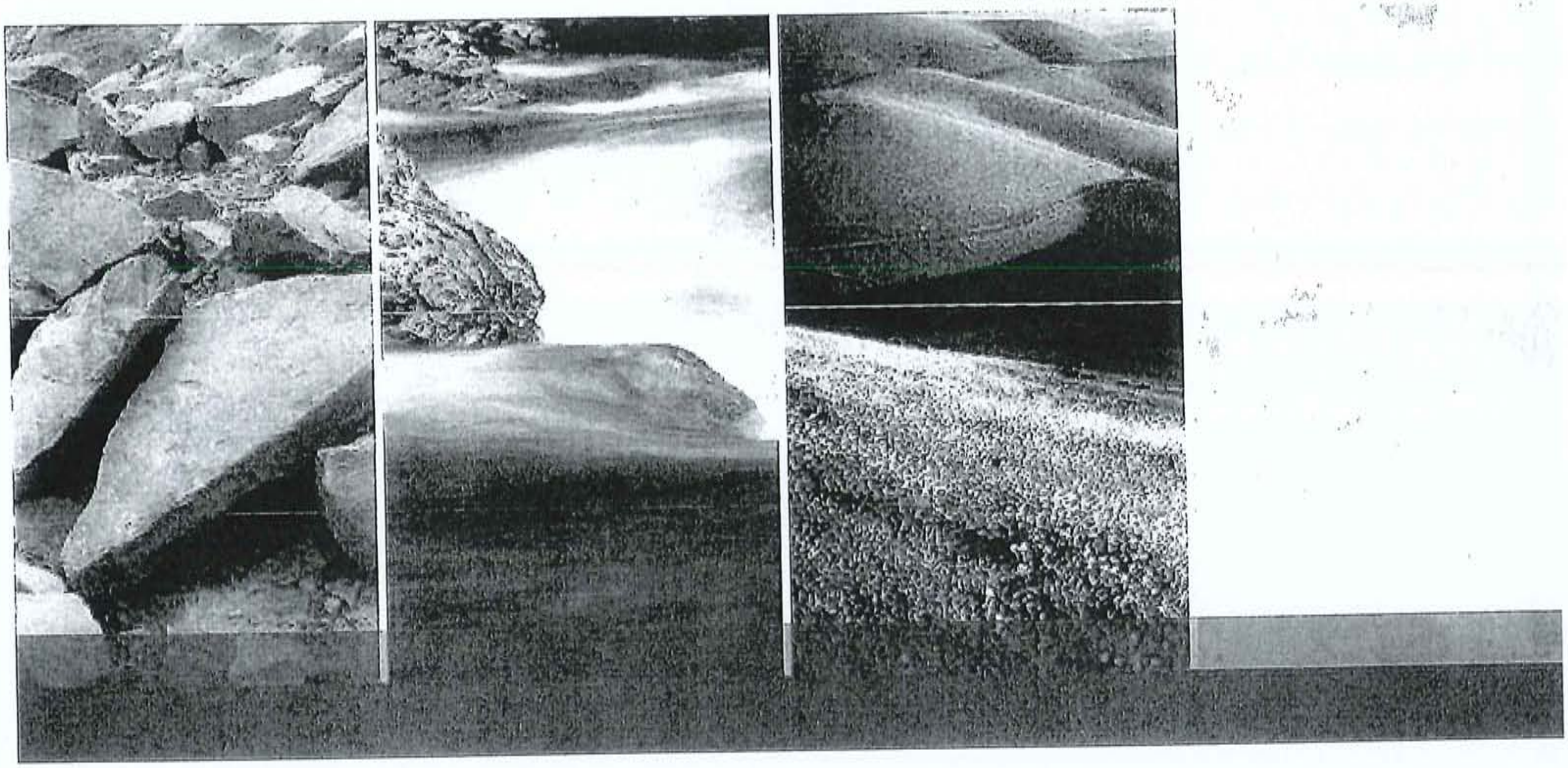
SAMPLE RECEIPT: Total No. of Containers; Rec'd Good Cond/Cold; Conforms to Record; Temp. at Lab -°C; Sampler

	Date	Time
Relinquished By: [signature]	4/16/18	9am
Received By: [signature]	4/16/18	12:17am.
Relinquished By: [signature]	4/17/18	1:33pm
Received By: [signature]	4/17/18	1335
Relinquished By: [signature]	4/17/18	1520
Received By: [signature]	4/17/18	1526

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

Email Address: nserra@engeo.com



APPENDIX D

LIQUEFACTION ANALYSIS

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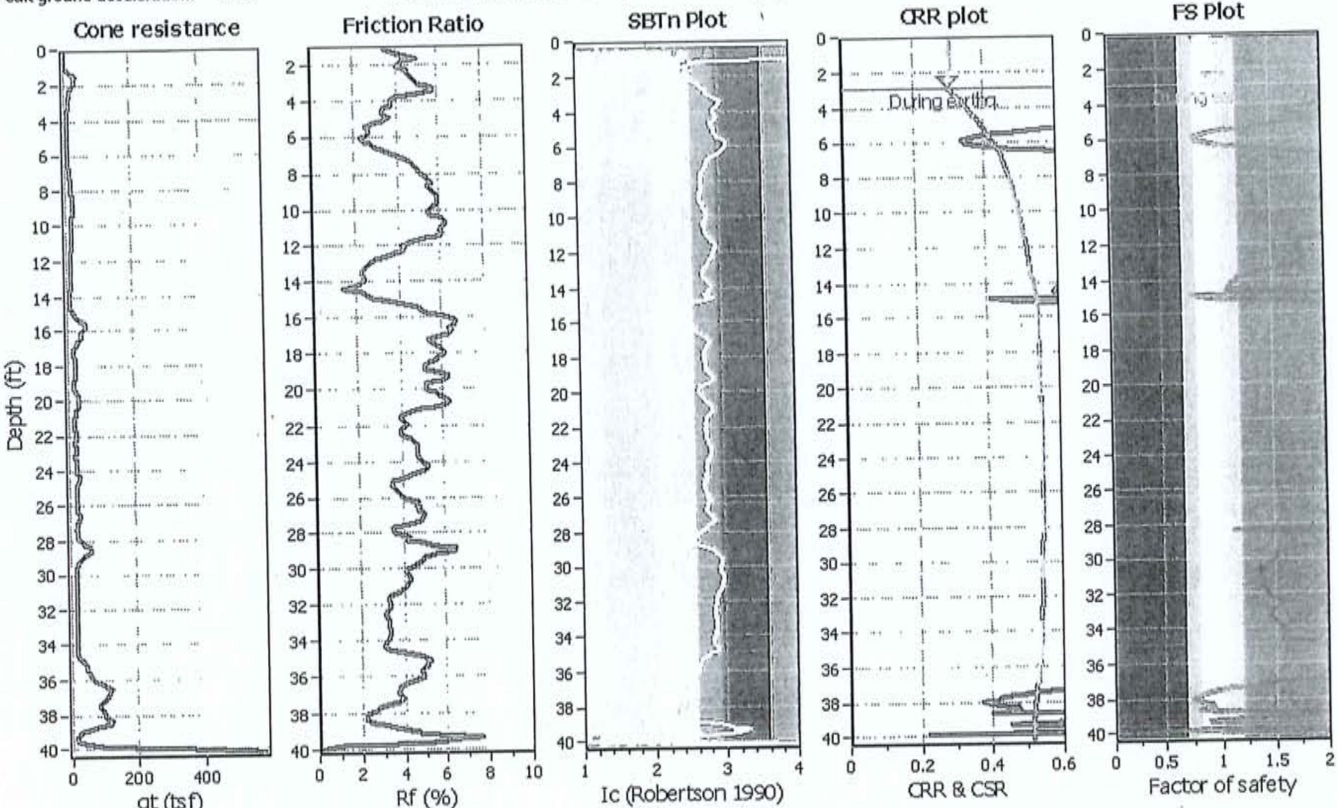
LIQUEFACTION ANALYSIS REPORT

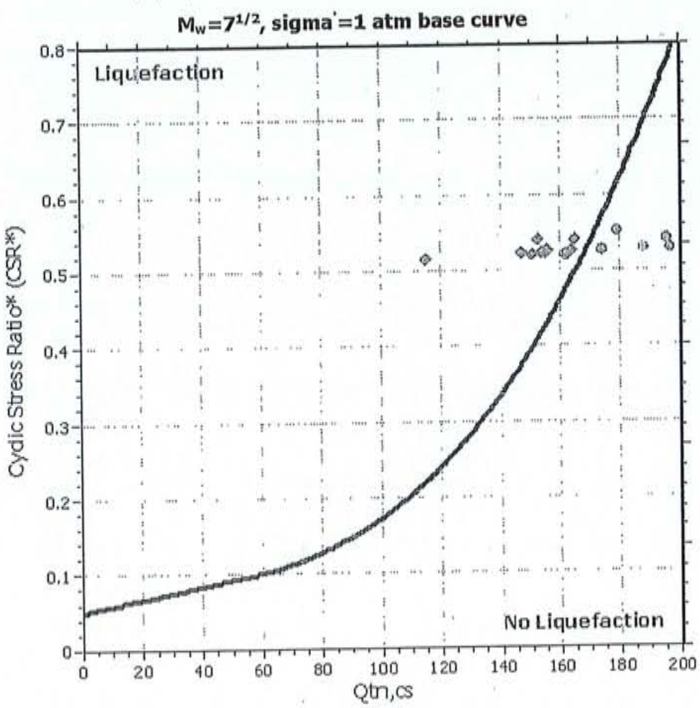
Project title : 3833 Redwood Highway **Location : San Rafael, CA**

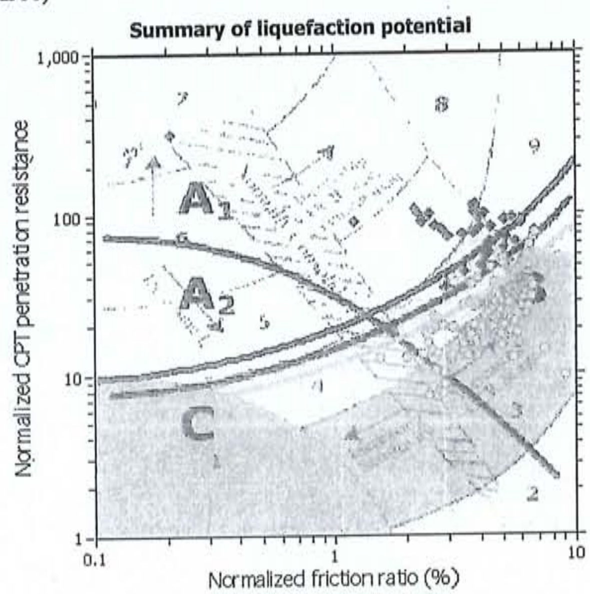
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

