

APPENDIX D

Preliminary Geotechnical Report



1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PRELIMINARY GEOTECHNICAL REPORT

SUBMITTED TO
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developURBAN
3077 Lascari Ave
Henderson, NV 89044

PREPARED BY
ENGEO Incorporated

July 12, 2021

PROJECT NO.
18918.000.001

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Mr. Neal Yung
developURBAN
3077 Lascari Ave
Henderson, NV 89044

Subject: 1430 Chapin Avenue
Burlingame, California

PRELIMINARY GEOTECHNICAL REPORT

Dear Mr. Yung:

We are pleased to present this preliminary geotechnical report for 1430 Chapin Avenue located in Burlingame, California. This report presents our preliminary geotechnical observations, as well as our conclusions and preliminary recommendations for the development of the site.

Based on our preliminary findings, it is our opinion from a geotechnical viewpoint that the site is suitable for potential development provided the findings, as well as preliminary conclusions and recommendations presented in this report are incorporated into future planning and design. Additionally, a supplemental design-level geotechnical exploration should be performed to develop site-specific design recommendations.

The main identified geotechnical concerns for the planned development at this site include:

- Existing near-surface deposits of undocumented “man-made” fill
- Existing expansive soil
- Potential liquefaction and seismically induced settlement
- Existing shallow and variable groundwater depths
- Presence of adjacent structures and existing retaining walls supporting the site
- Potential presence of below-grade shoring elements

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

Sincerely,

ENGEO Incorporated


Patrick Lam, CEG


Aryan Noroozi

pl/tpb/an/ar

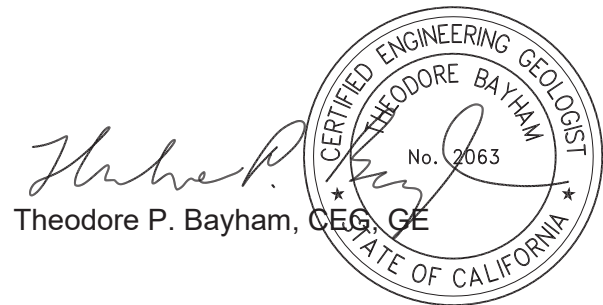
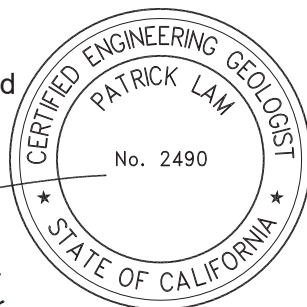


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APPENDIX B – Preliminary CPT-Based Liquefaction Analysis

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical report is to identify potential geologic hazards and geotechnical risks associated with the site and to provide preliminary recommendations to support early planning of a commercial building with below-grade parking at the site.

Our scope of services included the following tasks.

- Reviewing available literature and geologic maps
- Reviewing historical aerial photographs
- Performing CPT-based subsurface field exploration
- Preliminary analysis of geotechnical data collected
- Identifying potential geological and geotechnical constraints
- Providing preliminary geotechnical recommendations

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1.2 SITE LOCATION AND DESCRIPTION

The rectangular-shaped 0.6-acre site is located at 1430 Chapin Avenue, Burlingame, California (Figures 1 and 2). The site is zoned for mixed residential and commercial use. It is currently occupied by a single-story commercial building, paved parking area, and landscaping. The property fronts Chapin Avenue along the southeast. A single-family residence exists to the northeast, and two multi-family buildings are located to the northwest. A commercial office building with a two-level basement abuts the property to the southwest.

1.3 PROPOSED DEVELOPMENT

We understand that the conceptual future development of the property may consist of a multi-level CLT lumber commercial building. In addition, the building may include two or three levels of below-grade parking. Information regarding structure type is not available at the time. It is anticipated that the future development will include site improvements such as underground utilities, stormwater treatment, and landscaped areas.

2.0 FINDINGS

2.1 SITE HISTORY

Historical topographic maps and aerial photographs were reviewed for information regarding past conditions and land use at the property. To review site conditions between 1941 to 2021, we used historicaerials.com, UC Santa Barbara, and Google Earth images.

Based on our review, the aerial images show that the site and surrounding buildings were occupied by single-family residences. Between 1968 and 1980, the site was redeveloped to a commercial building. The buildings surrounding the site were mainly reconstructed between 1968 and 1982 to multi-story residential building. The site appears relatively unchanged since 1982.

Several retaining walls support the property. To the southeast (1424 Chapin Avenue), a retaining wall accommodates the grade change down to the single-family residence/office building, which is up to approximately 2 to 3 feet lower than the property. To the northeast (1425 and 1435 Bellevue Avenue), a retaining wall accommodates a grade change down to the adjoining parking level, which is approximately 5 to 8 feet below the property to the northwest (1440 Chapin Avenue).

2.2 REGIONAL GEOLOGY

The site is located in the northern California Coast Ranges, which are a series of northwesterly trending uplifted ranges and intervening valleys. According to geologic mapping by Pampeyan (1994), as shown on Figure 3, the site is shown as Pleistocene older alluvium (Qoa). These deposits consist of weathered, unconsolidated to moderately consolidated gravel, sand, silt, and clay in various proportions and combinations generally associated with older alluvial fan deposits.

Figure 4 depicts the presence of a historical creek on the parcels adjoining the site. The channelized creek (blue) is visible north of the site (green) on the 1941 historical aerial photo below. These features are consistent with the seismic hazards zone map (Figure 5), which depicts the area as requiring investigation for liquefaction hazards. This study addresses the zone of required liquefaction investigation requirement.

EXHIBIT 2.2-1



2.3 SITE SEISMICITY

The San Francisco Bay Area contains numerous active earthquake faults. Nearby active faults include the Andreas, Pilarcitos, and San Gregorio faults, located 2.65, 4.34, and 6.69 miles from the site, respectively. The California Geologic Survey qualifies an active fault as one that has had surface displacement within Holocene time (about the last 11,000 years). The active faults are identified in the region by Bryant and Hart, 2007. Figure 6 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region.

To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool and disaggregated the hazard at the peak ground acceleration (PGA) and for spectral periods of 0.5, 1.0, and 5.0 seconds for a 2,475-year return period, with the resulting faults listed below in Table 2.3-1.

TABLE 2.3-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

SOURCE	R _{RUP}		M _w
	(KM)	(MILES)	
San Andreas (Peninsula) [8]	4.26	2.65	7.82
Pilarcitos [7]	6.98	4.34	7.35
San Gregorio (North) [9]	15.59	9.69	7.69

*Based on USGS Unified Hazard Tool: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

Latitude: 37.578189 degrees, **Longitude:** -122.350532 degrees

The site is not located within a designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site; as such, the risk of fault rupture through the site is low. Based on review of the Fault Activity Map of California (Jennings and Bryant, 2010), the Serra fault is mapped approximately 1.5 miles northwest of the site. The fault is identified as a part of certain faults in the South San Francisco Bay Area and is identified as late Quaternary in age (less than 700,000 years). The potential for surface fault rupture from seismic activity along this fault is considered very low.

2.4 PRELIMINARY FIELD EXPLORATION

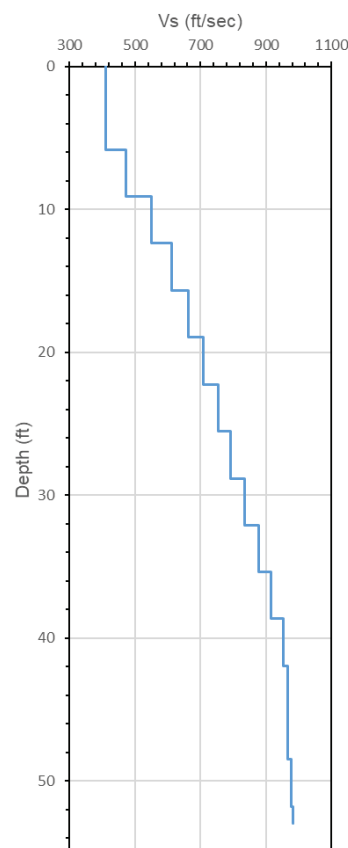
We retained the services of a contractor to advance cone penetration tests (CPT) at four locations (Figure 2) in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the friction sleeve (F_s), and pore pressure (U) (Robertson and Campanella, 1988). The CPT probes were advanced to an approximate depth of

53 feet below the ground surface (bgs). We performed shear wave velocity (V_s) measurements in ECPT-1, using the downhole seismic method specified in ASTM D-7400. We present the CPT logs and the V_s profiles in Appendix A. As shown in Appendix A, the time-averaged shear wave velocity over the top 50 feet or 15 meters (V_{s30}) representative of the site is 983 feet per second (ft/s). The results of the time-averaged, shear wave velocity are shown in Exhibit 2.4-1.

2.5 SUBSURFACE CONDITIONS

Based on correlations to the CPT data, the subsurface conditions encountered consist predominantly of clay with varying silt content and cone resistance of under 100 tons per square foot (tsf). An approximately 10-foot-thick layer of dense material was encountered from 29 to 41 feet bgs, and from 23 to 33 feet bgs, in ECPT-1 and ECPT-4, respectively.

EXHIBIT 2.4-1: Vs Measurements



2.6 GROUNDWATER CONDITIONS

We performed pore pressure dissipation tests in the three CPTs. These tests suggest that the groundwater level is approximately 14.4 feet bgs, which corresponds to approximately Elevation 23 feet (datum WGS 1984), as presented in Table 2.6-1 below.

TABLE 2.6-1: Recorded Groundwater Levels (at Time of Exploration)

CPT	APPROXIMATE EXISTING GROUND ELEVATION (feet) ²	APPROXIMATE GROUNDWATER DEPTH (feet bgs) ¹	CORRESPONDING GROUNDWATER ELEVATION (feet) ^{1,2}
ECPT-1	35	14 ½	21
ECPT-2	37	14 ½	23
ECPT-4	37	14 ½	22

1. Assumed phreatic surface based on pore pressure dissipation tests assuming hydrostatic conditions.

2. Elevation datum WGS 1984 (Google Earth).

Plate 1.2 of the Seismic Hazard Zone Report for the San Mateo Quadrangle (2019) indicates the highest historical groundwater within the site's vicinity to be between 10 and 20 feet below the ground surface. For purposes of our analyses and recommendations, we consider an appropriate groundwater depth of 10 feet below ground surface be used for initial planning. Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in irrigation and drainage patterns, and due to potential off-site dewatering activities (e.g., 1440 Chapin Avenue basement dewatering).

3.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby occurring moderate to major earthquake can generally be classified as “primary” and “secondary.” The primary seismic hazard is ground rupture, also called surface faulting. Common secondary seismic hazards include ground shaking, liquefaction (and associated settlements, lateral spreads, sand boils, etc.), and landslides. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

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, we consider the risk of potential fault rupture at the site to be low to remote.

3.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated on a fault within the nearby San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in past major events. To mitigate the effects of ground shaking, all structures should be designed using sound engineering judgment and in accordance with the latest edition of the California Building Code (CBC), as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some 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, 1999). Section 4.1 presents California Building Code (CBC, 2019) seismic design parameters.

3.1.3 Liquefaction / Cyclic Softening

The site is located within a mapped State of California Seismic Hazard Zone (CGS, 2006) for areas that may be susceptible to liquefaction (Figure 5). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective, confining stress from the overlying soil is said to have liquefied. If the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also undergo “cyclic-softening” or strength loss as a result of cyclic loading. We considered both liquefaction and cyclic-softening hazards in our analyses.

3.1.3.1 Preliminary CPT-Based Analysis

We performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq (Version 2.2) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). This software also incorporates the method introduced by Idriss and Boulanger (2008) and updated by Boulanger and Idriss (2014).

For our analysis, we used a Peak Ground Acceleration (PGAM) value of 0.96 (CBC 2019) and a maximum moment magnitude (M_w) of 7.90 based on a 2-percent in 50 years exceedance probability deaggregation and a groundwater depth of 5 feet. We performed the liquefaction and cyclic softening assessment based on the methodology by Idriss and Boulanger (2008).

Based on our preliminary analysis, fine-grained material at the project site may be potentially liquefiable. At ECPT-1, several layers of potentially liquefiable zones were encountered; the upper zone was between 10 and 24 feet and the lower zone between 28 and 38 feet. At ECPT-2, liquefiable material was encountered between 10 and 18 feet and between 22 and 28 feet. At ECPT-3, liquefiable layers were encountered between 21 to 26 feet and between 34 to 39 feet. At ECPT-4, liquefiable layers were encountered between 8 to 28 feet and between 21 to 38 feet. We estimate potential seismically induced settlement for a structure with a one-level basement to be up to 3 inches and with a two-level basement to be up to 1½ inches.

The CPT-based liquefaction analysis provides a preliminary indication of the liquefaction hazard at the site. The settlement estimates described in this section are not settlement design recommendations. Liquefaction and cyclic-softening susceptibility and associated seismically induced settlement should be further evaluated during design-level study with laboratory testing of the susceptible soil as described in Section 3.1.3.4.

3.1.3.2 [Liquefaction-Induced Surface Rupture](#)

In addition to the above preliminary liquefaction analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. We based our analyses and review on guidelines provided by Ishihara (1985), and Youd and Garris (1995). Based on our preliminary assessment, site improvements are not susceptible to secondary ground effects such as sand-boils, cracks, and other surface expression.