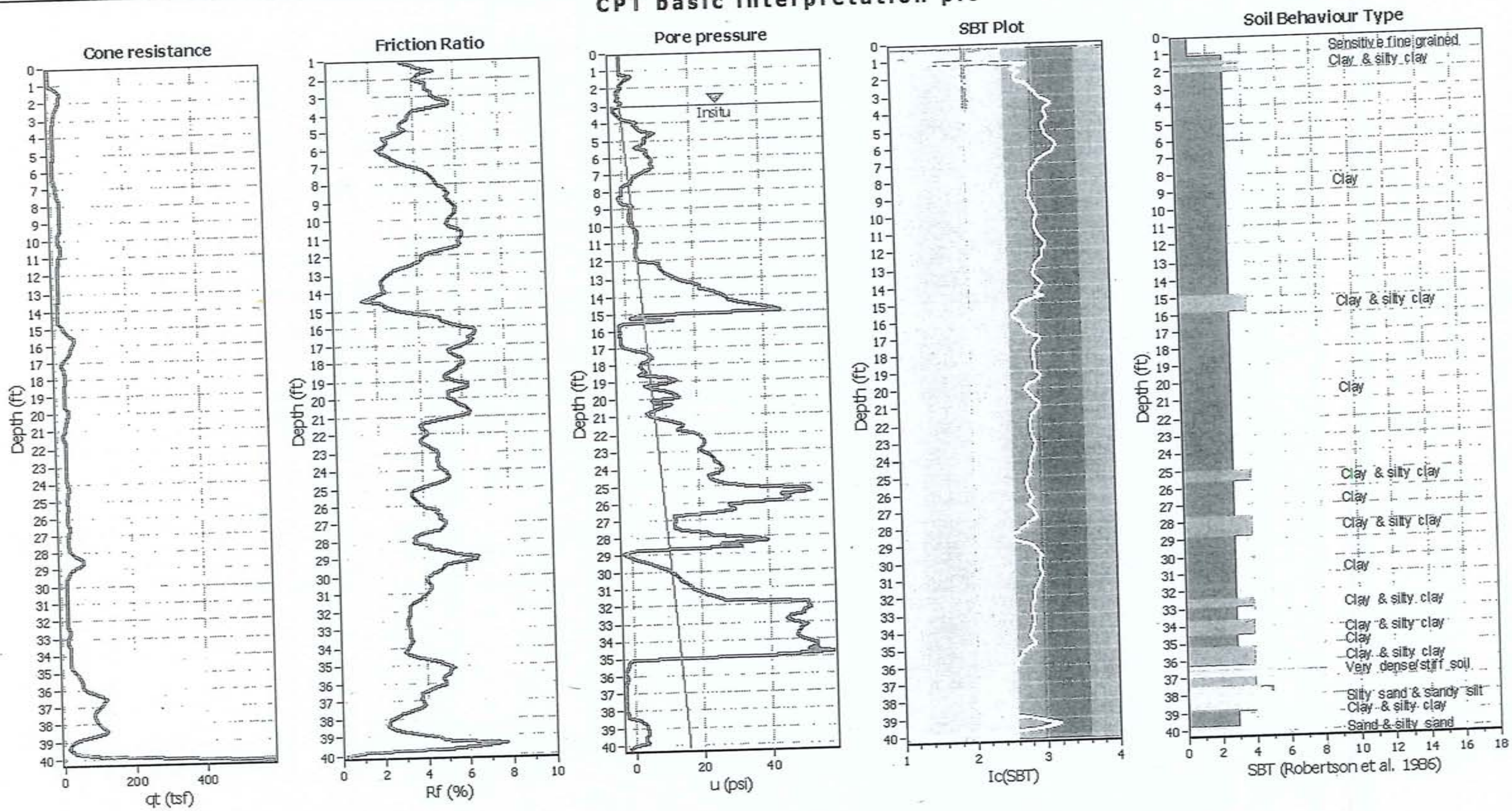






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



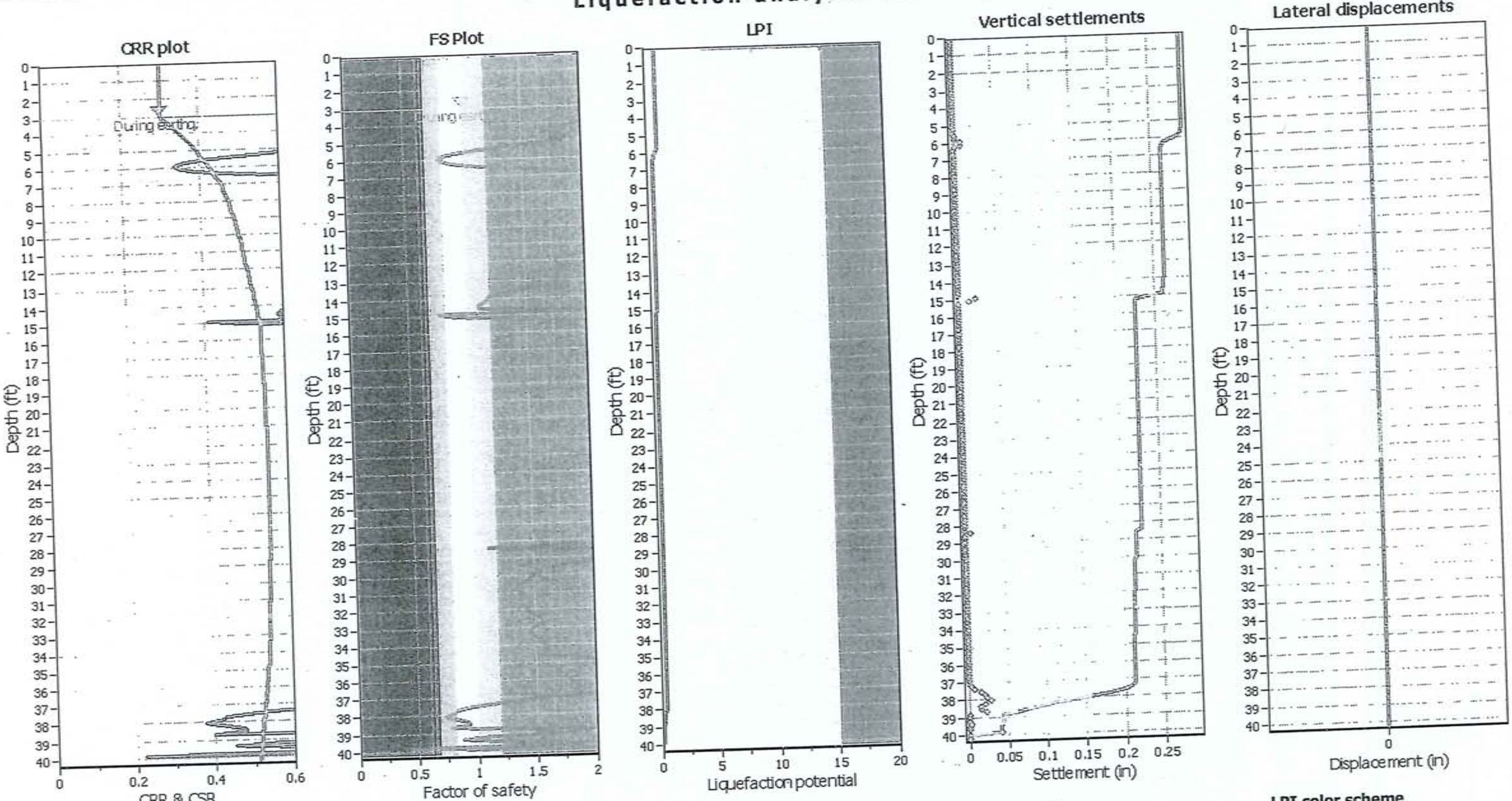
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

Liquefaction analysis overall plot



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

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

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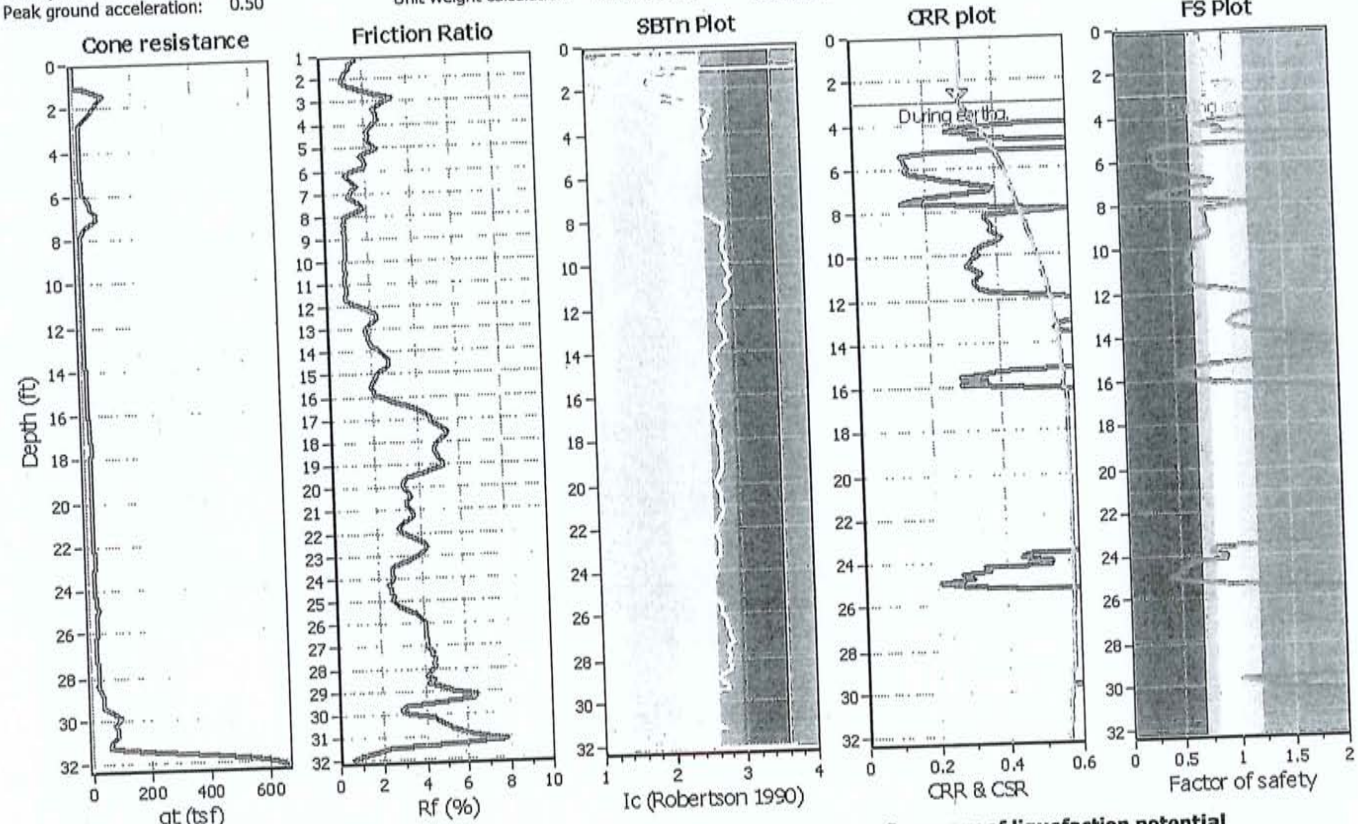
LIQUEFACTION ANALYSIS REPORT

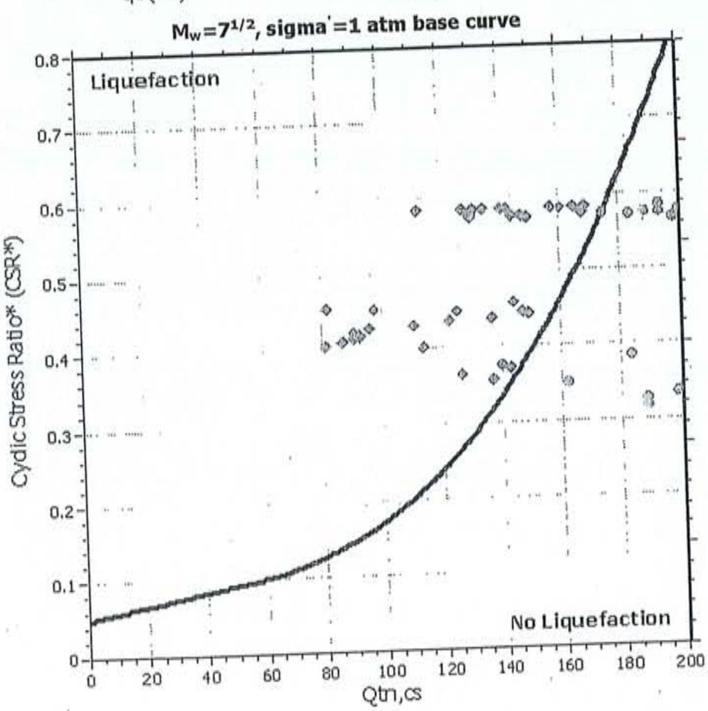
Project title : 3833 Redwood Highway **Location : San Rafael, CA**

CPT file : 1-CPT2

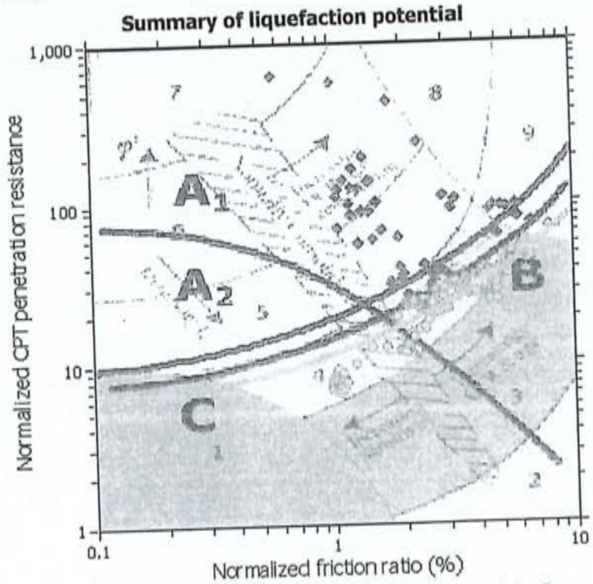
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		



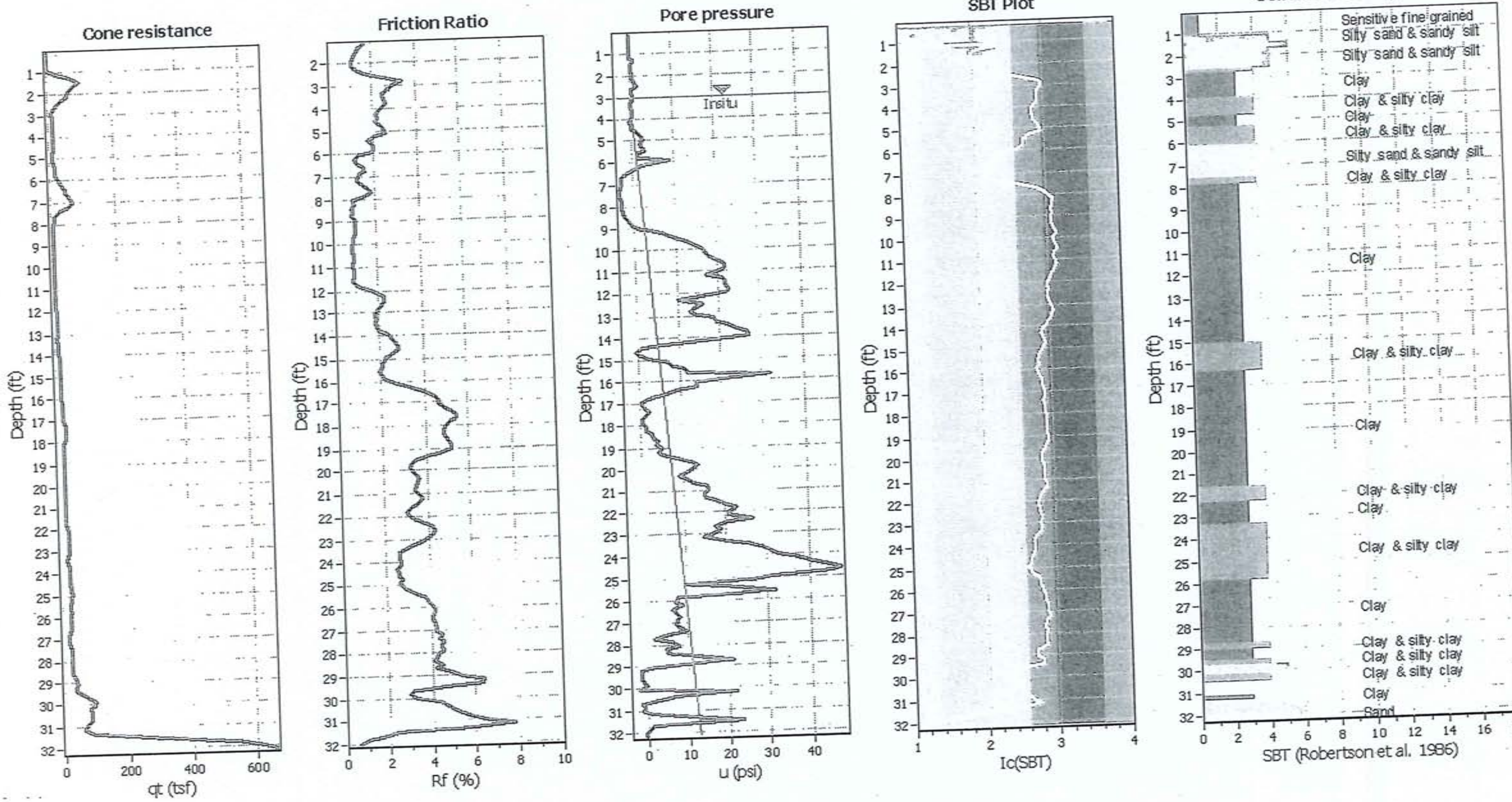


Summary of liquefaction potential



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

CPT basic interpretation plo



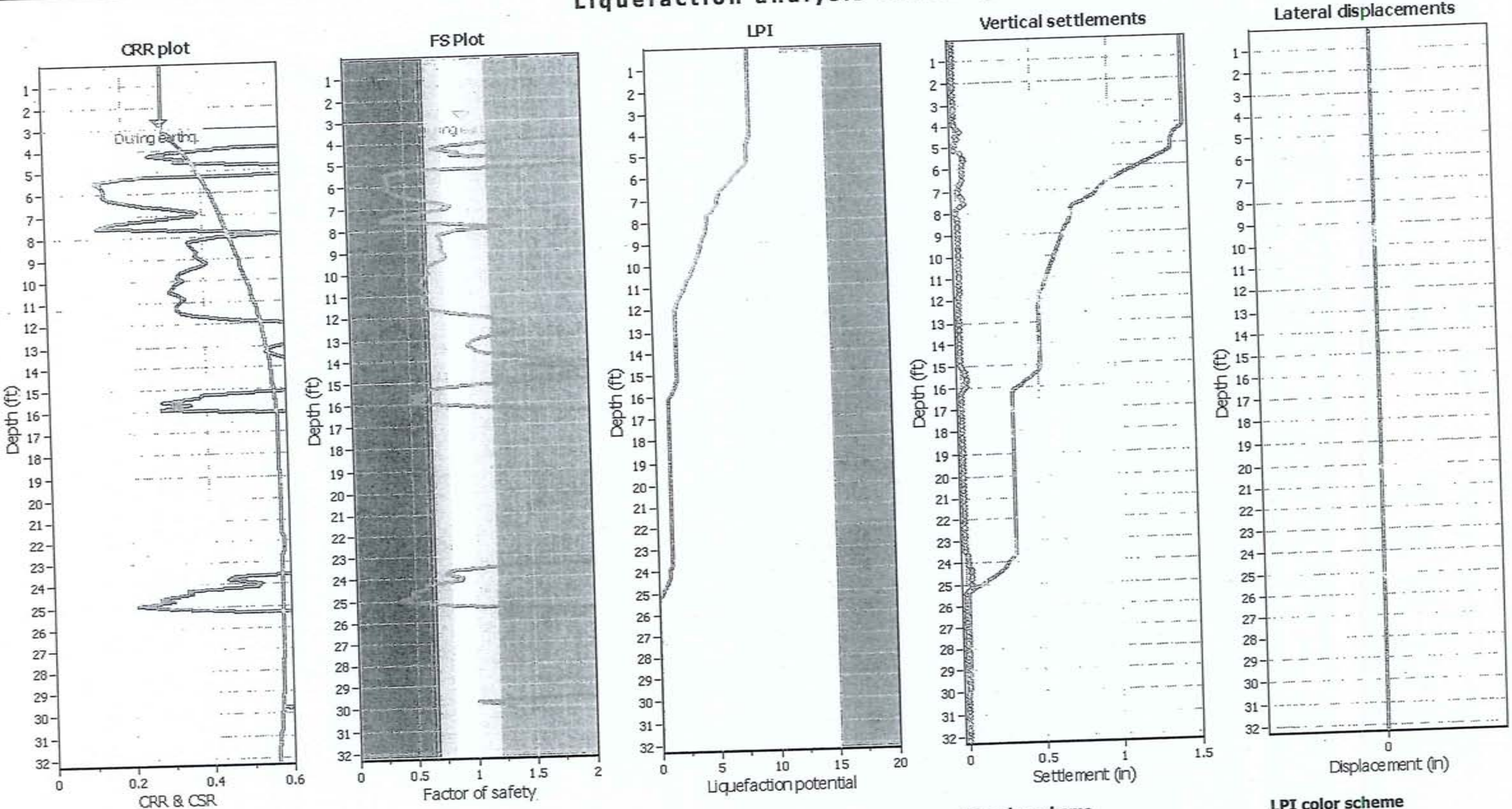
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

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

LPI color scheme
- Very high risk
- High risk
- Low risk

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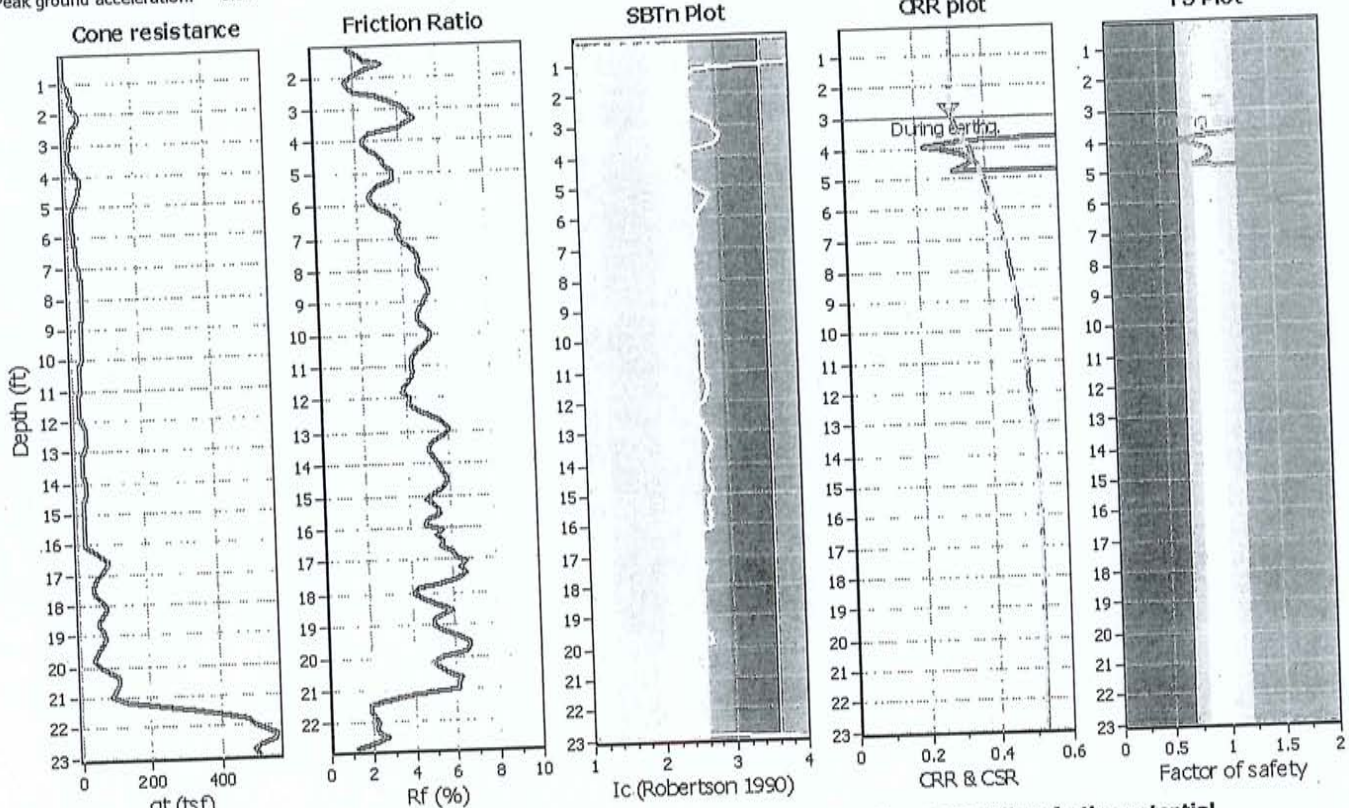
LIQUEFACTION ANALYSIS REPORT