3.1.3.3 [Lateral Spreading](#)

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope. The site is not located near a sloped face or free face soil and given the depths of potentially liquefiable material on site, the risk of lateral spreading at the site is low.

3.1.3.4 [Further Evaluation of Liquefaction](#)

The liquefaction potential and settlement analyses should be further evaluated during design-level exploration. Blow counts and soil samples should be collected from layers identified as potentially susceptible to liquefaction in our CPT-based assessment. The field blow counts should be used to further evaluate the liquefaction susceptibility of granular layers.

The cyclic-softening hazard for fine-grained materials should be analyzed with laboratory test results from in situ soil samples. Bray and Sancio (2006) observed that soil with a plasticity index (PI) less than 12 and a water content (w_c) to liquid limit (LL) ratio of more than 0.85 is susceptible to liquefaction/cyclic-softening. Soil with PI greater than 18 and/or water content divided by liquid limit (w_c/LL) less than 0.8 was deemed not susceptible to liquefaction because it is too plastic and/or the water contents are too low. An evaluation including the Bray and Sancio 2006 criteria should be used during design-level study to develop conclusions regarding the risk for cyclic softening of fine-grained materials at the site.

3.2 **UNDOCUMENTED EXISTING FILL**

Based on the history of development at the site (see historical photograph in Section 2.2), buried foundation elements, utilities, and existing undocumented “man-made” fill are present throughout the site. Given the geometry of existing retaining walls at the property line and a visual comparison of site elevations between adjoining properties, we expect the presence of undocumented non-engineered fill across the site. Note, if a subterranean garage is planned across the entire footprint of the site, we would expect that most, if not all, non-engineered fill would be mitigated by removal.

If non-engineered fill exists beneath proposed structural areas, the non-engineered fill should be subexcavated and replaced with engineered fill. The depth of subexcavation should be determined during design-level study once development plans are available.

3.3 EXPANSIVE SOIL

Expansive soil can shrink and swell because of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Successful construction on expansive soil requires special attention during grading. Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil is a generally cost-effective measure to address the expansive potential of the foundation and improvement supporting soil. It is imperative to keep exposed soil moist by occasional sprinkling. If the soil dries, it is extremely difficult to remoisturize (because of its clayey nature) without excavation, moisture conditioning, and recompaction.

Based on our experience in the area, the expansion potential for clay-bearing soil is considered moderate to high. The expansion potential of near-surface native soil should be evaluated during design-level study.

3.4 SOIL CORROSION POTENTIAL

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

3.5 SHALLOW GROUNDWATER

Based on our findings described in Section 2.6 of this report, dewatering may be necessary during utility installation. Shallow groundwater will also impact design of below-grade improvements if incorporated in the final development plan for the site. Generally, shallow groundwater conditions may result in the following impacts to site development.

1. Require construction dewatering
2. Result in unstable conditions at the base of excavation requiring stabilization prior to improvement construction
3. Cause moisture damage to sensitive floor coverings
4. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, and damage to computers and other sensitive equipment
5. Require waterproofing for basement structures, if any

4.0 PRELIMINARY GEOTECHNICAL CONCLUSIONS

Based on our review of available documentation, it is our preliminary opinion from a geotechnical viewpoint that this site is suitable for the planned development, provided the preliminary conclusions and recommendations are incorporated into future project planning. The main identified geotechnical concerns for the planned development at this site include:

- Existing near-surface deposits of undocumented “man-made” fill.
- Existing expansive soil.
- Potential liquefaction and seismic induced settlement.
- Existing shallow and variable groundwater depths.
- Presence of adjacent structures and existing retaining walls supporting the site.
- Potential presence of below grade shoring elements.

Supplemental design-level geotechnical exploration should be performed as part of the design process, which we anticipate will require field exploration, including borings and laboratory testing to provide specific recommendations for grading and excavations, remedial grading measures, expansive soil characterization and mitigation, foundations, and drainage. The exploration will also allow for a detailed evaluation of the geotechnical issues identified above and the opportunity to provide recommendations to mitigate potential geotechnical/geological hazards.

4.1 PRELIMINARY FOUNDATION CONSIDERATIONS

Based on preliminary findings, it is our assessment that shallow foundation systems, such as structurally reinforced mat foundations, may be suitable provided these may be designed to tolerate potential differential movements resulting from static compression and seismically induced settlements. The depressed portion of the building will need to consider presence of groundwater, potential uplift forces, and waterproofing design elements.

Depending on foundation design tolerances, it may be necessary to perform mitigation through ground improvement measures. Moreover, if shallow mat foundations are determined not to be adequate, then deep foundation systems may be required for building support, such as driven pile foundations.

A supplemental design-level study should be performed to develop design criteria for foundations, and this should include characterization and evaluation of static consolidation settlement, shallow groundwater, expansive soil, and corrosion potential. Depending on the final structural loads and foundation system, various foundation alternatives may be considered for support proposed structure(s).

4.2 2019 CBC SEISMIC DESIGN PARAMETERS

Based on the CPT shear wave velocity testing, we classified the site as Site Class D in accordance with the 2019 California Building Code (CBC). The 2019 CBC is based on the 2016 edition of the American Society of Civil Engineers document titled “Minimum Design Loads and Associated Criteria for Buildings and Other Structures” (ASCE 71-16).

We provide the 2019 CBC seismic design parameters in Table 4.2-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 4.2-1: 2019 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.03
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.84
Site Coefficient, F_A	1.0
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.03
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.35
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.96
Long-period transition-period, T_L (sec)	12

*Requires site-specific ground-motion hazard analysis

Latitude: 37.578189 degrees, **Longitude:** -122.350532 degrees

As noted in Table 4.2-1, a ground motion hazard analysis needs to be performed in accordance with Section 11.4.8 of ASCE 7-16, for structures on Site Class D with S_1 greater than or equal to 0.2 unless the structural engineer elects to take the exception described in Section 11.4.8. The necessity to perform a site-specific seismic hazard analysis should be coordinate with the Structural Engineer. This analysis can be provided during the design-level study.

4.3 CONSIDERATIONS FOR UNDERSEEPAGE, BASE HEAVE, AND UPLIFT FORCES

For planned depressed building levels and construction, the excavation and its subgrade may experience heave and recompression. Shoring walls or groundwater cut-off systems should be designed to reduce the risk of subgrade heave on the project. The risk of underseepage, base heave, and hydrostatic uplift should be assessed as part of the supplemental design-level report once development plans are finalized. Temporary construction shoring and dewatering recommendations should be developed as part of the supplemental design-level study, if subterranean levels are proposed.

4.4 CONSIDERATIONS FOR ADJACENT STRUCTURES

Plans for future development and construction should consider the presence of existing structures on adjacent properties. Specifically, shoring, retaining wall support, and/or underpinning as deemed necessary for existing developments east and north of the property.

Demolition and construction of temporary shoring for any contemplated basement structure need to consider sequencing and support of the existing parking areas to the east and north. Furthermore, if shoring systems contemplate tie-backs and/or post grouting, the project needs to consider the risks to adjacent properties, including, but not limited to the amount of soil cover above the tie backs.

The adjacent structure to the west (1440 Chapin Avenue) consists of a two-level below-grade parking structure. The means and methods of temporary shoring used during construction of the off-site structure is not yet known, but could likely have incorporated tiebacks that encroach beneath the project site. Review of as-builts, if available at the City of Burlingame, should be reviewed during final design.

If below-grade excavation is contemplated to extend below the adjacent basement, then the temporary shoring and permanent basement walls should consider adjacent structure surcharge and at-rest pressures during final design. Alternatively, a partial three-level basement could be considered with the three-level portion of the basement situated away from 1440 Chapin Avenue.

4.5 PRELIMINARY GRADING RECOMMENDATIONS

4.5.1 Site Preparation

Site preparation should commence with demolition of all existing surface and underground improvements. Care should be taken during site preparation to ensure all buried improvements remaining from previous site development are identified and removed. All excavations should be cleaned to a firm undisturbed soil surface, as determined by our representative. No loose or uncontrolled backfilling of depressions outside or below building envelopes is permitted. The requirements for backfill materials and placement operations will be provided in the design-level report. Grading operations should be observed and tested by our field representative.

Areas containing surface vegetation or organic-laden topsoil within the areas to be improved should be stripped to an appropriate depth to remove these materials. The amount of actual stripping and tree root removal should be determined in the field by our representative at the time of construction. Subject to approval by the Landscape Architect, strippings and organically contaminated soil can be used in landscape areas. Otherwise, such soil 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.

4.5.2 Acceptable Fill Material

On-site soil material may be suitable as fill material (pending environmental approval) provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension. Imported fill materials should meet the above requirements and have a plasticity index less than 20 and at least 20 percent passing the No. 200 sieve. It is important that we sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

4.5.3 Surface Drainage

The project Civil Engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.3 specifies minimum slopes for pervious surfaces of 5 percent away from foundations. At a minimum, we recommend the following.

1. Roof downspouts should discharge into closed conduits and direct away from foundations to appropriate drainage devices.
2. Design should consider the use of surface drainage collection system to reduce ponding of water at the ground surface near the foundation, pavements, or exterior flatwork.

4.6 TEMPORARY CONSTRUCTION DEWATERING

Where shallow groundwater is present, below-grade excavations may require temporary dewatering during construction to keep the excavation and working areas reasonably dry. In general, groundwater level within excavations should be lowered and maintained below the bottom of the base of the excavation.

The selection of equipment and method should be determined by the contractor. The dewatering system implemented should be selected to impose minimal impact on the groundwater level surrounding the proposed excavations. The dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering could cause settlement of the general area and affect existing improvements in the vicinity of the site.

4.7 PAVEMENT DESIGN

4.7.1 Flexible Pavement

Based on our field exploration, we assume that site soil will likely have a resistance (R-value) value of 5, the minimum value. The following sections have been determined based on an assumed R-value of 5 according to the method contained in the Highway Design Manual by CALTRANS.

TABLE 4.7.1-1: Preliminary Asphalt Concrete Pavement Design

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (inches)	CLASS 2 AGGREGATE BASE (inches)
4	3	8
5	3½	10
6	4½	13
7	5½	16

Notes: AB is Aggregate Base Class 2 Material with minimum R = 78

The Civil Engineer should determine the appropriate Traffic Indexes (TI) based on the estimated traffic loads and frequencies.

4.7.2 Rigid Pavement

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

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Provide concrete with a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

5.0 DESIGN-LEVEL GEOTECHNICAL REPORT

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

- Additional field exploration and laboratory testing to support design-level for shoring and foundation recommendations.
- Design-level analyses related to geologic and geotechnical hazards.
- Site-specific seismic hazard analysis based on CBC 2019 (ASCE 7-16).
- Design recommendations for structural mat foundation systems.
- Considerations during design of dewatering.
- Design-level earthwork and improvement design and construction recommendations.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the 1430 Chapin Avenue project discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

Our services did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials should be notified immediately.

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

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to ENGEO's recommendations. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include 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.

SELECTED REFERENCES

- Boulanger, R. W., & Idriss, I. M., 2008. Soil liquefaction during earthquake. Engineering monograph, EERI, California, USA, 266.
- Bray, J. D., & Sancio, R. B, 2006, Assessment of the liquefaction susceptibility of fine-grained soils, Journal of Geotechnical and Geoenvironmental Engineering, 132(9), 1165-1177.
- Bryant, W. A., & Hart, E. W., 2007, Fault-Rupture Hazard Zones in California: Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps. California Geological Survey Special Publication 42, 41.
- California Building Code, 2019.
- California Department of Transportation (Caltrans) (2020), Highway Design Manual.
- Historical Aerials, www.historicaerials.com.
- Jennings, C.W. and Bryant, W.A., 2010, Fault Activity Map of California, State of California, Department of Conservation, California Geologic Survey.
- Pampeyan, E.H., 1994, Geologic map of the Montara Mountain and San Mateo 7-1/2' quadrangles, San Mateo County, California, Scale 1:24,000.
- Robertson, P. K. and Campanella, R. G., 1988, Guidelines for Geotechnical Design Using CPT and CPTU Data. Civil Engineering Department, University of British Columbia.
- Robertson, P. K., 2009, Performance based earthquake design using the CPT, Gregg Drilling and Testing, Inc.
- UCSB Library, University of California, Santa Barbara, Geospatial Collection, Flight C-6660-355, March 23, 1941, Scale 1:7,200.
- Youd, T. L. and I. M. Idriss, 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils.
- Zhang, G. Robertson. P.K, Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.



FIGURES

FIGURE 1: Vicinity Map

FIGURE 2: Site Plan

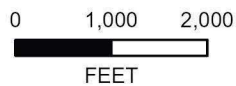
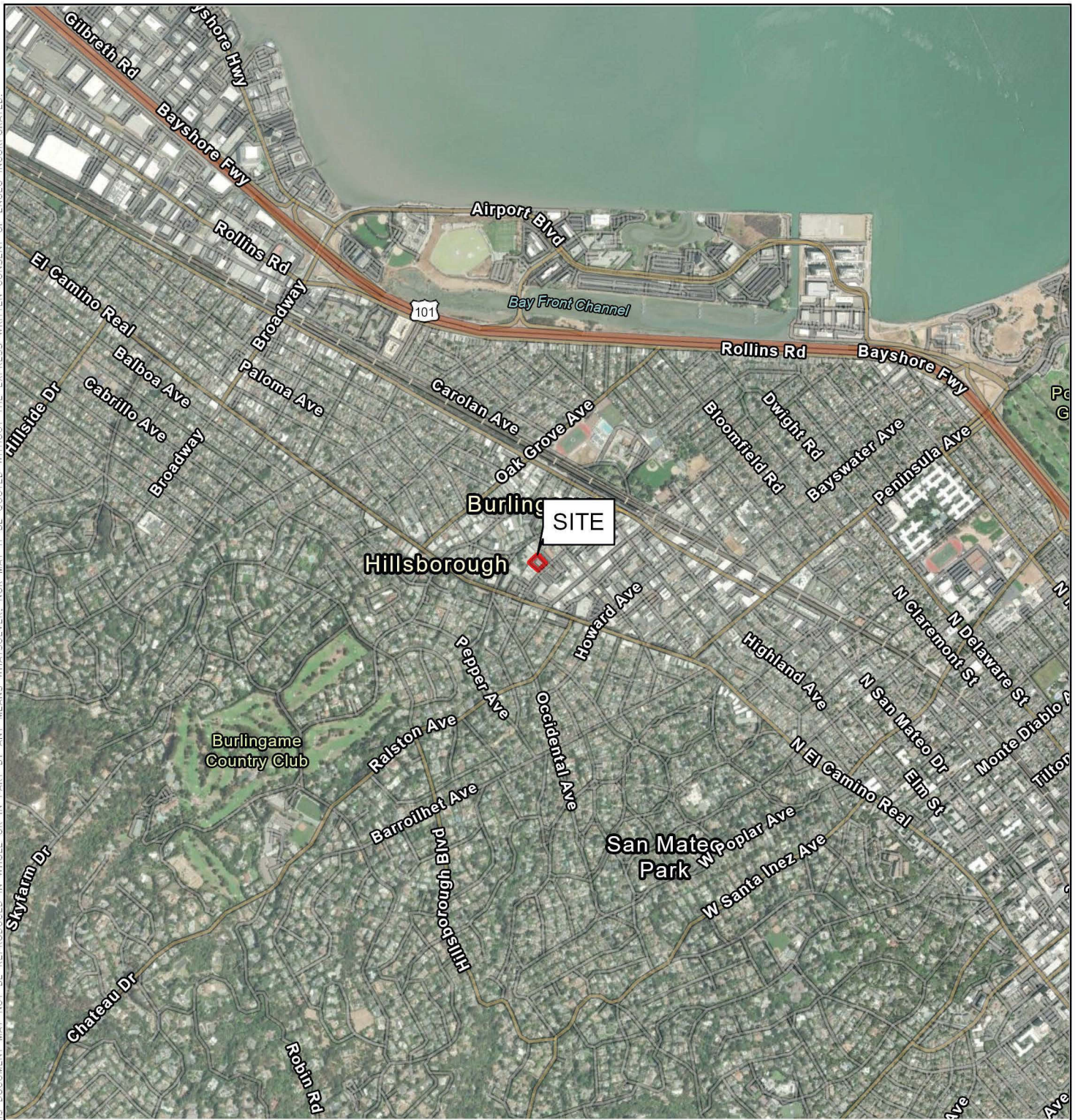
FIGURE 3: Regional Geologic Map

FIGURE 4: Historic Creek Map

FIGURE 5: Seismic Hazard Zones Map

FIGURE 6: Regional Faulting and Seismicity

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BASEMAP SOURCE: ESRI MAPPING SERVICE 11/1/2019



VICINITY MAP
1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PROJECT NO. : 18918.000.001

SCALE: AS SHOWN

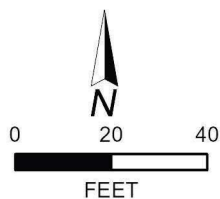
DRAWN BY: JV

CHECKED BY: TPB

FIGURE NO.

1

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

 CONE PENETRATION TEST (ENGEO, 2021)

BASEMAP SOURCE: GOOGLE EARTH MAPPING SERVICE 2020



SITE PLAN
1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PROJECT NO. : 18918.000.001

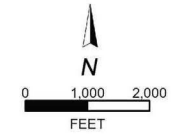
SCALE: AS SHOWN

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FIGURE NO.

2



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qam	MEDIUM-GRAINED ALLUVIUM
Qac	COARSE-GRAINED ALLUVIUM
Qsr	SLOPE WASH, RAVINE FILL, AND COLLUVIUM
Qm	BAY MUD
Qb	BEACH DEPOSITS
Qf1	ARTIFICIAL FILL, UNIT 1
Qoa	OLDER ALLUVIUM
Qc	COLMA FORMATION
QTs	SEDIMENTARY DEPOSITS, UNDIVIDED
Qsr	SLOPE WASH, RAVINE FILL, AND COLLUVIUM
Qyl	YOUNGER LANDSLIDE DEPOSITS
Tme	MERCER FORMATION
fs	SANDSTONE
fg	GREENSTONE
fc	CHERT
fsr	SHEARED ROCK
sp	SERPENTINITE

BASE MAP SOURCE: PAMPEYAN, 1994

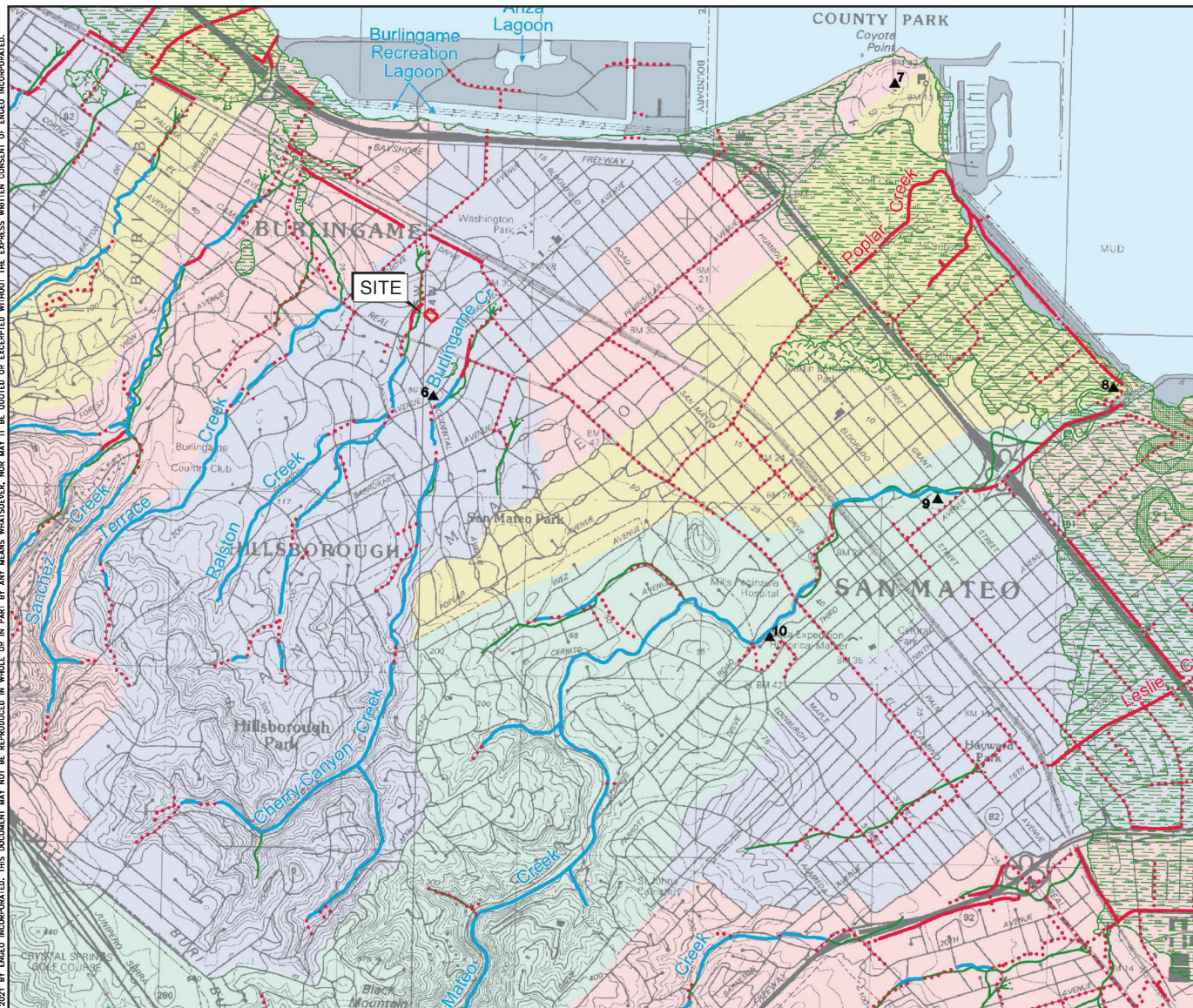


REGIONAL GEOLOGIC MAP
1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PROJECT NO. : 18918.000.001
SCALE: AS SHOWN
DRAWN BY: JV CHECKED BY: TPB

FIGURE NO.
3

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- CREEKS, WATERSHED AREA $\geq 0.2 \text{ km}^2$
- UNDERGROUND STORM DRAINS $\geq 24"$
- ENGINEERED CHANNELS
- BAY, OCEAN OR NATURAL LAKES
- ARTIFICIAL BODIES OF WATER
- BAY FILL
- PRESENT WATERSHEDS
- MODERN TIDAL MARSH FORMED AFTER ~1850
- HISTORICAL TIDAL MARSH, CIRCA 1850, STILL PRESENT
- HISOTICAL FEATURES, CIRCA 1850**
- CREEKS, BURIED OR DRAINED, DASHED WHERE LOCATION UNCERTAIN
- SHORELINE OR MARSH BOUNDARY
- EPHEMERAL CREEK
- LAKES
- WATER SPREADS OVER THE GROUND
- WILLOW GROVE
- TIDAL MARSH AND SLOUGHS
- NOW FILLED LAND

BASE MAP SOURCE: SAN FRANCISCO ESTUARY INSTITUTE, 2006

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Expect Excellence

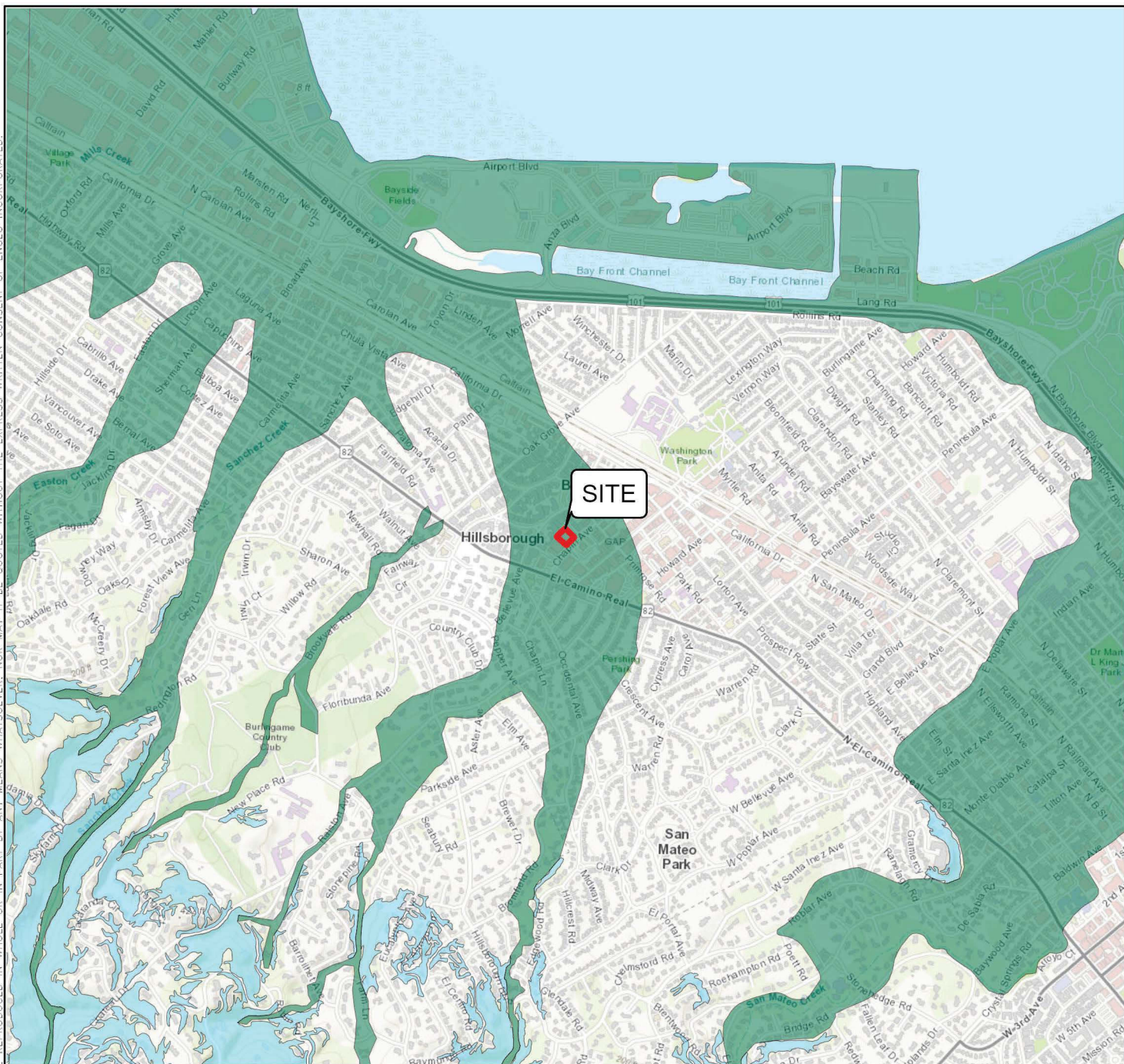
HISTORIC CREEK MAP
1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PROJECT NO.: 18918.000.001
SCALE: AS SHOWN
DRAWN BY: JV CHECKED BY: TPB