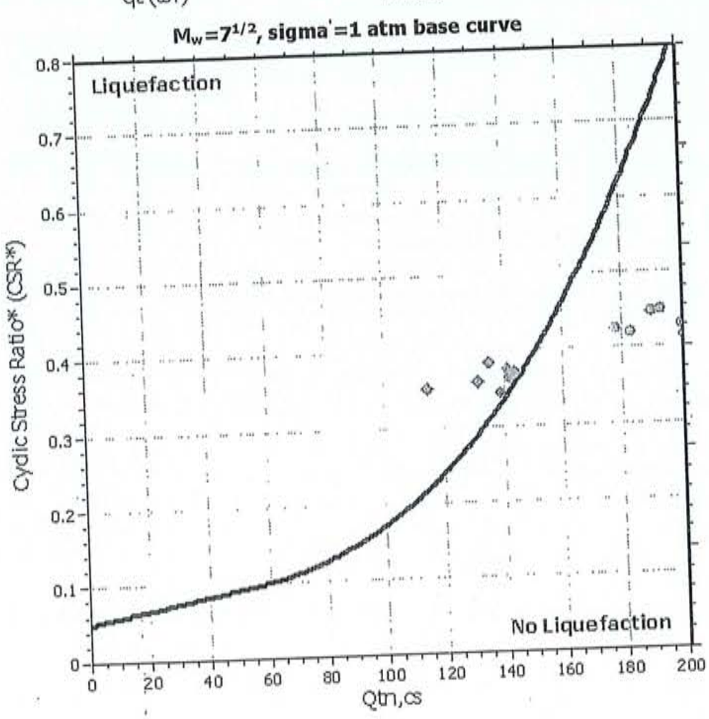
Project title : 3833 Redwood Highway **Location : San Rafael, CA**

CPT file : 1-CPT3

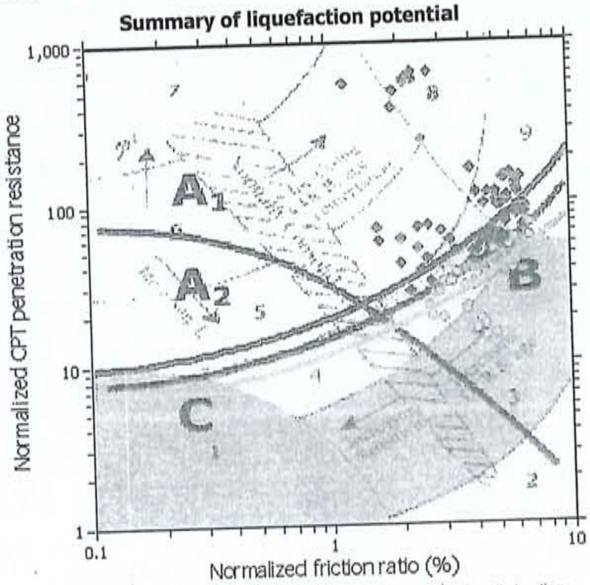
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		



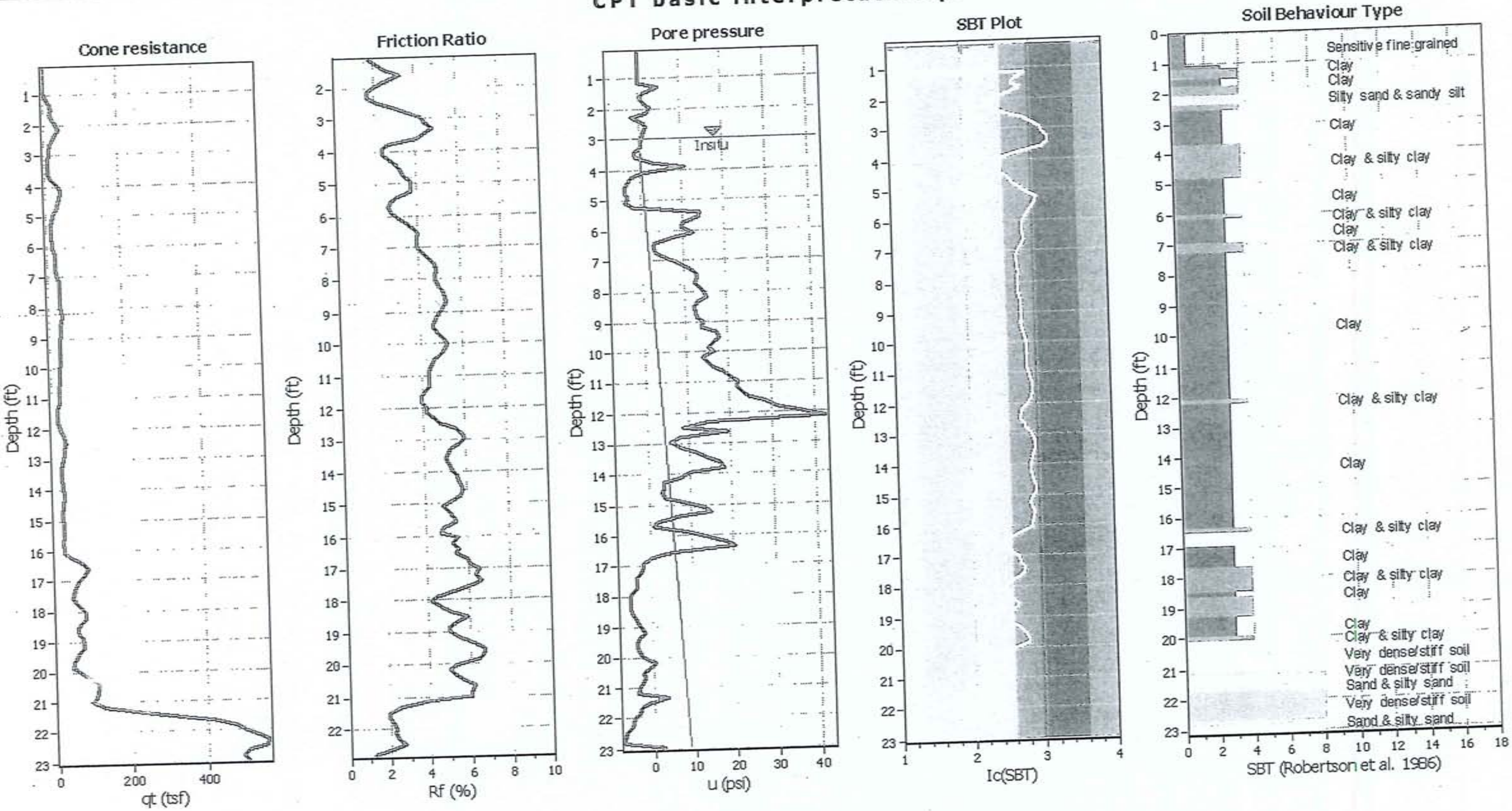


Summary of liquefaction potential



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

CPT basic interpretation plo



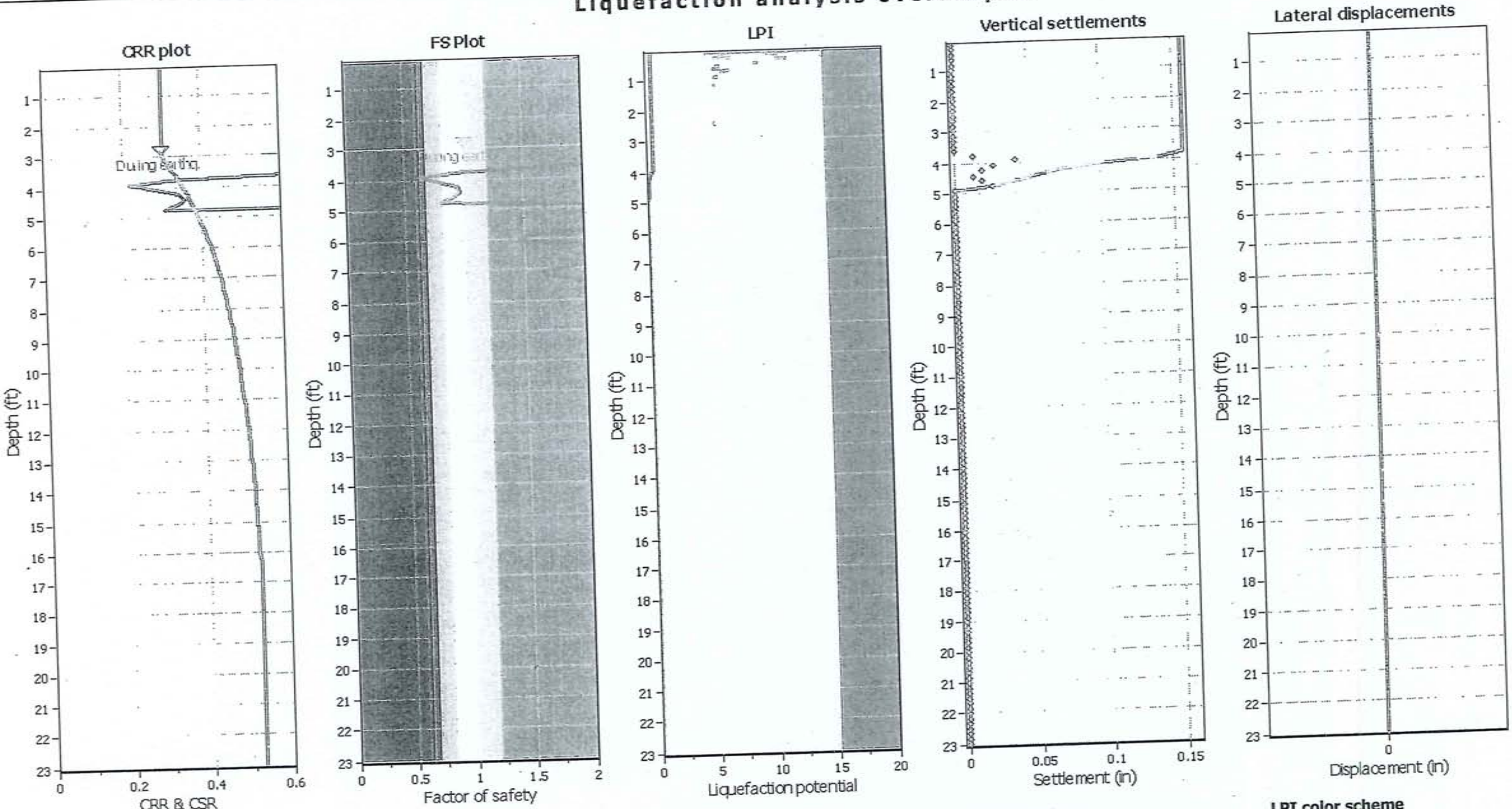
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

Liquefaction analysis overall plot



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

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

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LIQUEFACTION ANALYSIS REPORT