FIGURE NO.
4

ORIGINAL FIGURE PRINTED IN COLOR

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

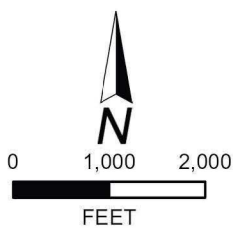
EARTHQUAKE-INDUCED LANDSLIDE ZONES

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



LIQUEFACTION ZONE

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



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



SEISMIC HAZARDS ZONE MAP

1430 CHAPIN AVENUE
BURLINGAME, CALIFORNIA

PROJECT NO. : 18918.000.001

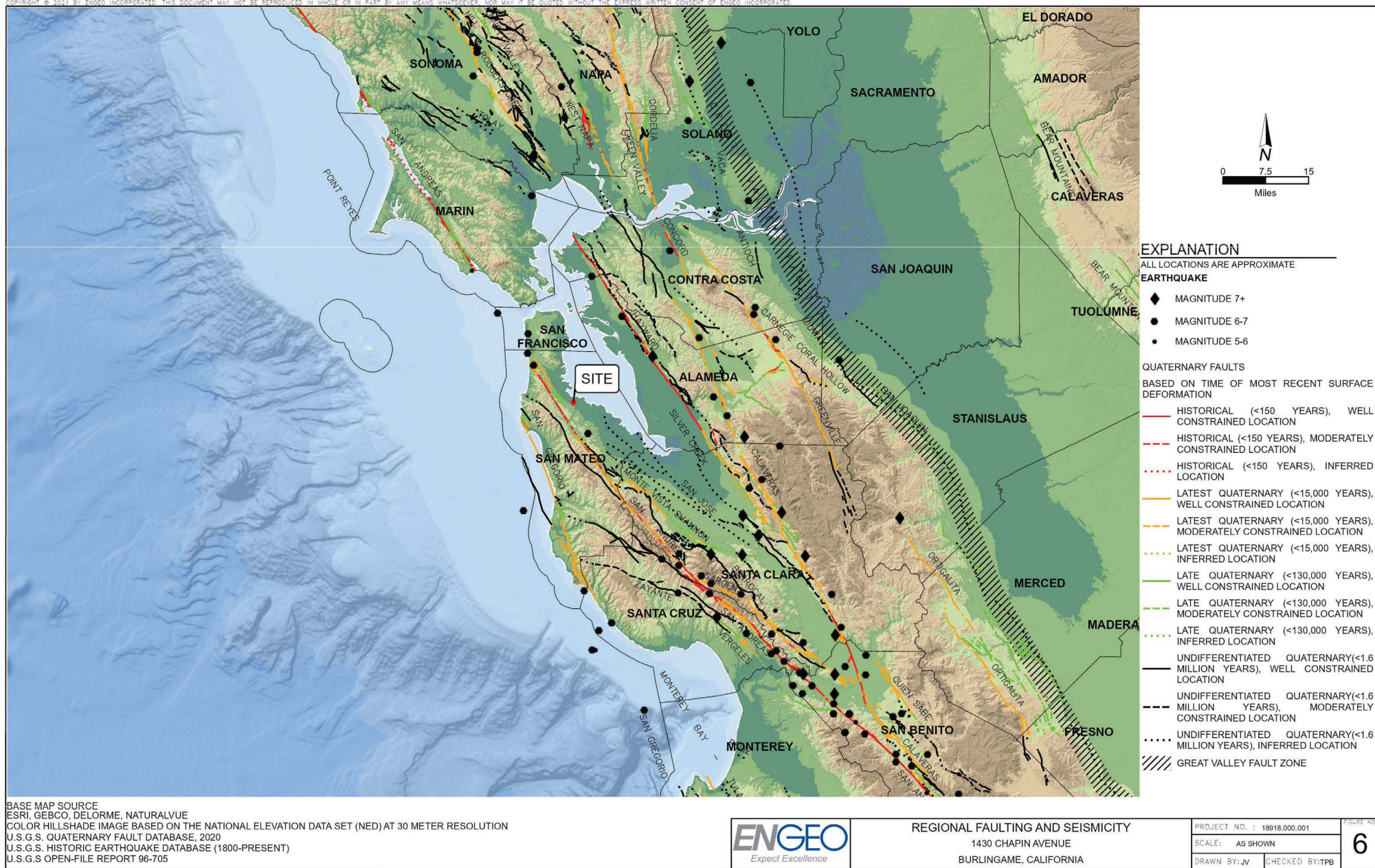
SCALE: AS SHOWN

DRAWN BY: JV

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FIGURE NO.

5





APPENDIX A

CONE PENETRATION TEST REPORT

PRESENTATION OF SITE INVESTIGATION RESULTS

1430 Chapin Avenue

Prepared for:

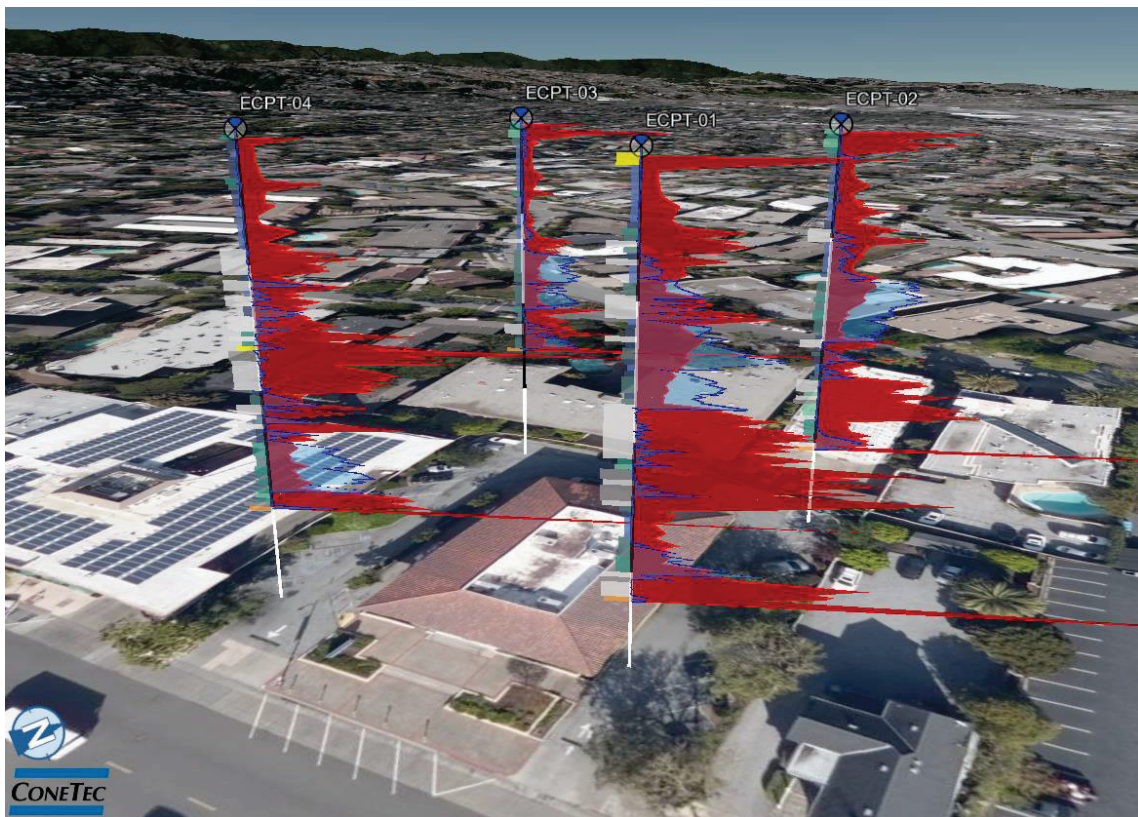
ENGEO Incorporated

ConeTec Job No: 21-56-22652

Project Start Date: 29-Jun-2021

Project End Date: 29-Jun-2021

Report Date: 30-Jun-2021



Prepared by:

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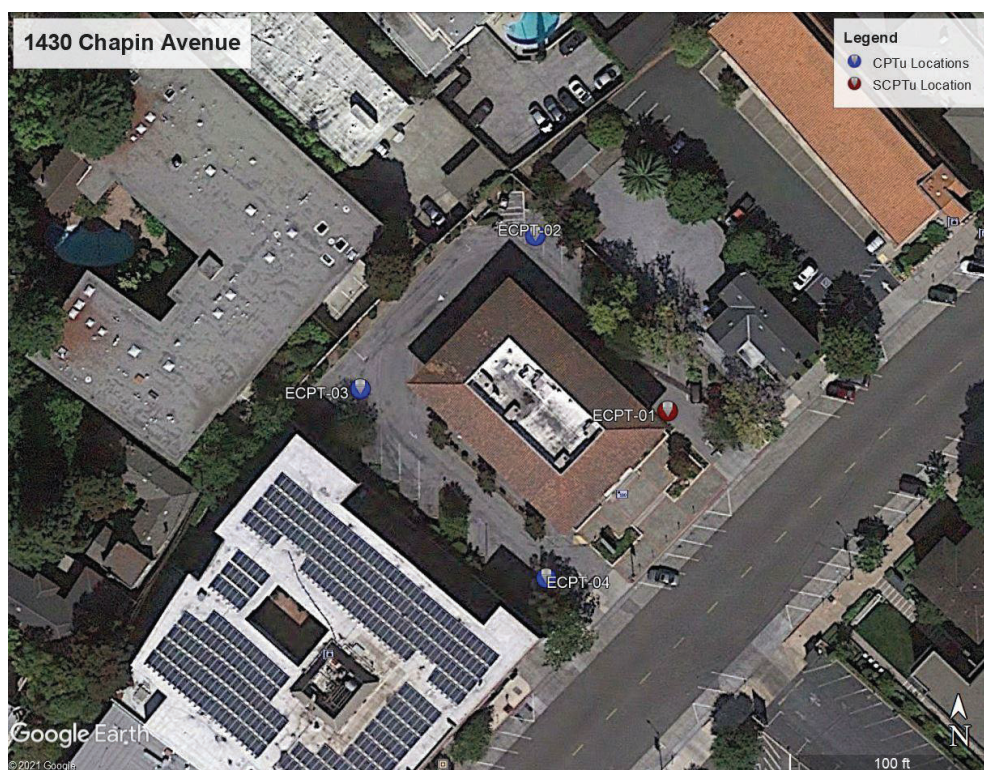
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for ENGEO Incorporated of San Ramon, California. The program consisted of cone penetration testing (CPTu) at four (4) locations. Shear wave velocities were recorded in one (1) sounding. The assumed phreatic surface used for the calculated parameters is based on the shallowest pore pressure dissipation test to reach equilibrium within or nearest each sounding.

Project Information

Project	
Client	ENGEO Incorporated
Project	1430 Chapin Avenue
ConeTec Project #	21-56-22652

An aerial overview from Google Earth including the CPTu test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C15)	30-ton truck mounted cylinder	CPTu/SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu/SCPTu	Consumer grade GPS	32610

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)
448:T1500F15U500	448	15	225	1500	15	500
Cone 448 was used for all soundings.						

Cone Penetration Test	
Depth reference	Depths are referenced to the existing ground surface at the time of test.
Tip and sleeve data offset	0.1 Meter This has been accounted for in the CPT data files.
Additional Comments	Advanced plots, Normalized plots, Seismic plots, as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package.