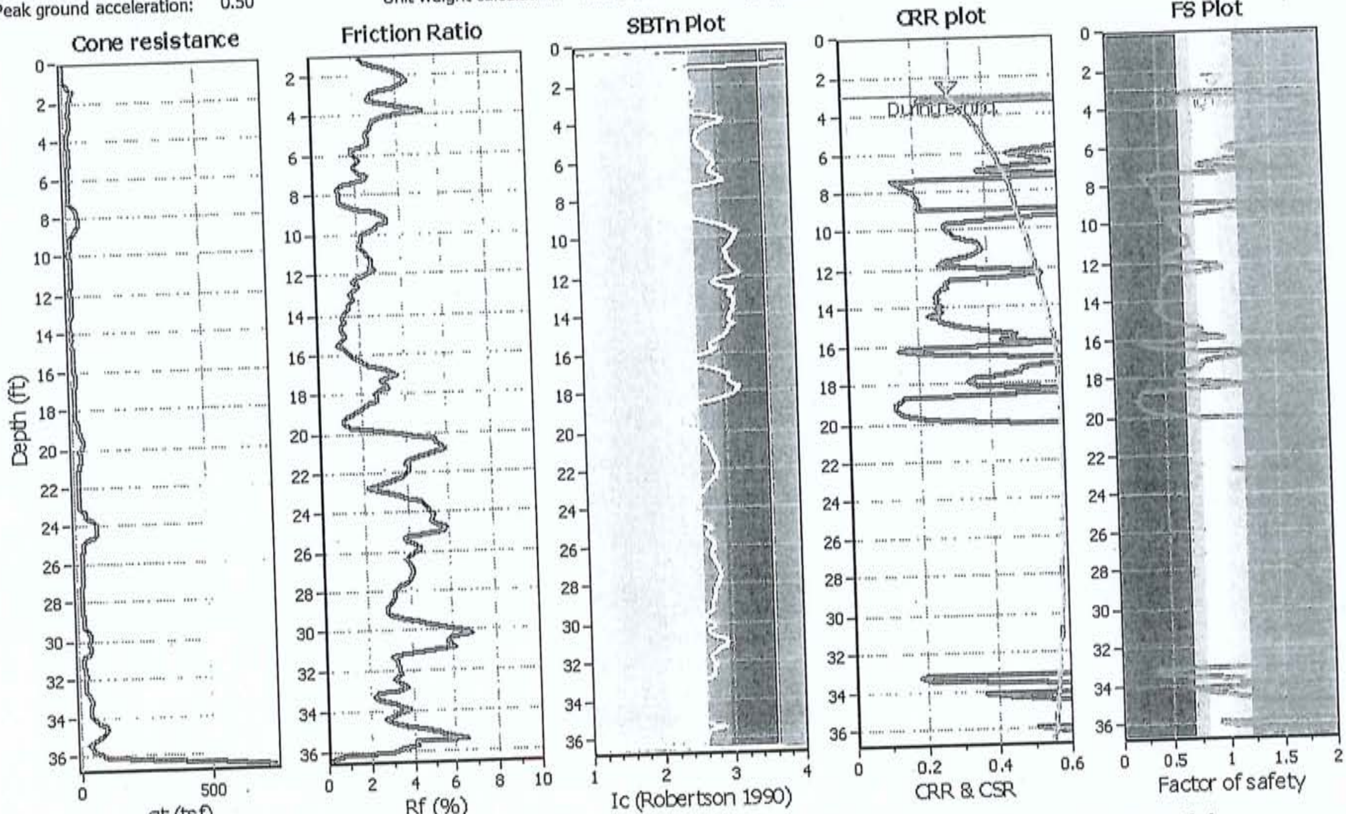
Project title : 3833 Redwood Highway

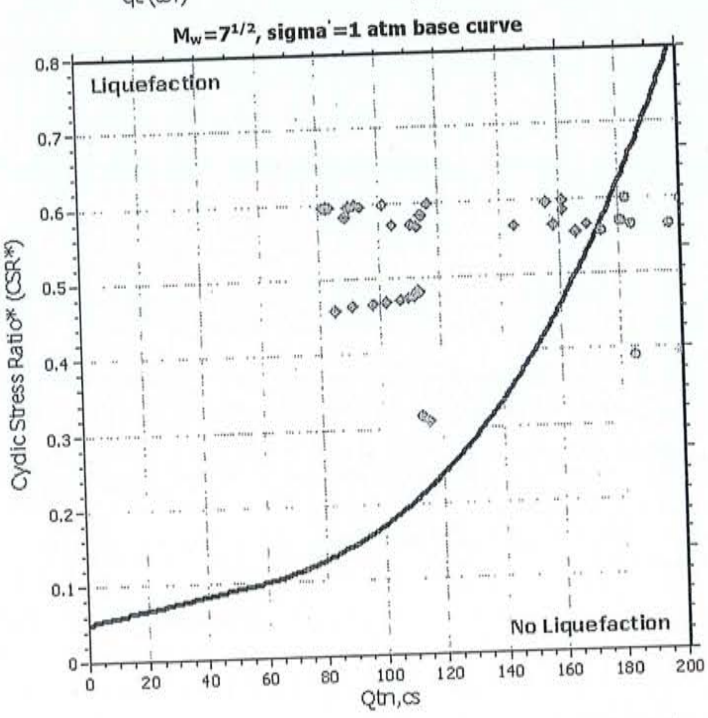
Location : San Rafael, CA

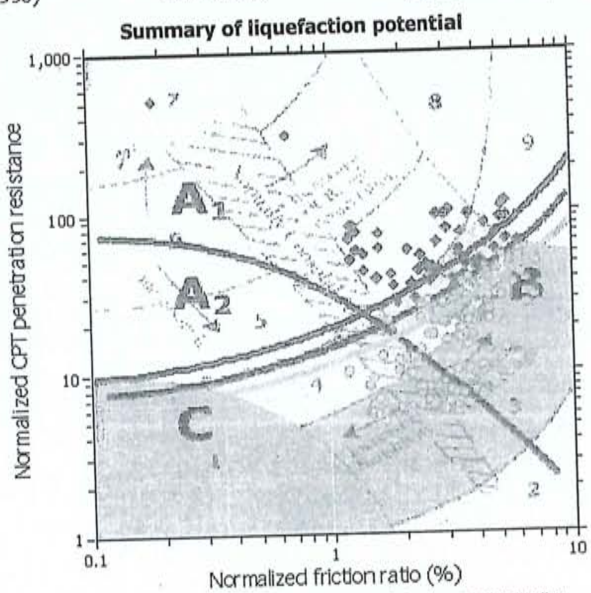
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		