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

Limitations

This report has been prepared for the exclusive use of ENGEO Incorporated (Client) for the project titled "1430 Chapin Avenue". The report's contents may not be relied upon by any other party without the express written permission of ConeTec, Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

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

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

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm², 10 cm² and 15 cm² 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 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross-sectional area (typically forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

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

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position ([ASTM Type 2](#)). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

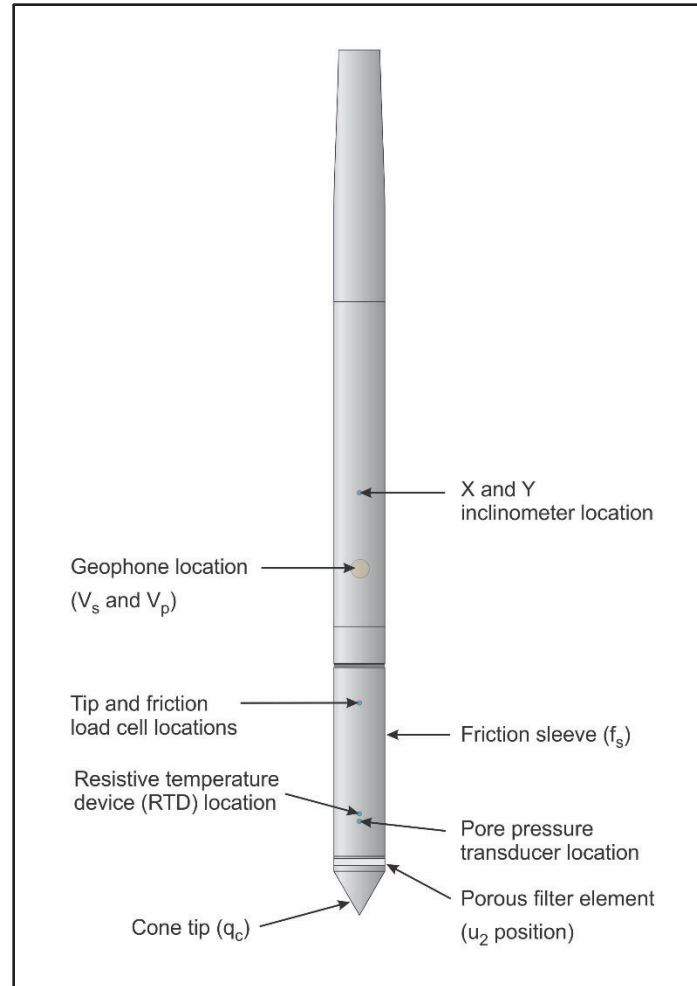


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a sixteen 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 interval is 2.5 centimeters; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

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

All testing is performed in accordance to ConeTec's 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 silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by [Robertson et al. \(1986\)](#) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

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. \(1986\)](#):

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

The friction ratio (R_f) 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 \(2012\)](#).

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

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

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an uphole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in [Figure SCPTu-1](#).

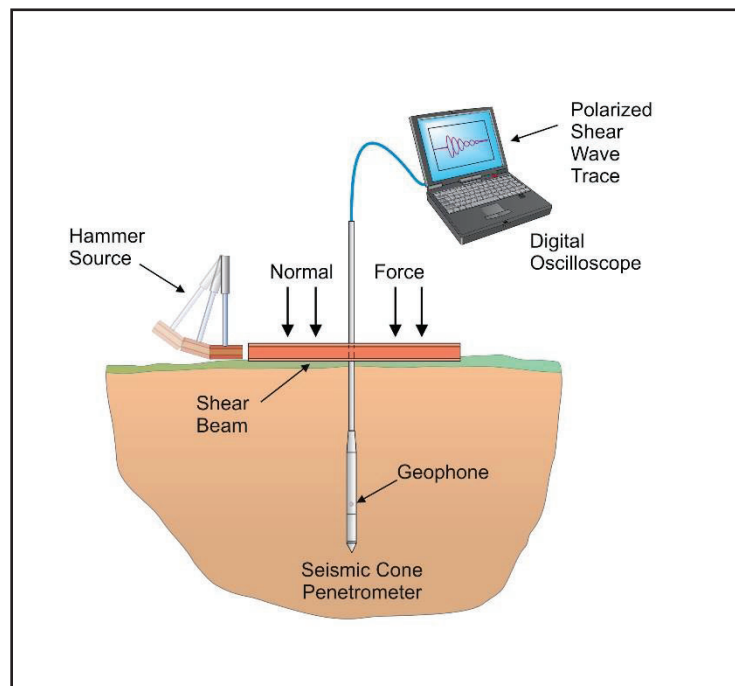


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

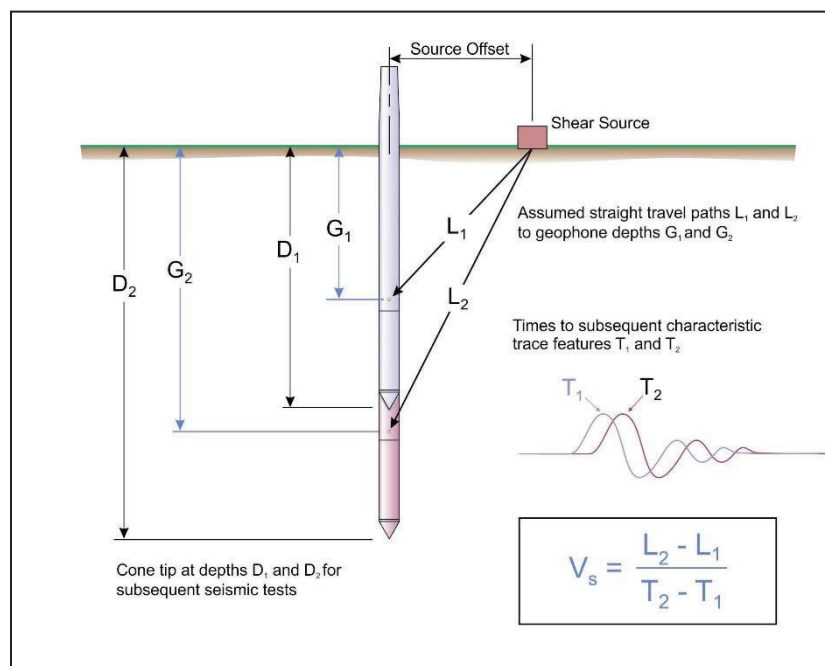


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
 d_i = the thickness of any layer between 0 and 100 ft (30 m)
 v_{si} = the shear wave velocity in ft/s (m/s)
 $\sum_{i=1}^n d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

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

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

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

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

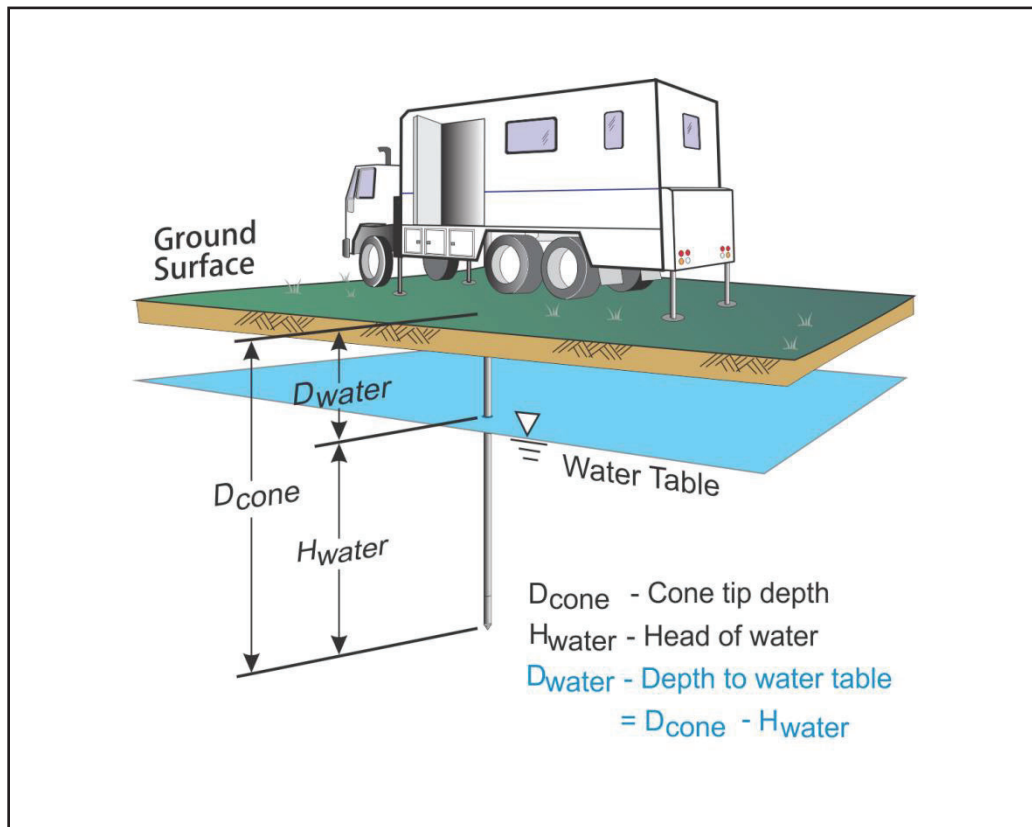


Figure PPD-1. Pore pressure dissipation test setup

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

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

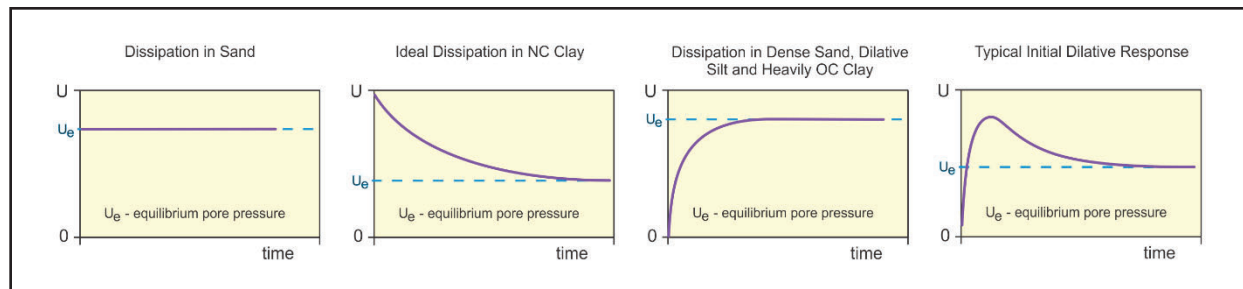


Figure PPD-2. Pore pressure dissipation curve examples

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

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.

REFERENCES

- American Society of Civil Engineers (ASCE), 2010, "Minimum Design Loads for Buildings and Other Structures", Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: [10.1061/9780784412916](https://doi.org/10.1061/9780784412916).
- ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-12](https://doi.org/10.1520/D5778-12).
- ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: [10.1520/D7400_D7400M-19](https://doi.org/10.1520/D7400_D7400M-19).
- Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: [1063-1073/T98-062](https://doi.org/10.1063-1073/T98-062).
- Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.
- Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.
- Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.
- Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: [10.1061/9780784412770.027](https://doi.org/10.1061/9780784412770.027).
- Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.
- Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.
- Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
- Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: [10.1061/\(ASCE\)0733-9410\(1986\)112:8\(791\)](https://doi.org/10.1061/(ASCE)0733-9410(1986)112:8(791)).
- Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: [10.1139/T90-014](https://doi.org/10.1139/T90-014).
- Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 539-550. DOI: [10.1139/T92-061](https://doi.org/10.1139/T92-061).

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: [10.1139/T09-065](https://doi.org/10.1139/T09-065).

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. DOI: [10.1139/T98-105](https://doi.org/10.1139/T98-105).

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: [10.1680/geot.1991.41.1.17](https://doi.org/10.1680/geot.1991.41.1.17).

The following appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Normalized Cone Penetration Test Plots
- SBT Zone Scatter Plots
- Seismic Cone Penetration Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Wave Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 21-56-22652
Client: ENGEO Incorporated
Project: 1430 Chapin Avenue
Start Date: 29-Jun-2021
End Date: 29-Jun-2002

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (ft)	Final Depth (ft)	Northing ² (m)	Easting ² (m)	Elevation ³ (ft)	Refer to Notation Number
ECPT-01	21-56-22652_SP01	29-Jun-2021	448:T1500F15U500	14.3	52.99	4159217	557361	36	
ECPT-02	21-56-22652_CP02	29-Jun-2021	448:T1500F15U500	14.3	50.28	4159243	557341	37	
ECPT-03	21-56-22652_CP03	29-Jun-2021	448:T1500F15U500	14.3	41.83	4159220	557315	37	4
ECPT-04	21-56-22652_CP04	29-Jun-2021	448:T1500F15U500	14.5	49.38	4159192	557343	37	

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



ENGEO

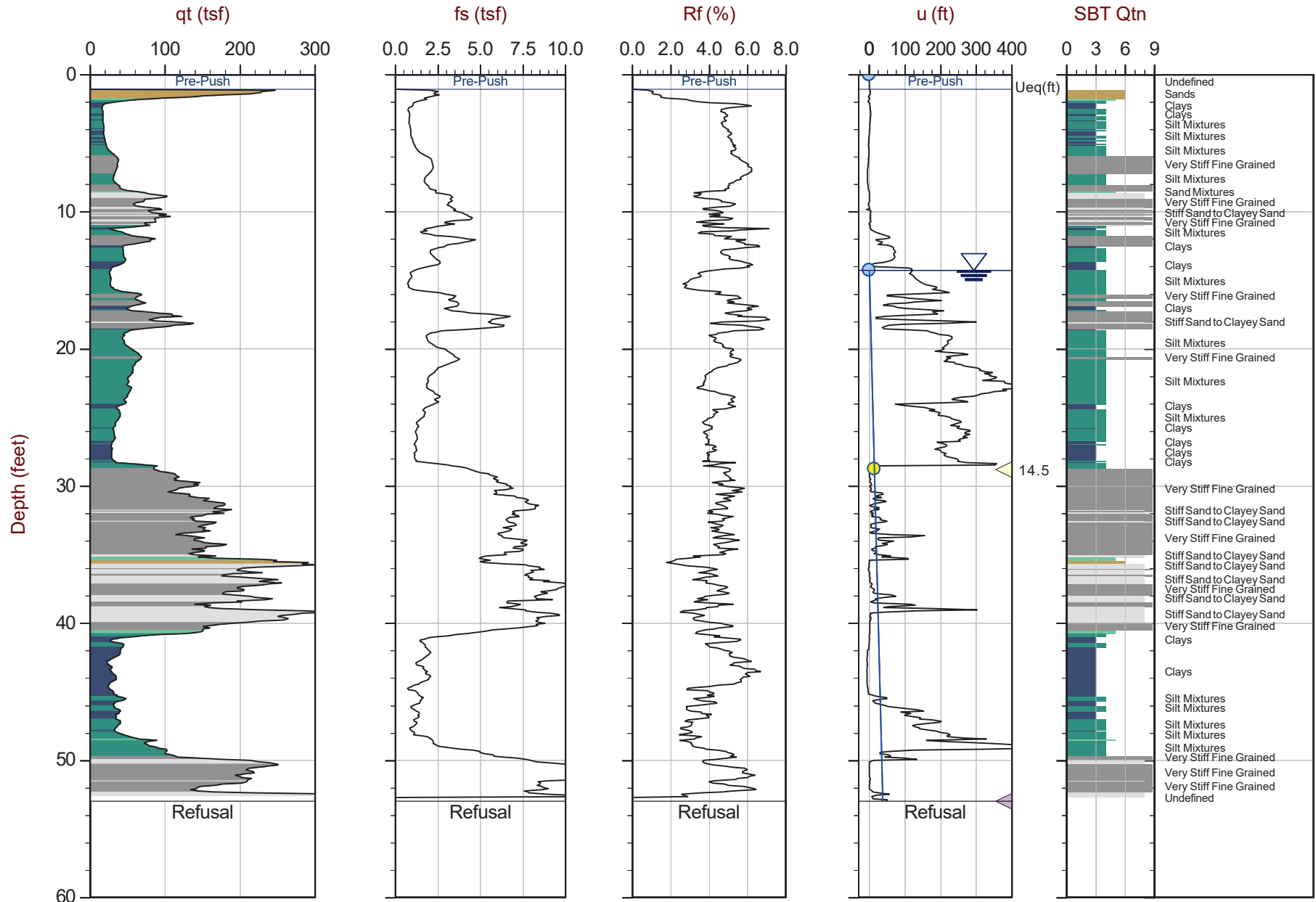
Job No: 21-56-22652

Date: 2021-06-29 07:25

Site: 1430 Chapin Avenue

Sounding: ECPT-01

Cone: 448:T1500F15U500



Max Depth: 16.150 m / 52.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_SP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159217m E: 557361m

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

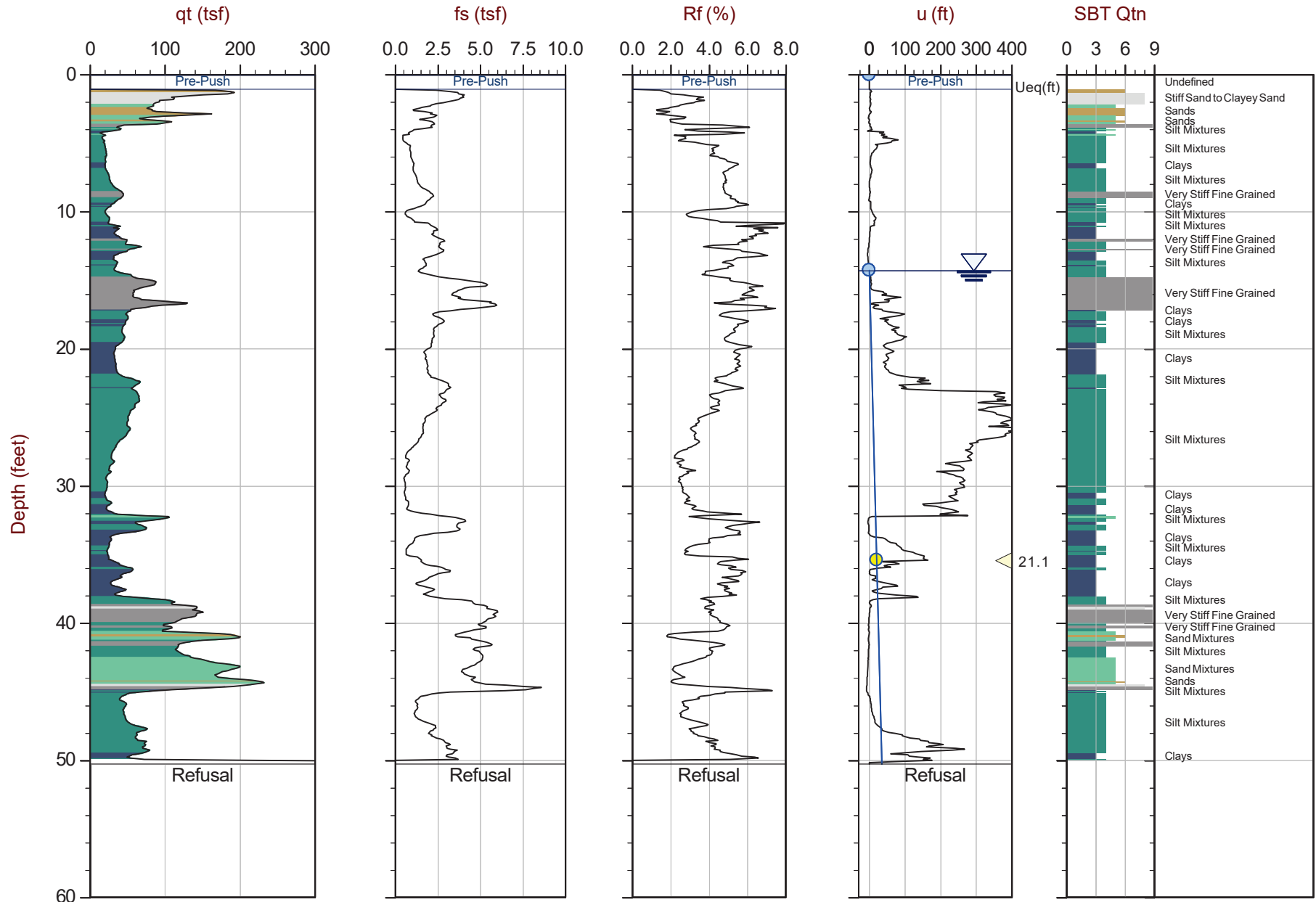
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



ENGEO

Job No: 21-56-22652
Date: 2021-06-29 08:25
Site: 1430 Chapin Avenue

Sounding: ECPT-02
Cone: 448:T1500F15U500



Max Depth: 15.325 m / 50.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_CP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159243m E: 557341m

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

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

SBT: Robertson, 2009 and 2010
 Coords: UTM 10S N: 4159220m E: 557315m

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

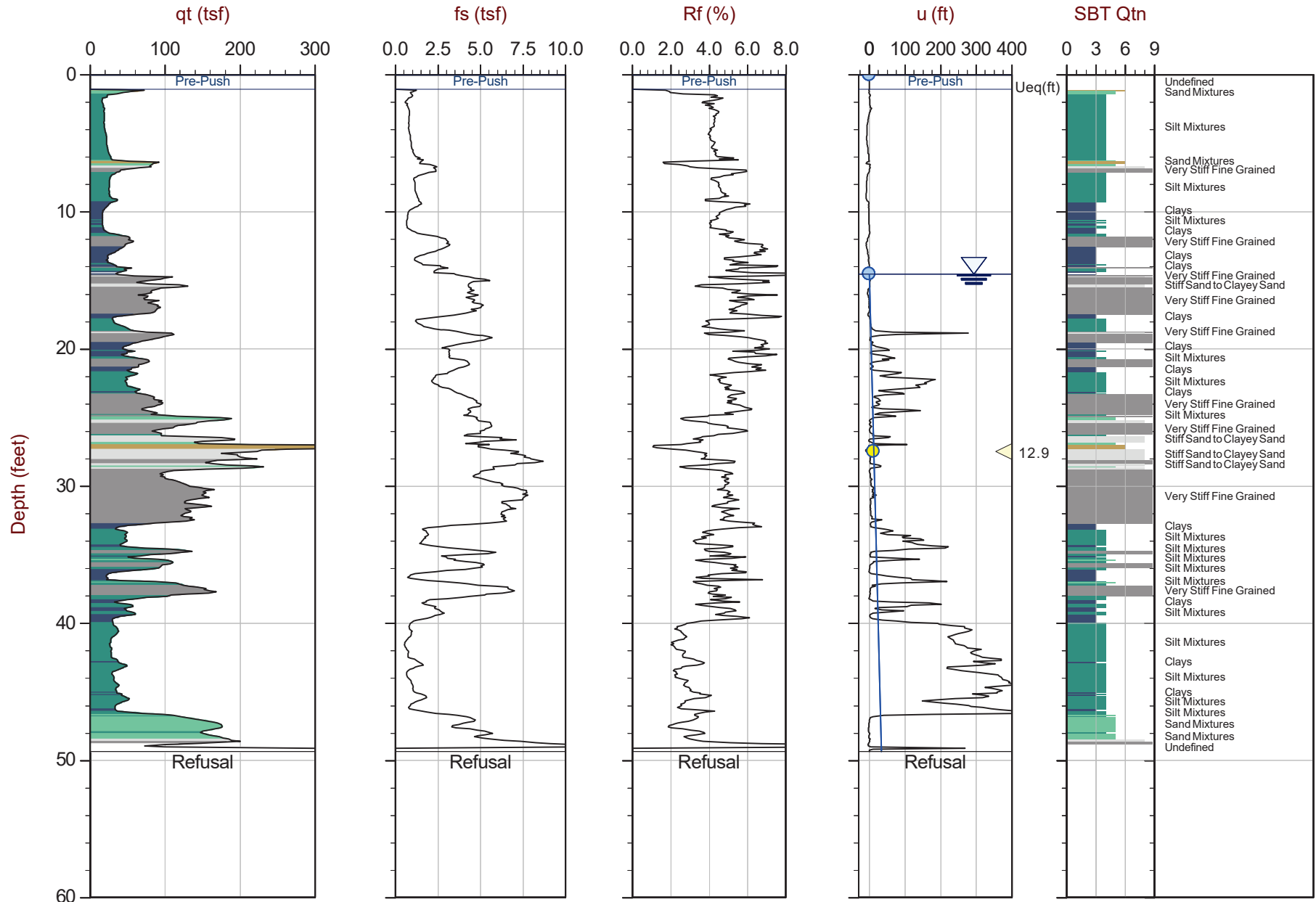
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



ENGEO

Job No: 21-56-22652
Date: 2021-06-29 10:05
Site: 1430 Chapin Avenue

Sounding: ECPT-04
Cone: 448:T1500F15U500



Max Depth: 15.050 m / 49.38 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