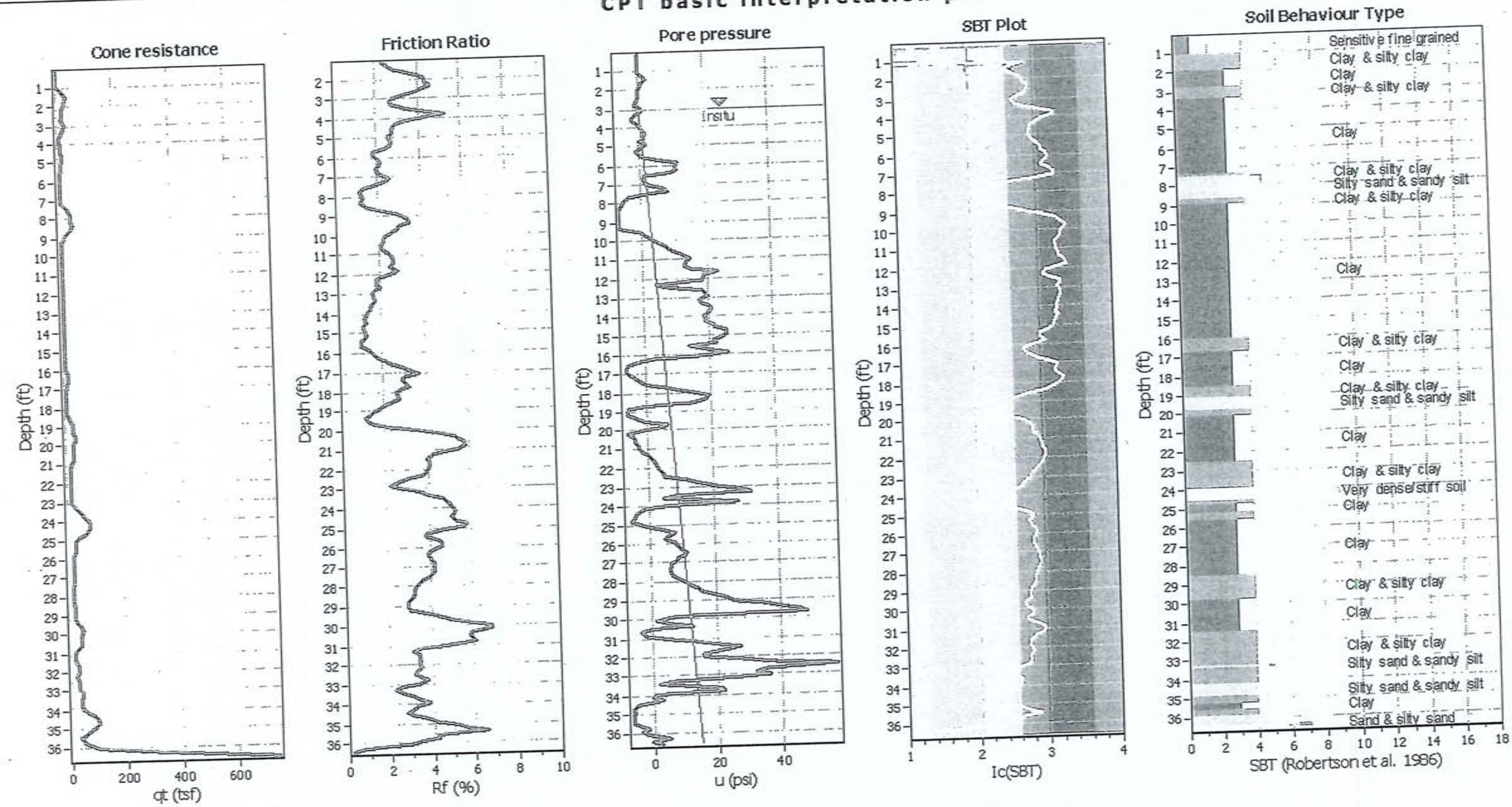




Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

This software is licensed to: ENGEO Incorporated

CPT basic interpretation plo



Input parameters and analysis data

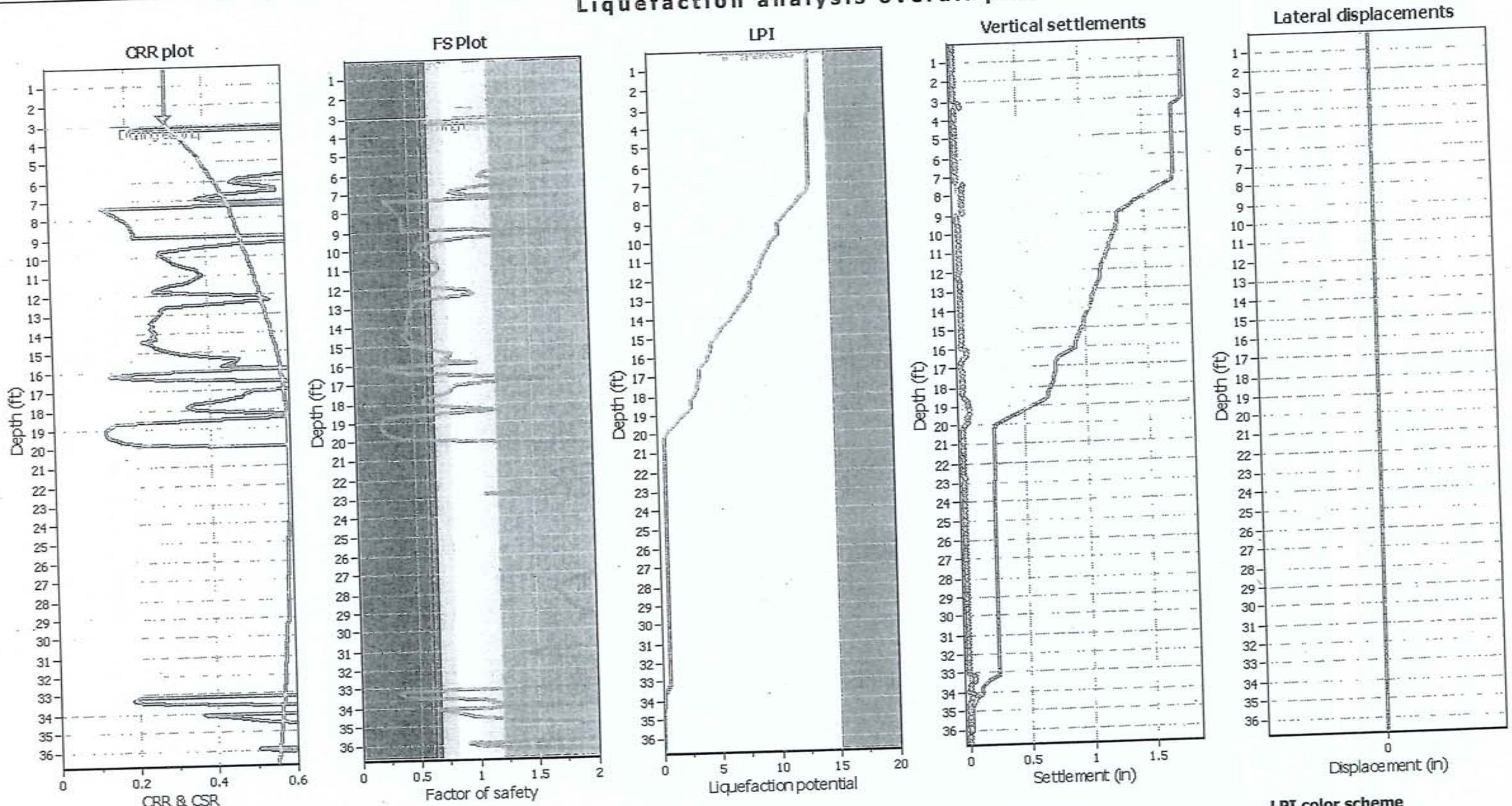
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

This software is licensed to: ENGEO Incorporated

Liquefaction analysis overall plot



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

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

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LIQUEFACTION ANALYSIS REPORT

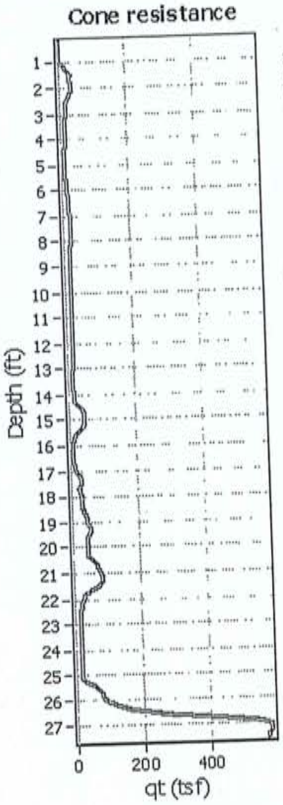
Project title : 3833 Redwood Highway **Location : San Rafael, CA**

CPT file : 1-CPT5

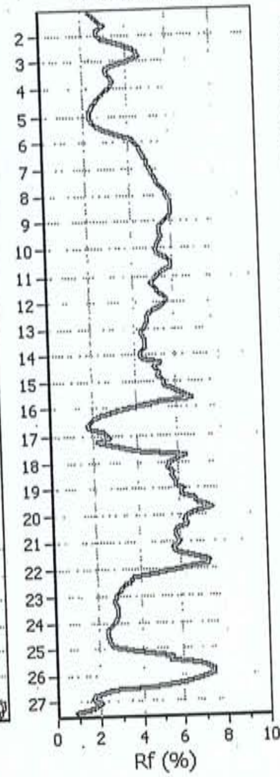
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