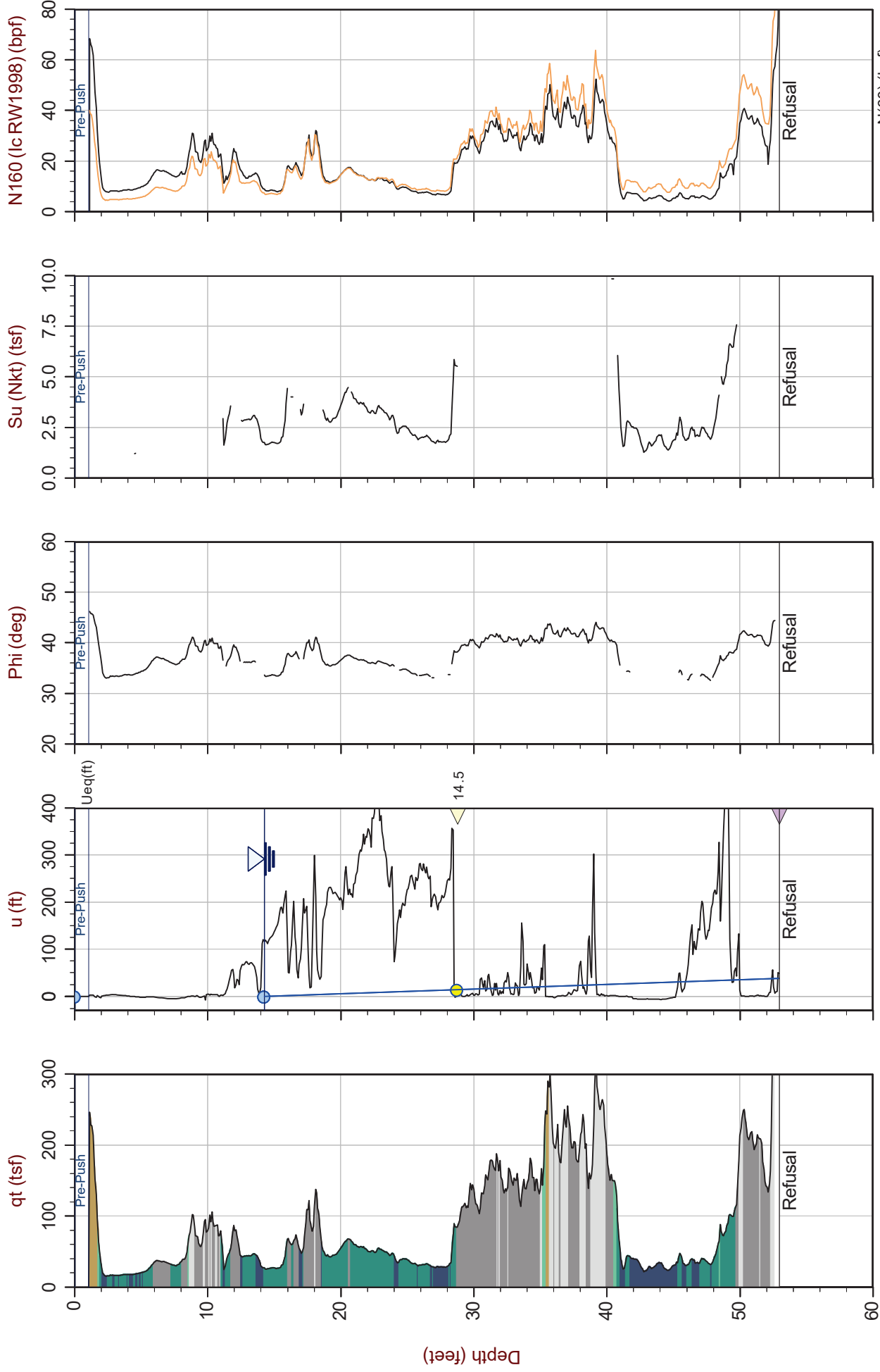
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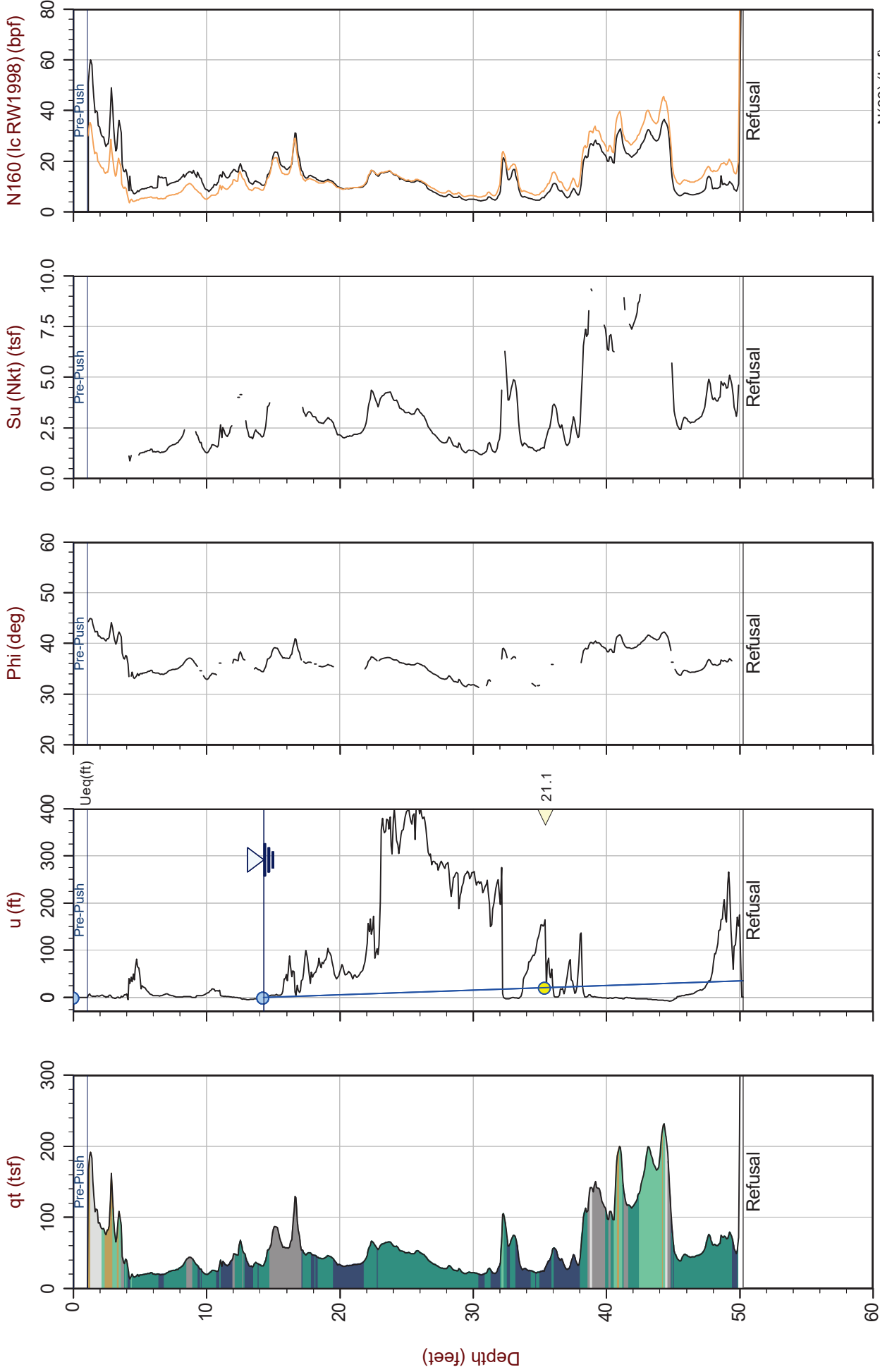
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159192m E: 557343m

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

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

Advanced Cone Penetration Test Plots



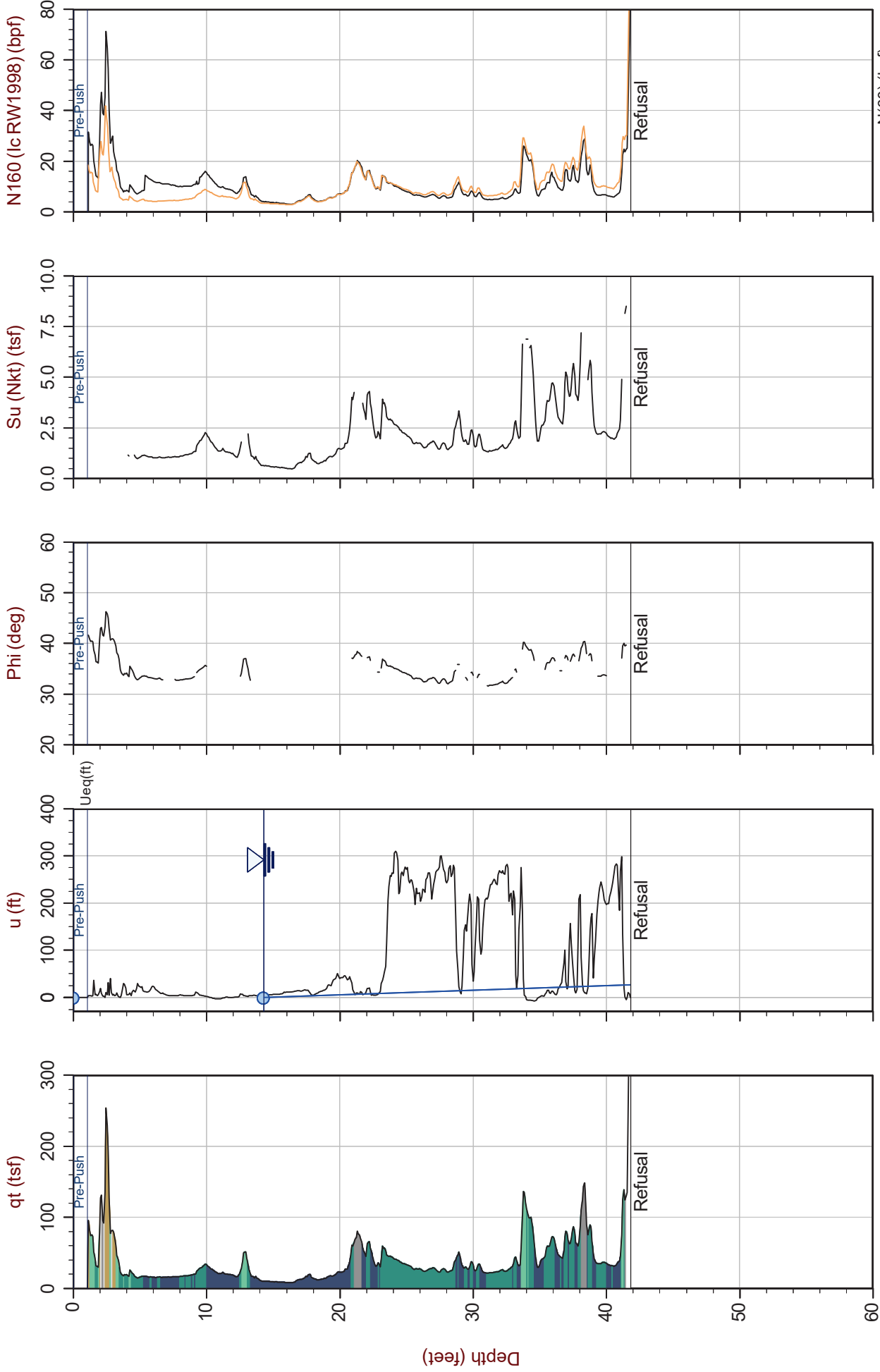


Max Depth: 15.325 m / 50.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159243m E: 557341m

Disipation, Ueq not achieved
The coordinates should not be used for design purposes.



Max Depth: 12.750 m / 41.83 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159220m E: 557315m

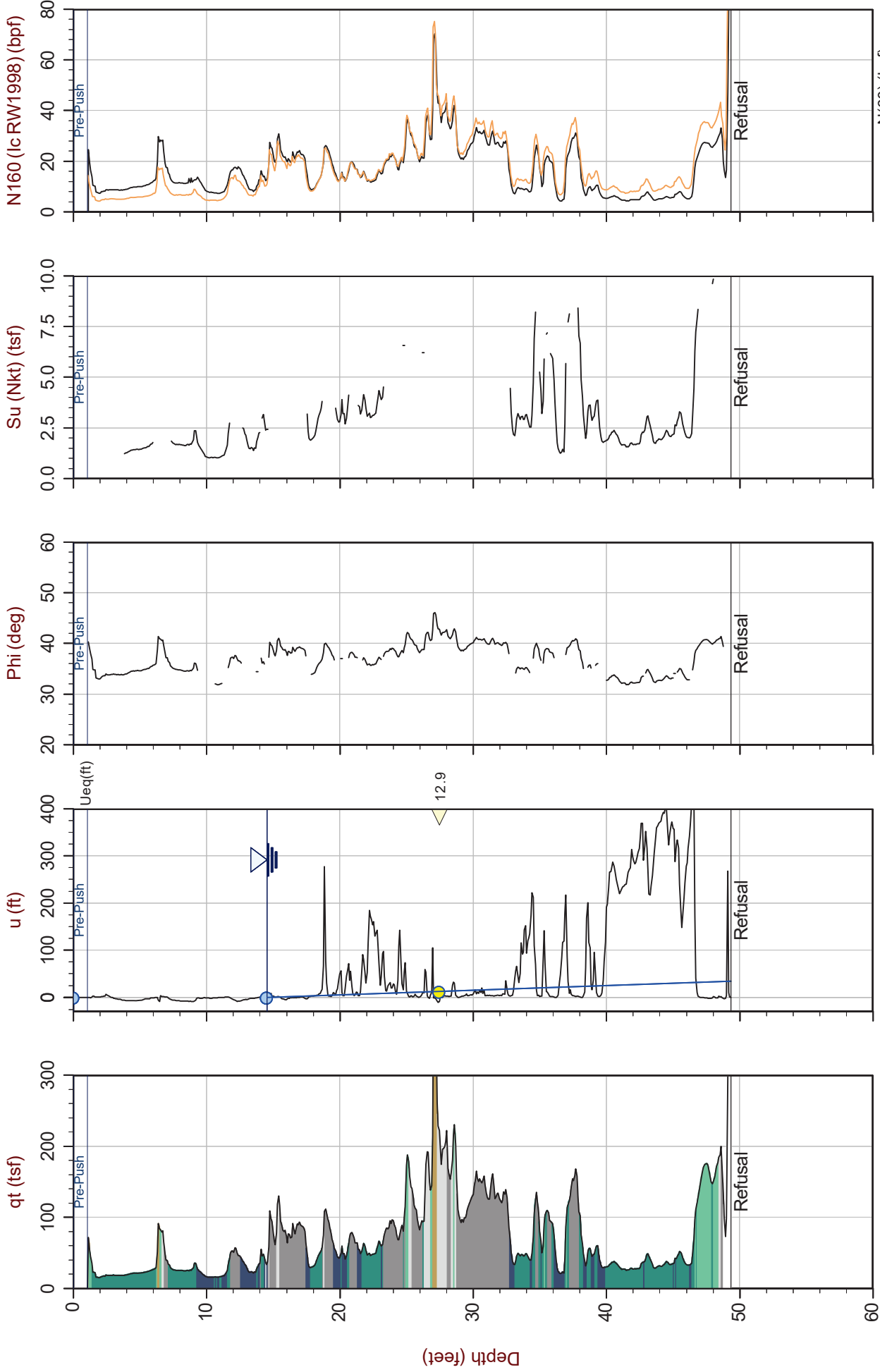
 **Equilibrium Pore Pressure (Ueq)**
 The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