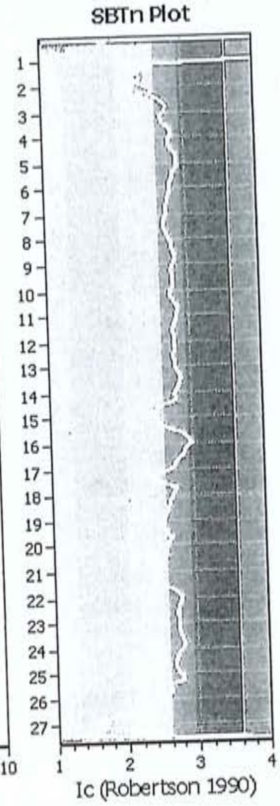
Cone resistance



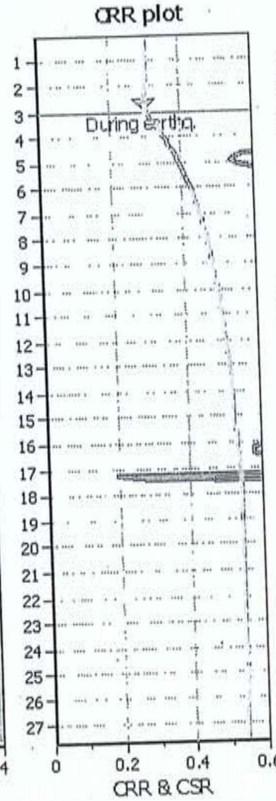
Friction Ratio



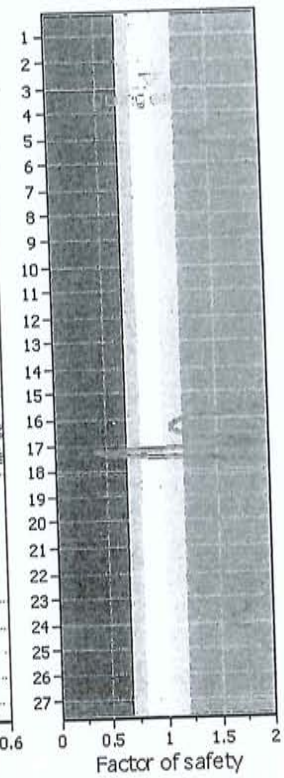
SBTn Plot



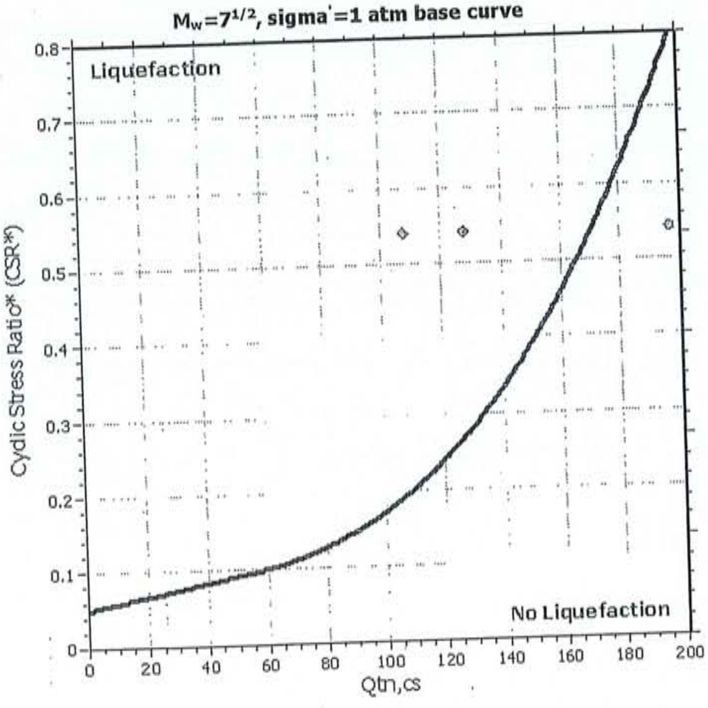
CRR plot



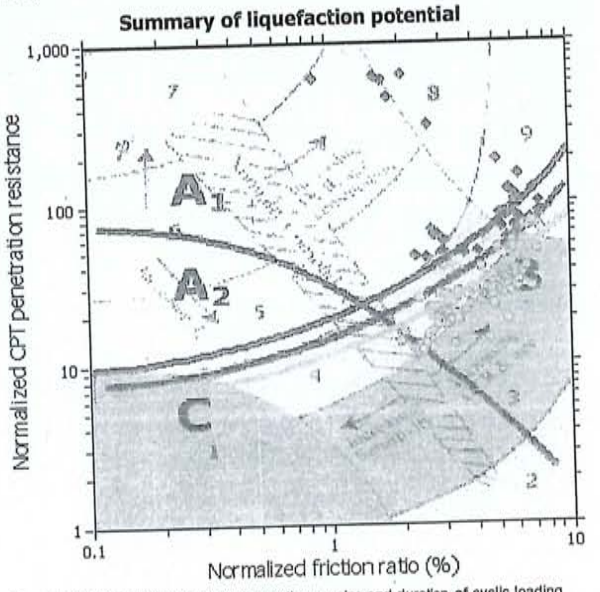
FS Plot



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

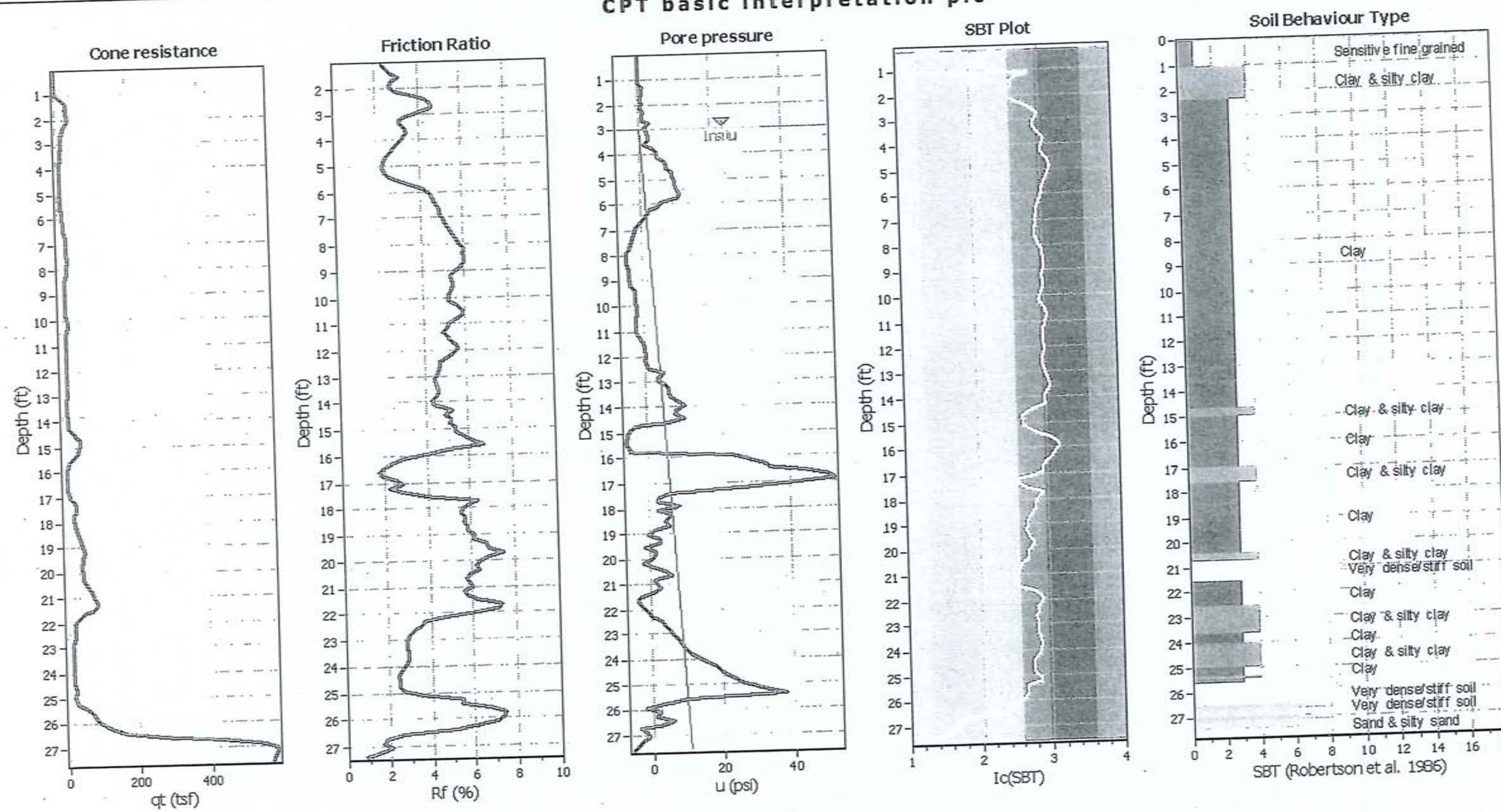


Summary of liquefaction potential



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

CPT basic interpretation plo



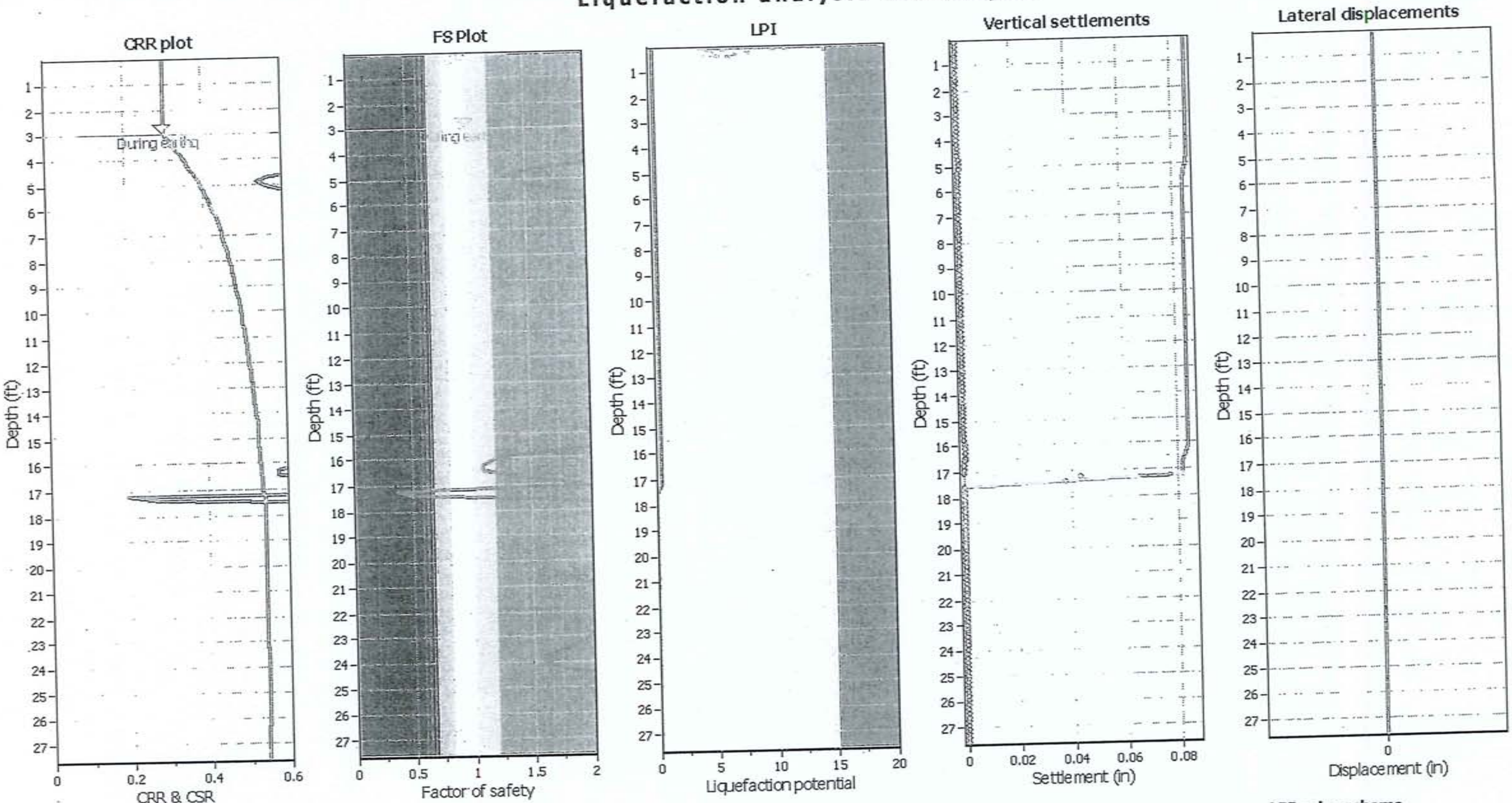
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

Liquefaction analysis overall plot



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

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

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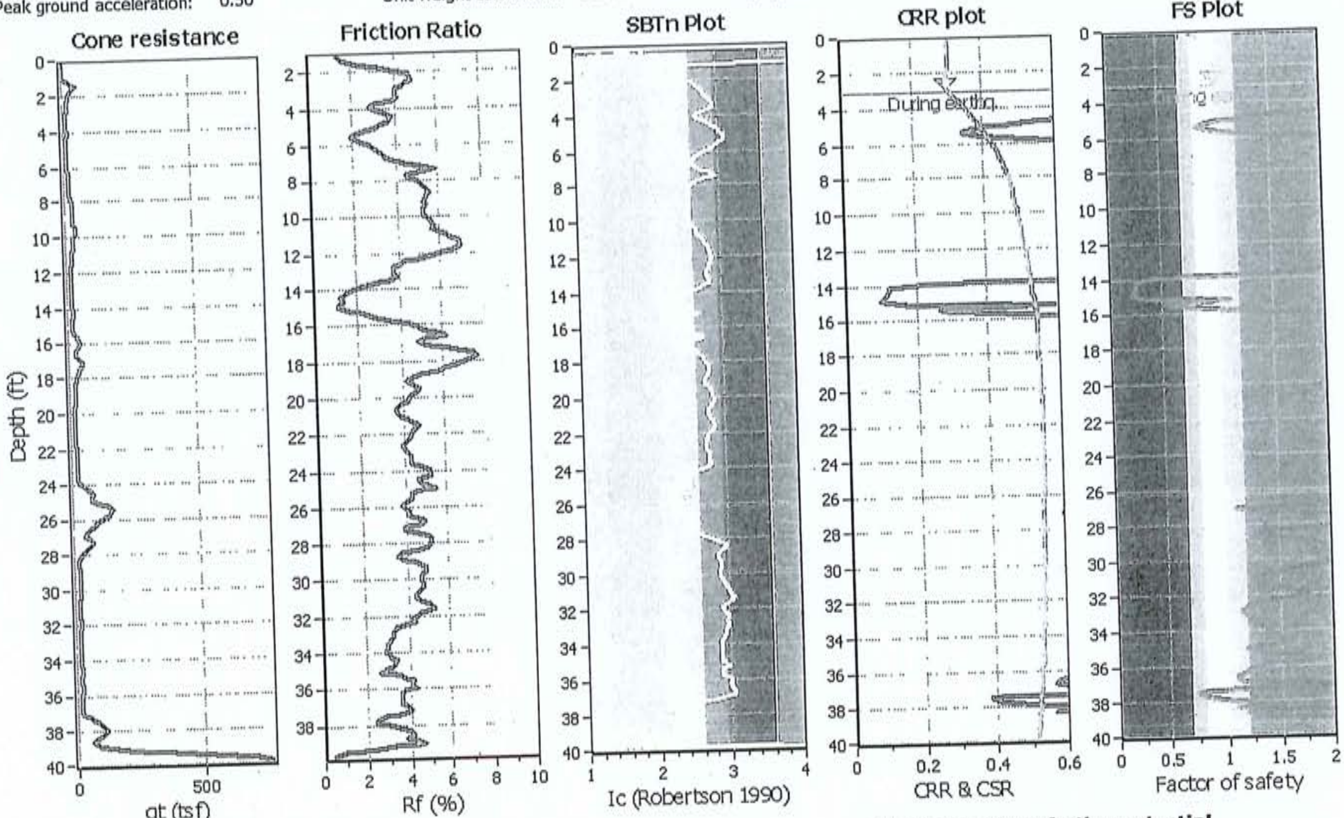
LIQUEFACTION ANALYSIS REPORT

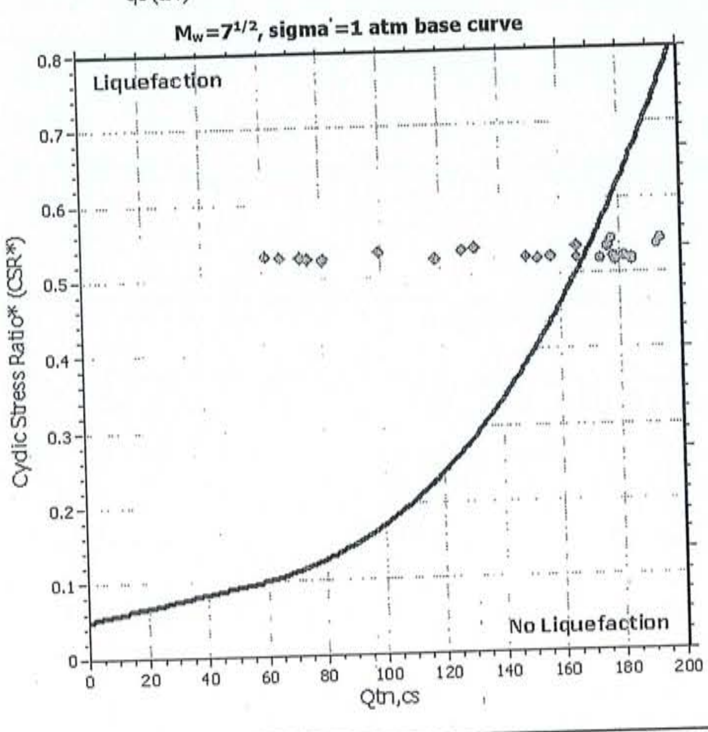
Project title : 3833 Redwood Highway **Location : San Rafael, CA**

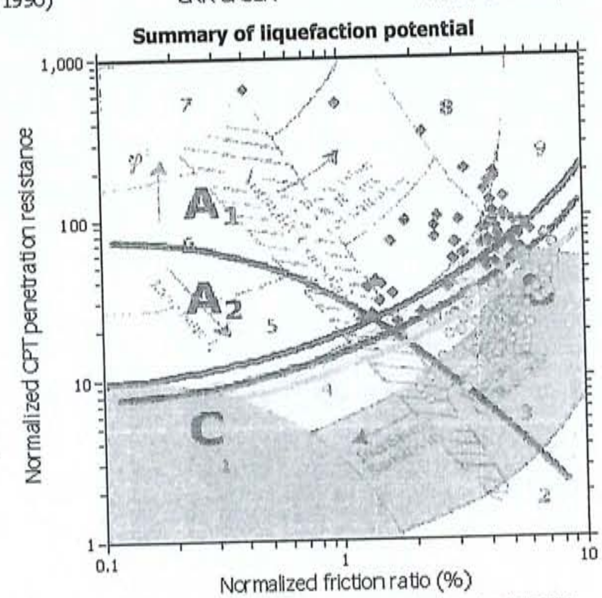
CPT file : 1-CPT6

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

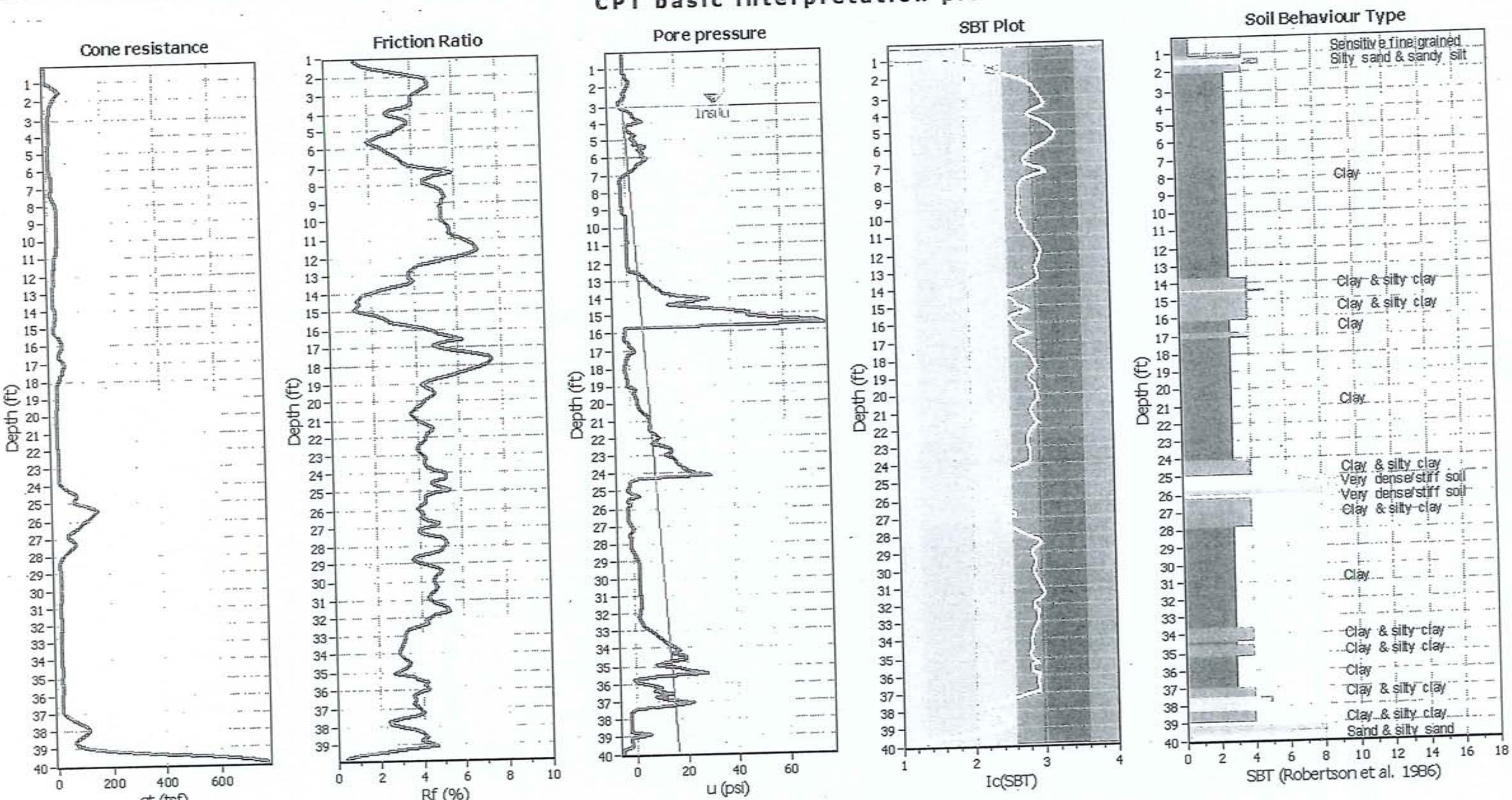






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



Input parameters and analysis data

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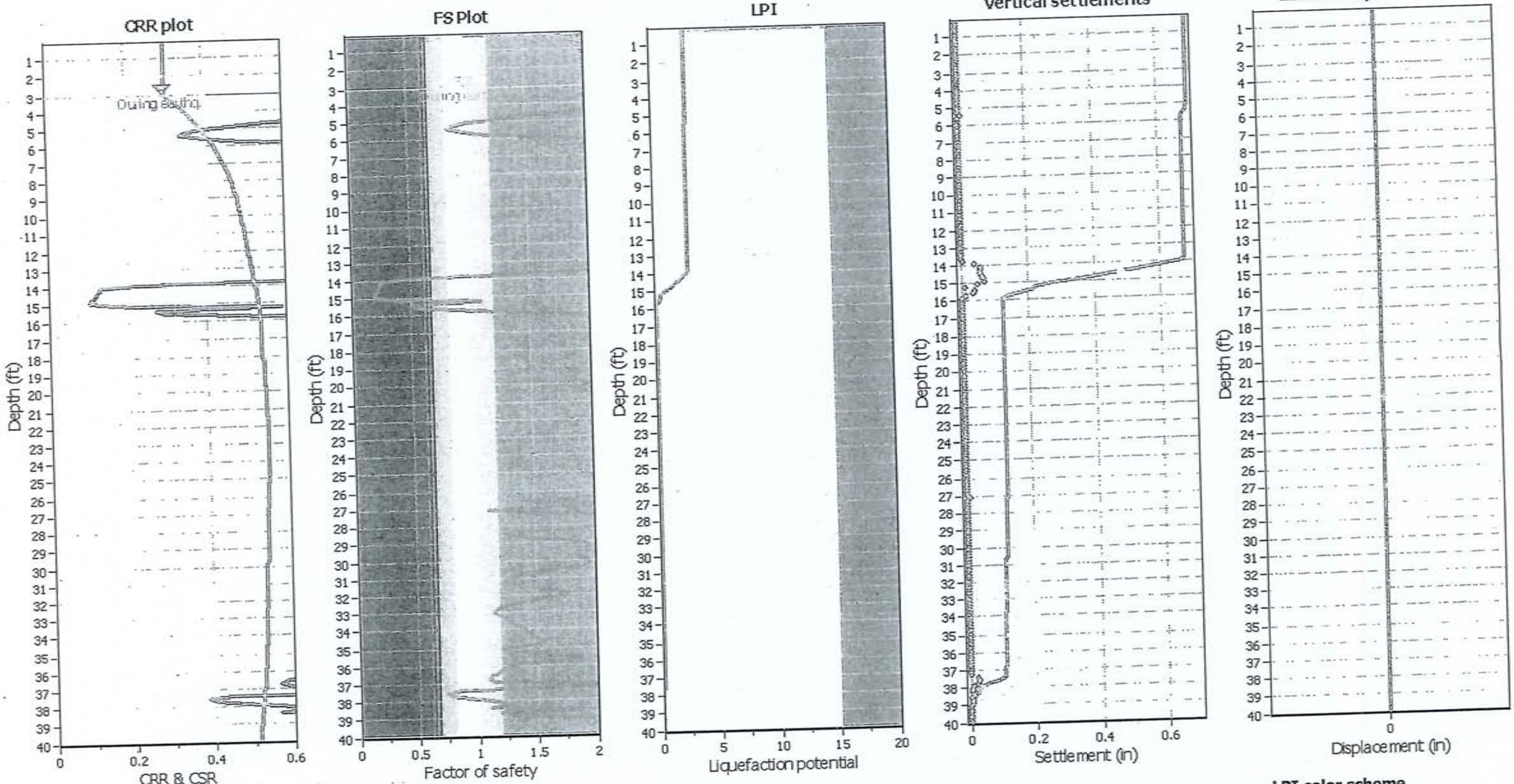
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 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

Liquefaction analysis overall plot



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Input parameters and analysis data

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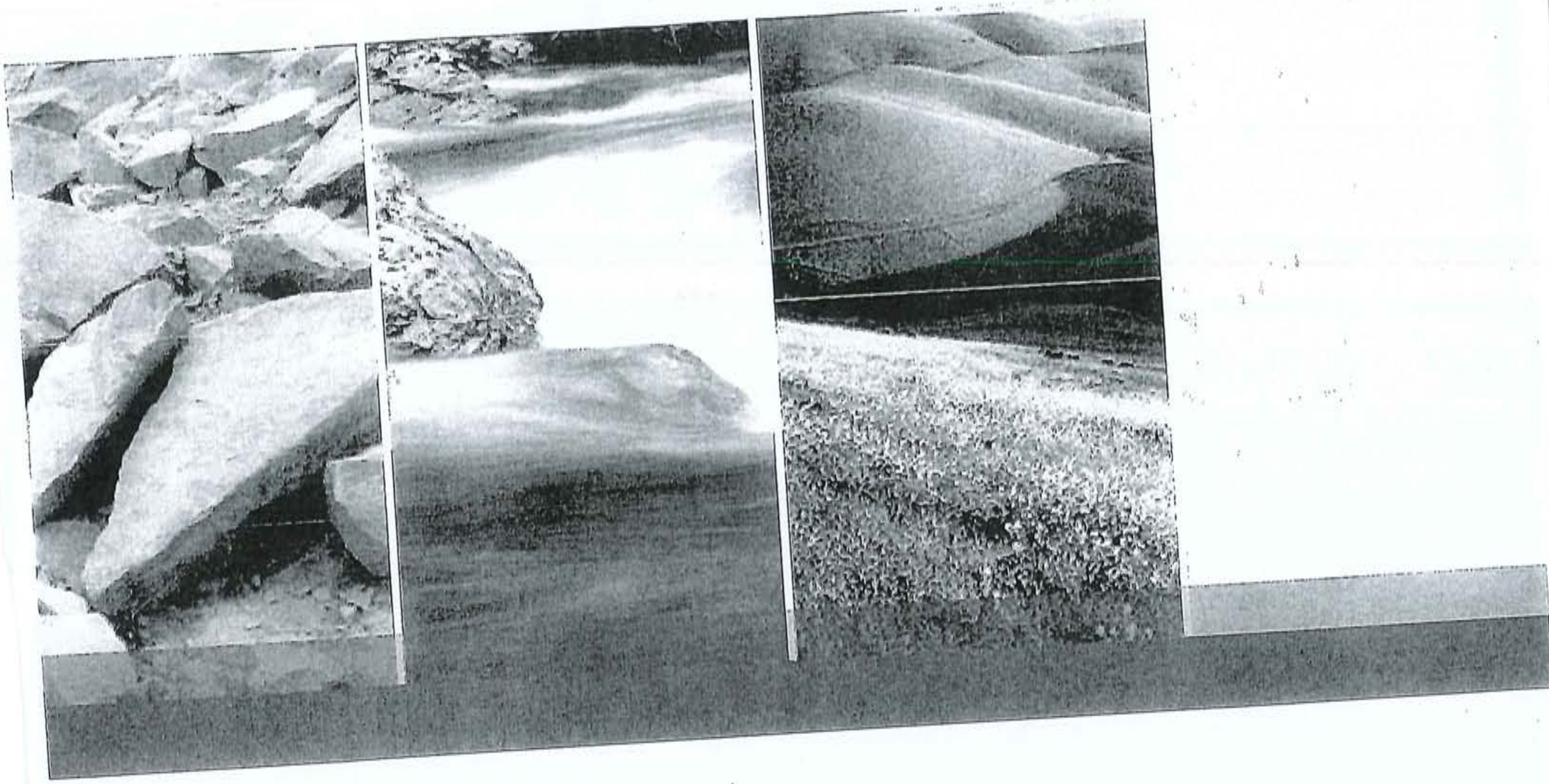

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SAN FRANCISCO
SAN JOSE
OAKLAND
LATHROP
ROCKLIN
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