 **Assumed Ueq**

 **Dissipation, Ueq achieved**

 **Dissipation, Ueq not achieved**

 **Hydrostatic Line**



Max Depth: 15.050 m / 49.38 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: Every Point
 File: 21-56-22652_CP04.COR
 Unit Wt: SBTQtn (PKR2009)
 Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
 Coords: UTM 10S N: 4159192m E: 557343m
 The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

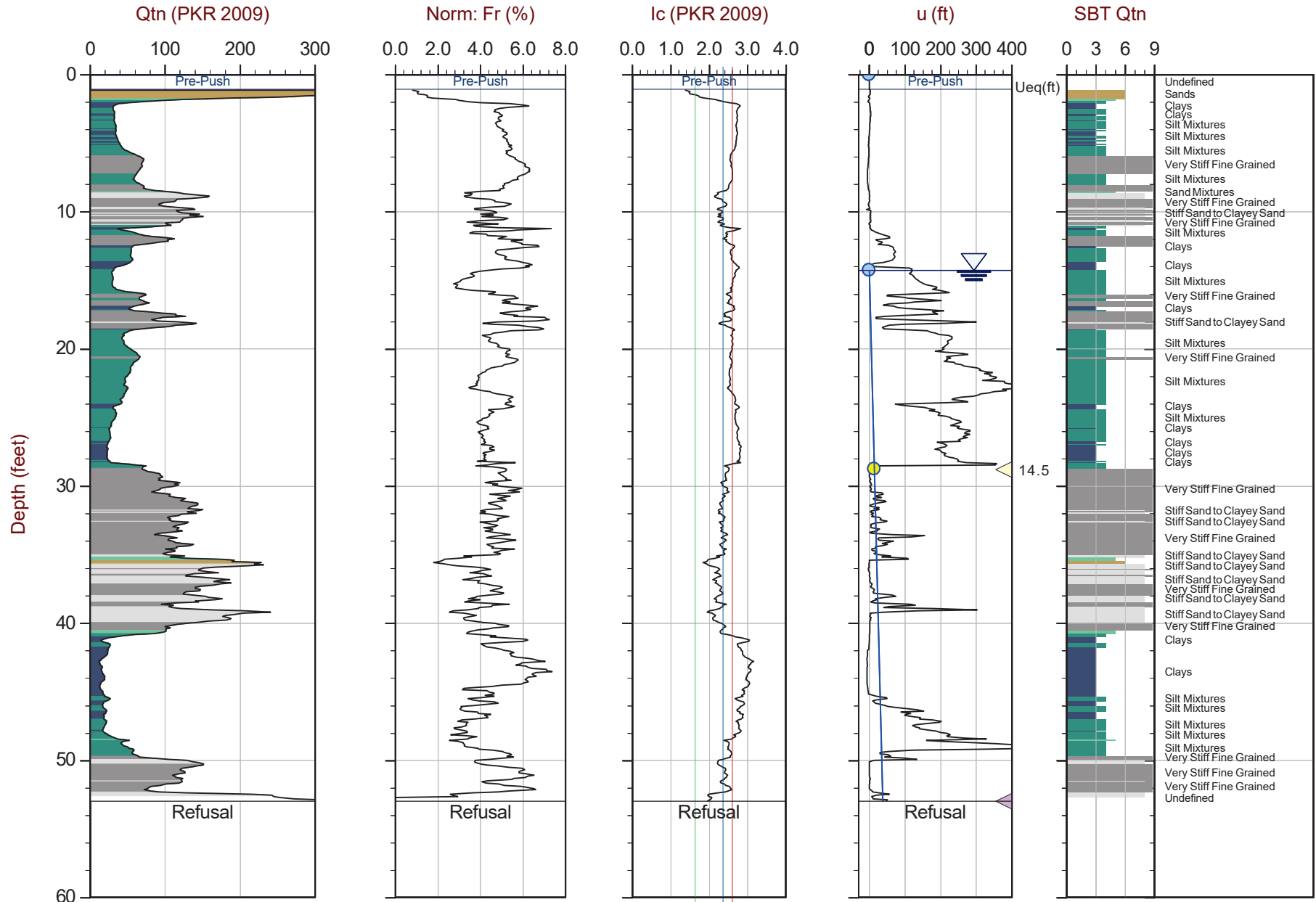
Normalized Cone Penetration Test Plots



ENGEO

Job No: 21-56-22652
Date: 2021-06-29 07:25
Site: 1430 Chapin Avenue

Sounding: ECPT-01
Cone: 448:T1500F15U500



Max Depth: 16.150 m / 52.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_SP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159217m E: 557361m

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

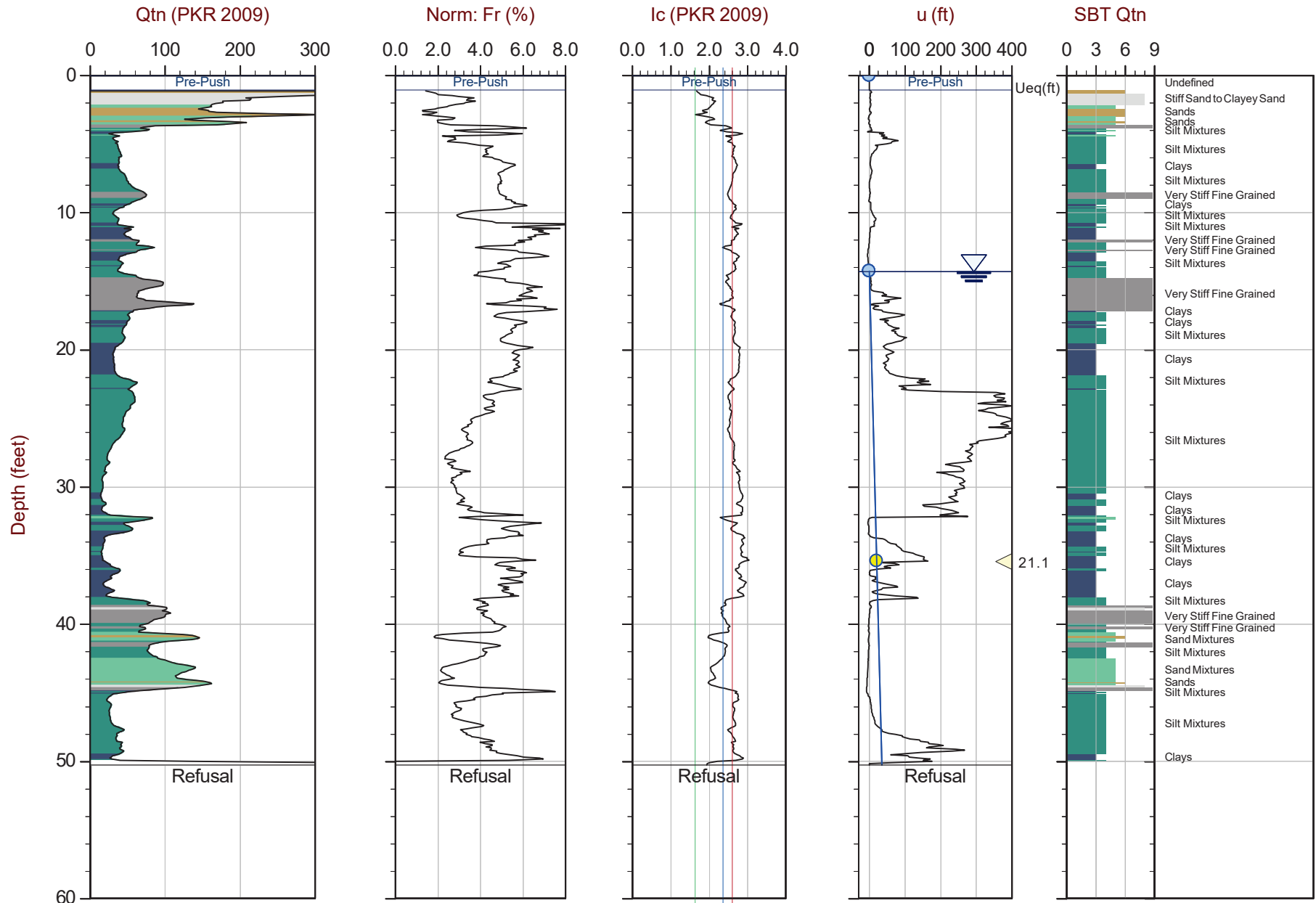
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ENGEO

Job No: 21-56-22652
Date: 2021-06-29 08:25
Site: 1430 Chapin Avenue

Sounding: ECPT-02
Cone: 448:T1500F15U500



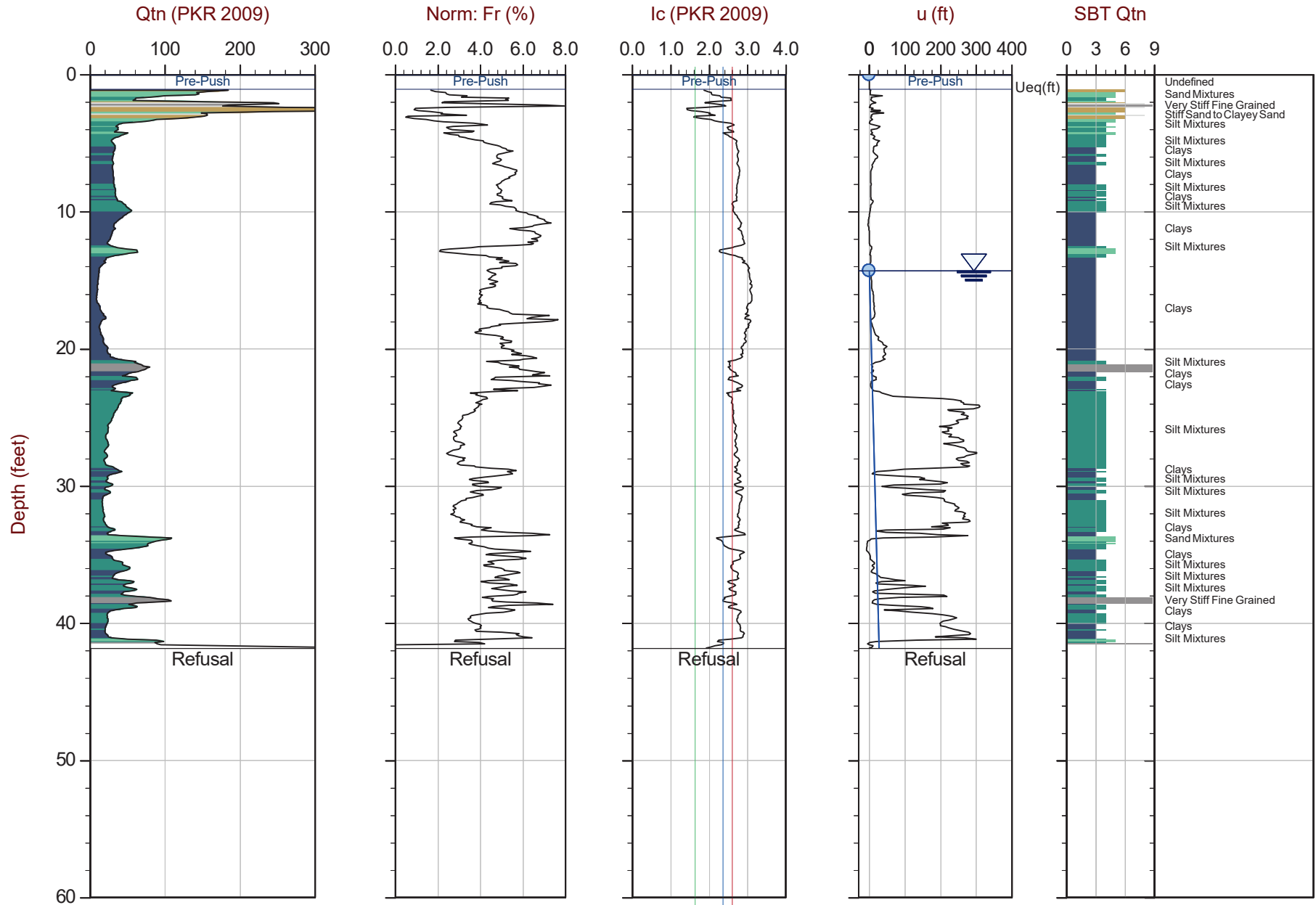
Max Depth: 15.325 m / 50.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_CP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159243m E: 557341m

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

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



Max Depth: 12.750 m / 41.83 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_CP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159220m E: 557315m

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

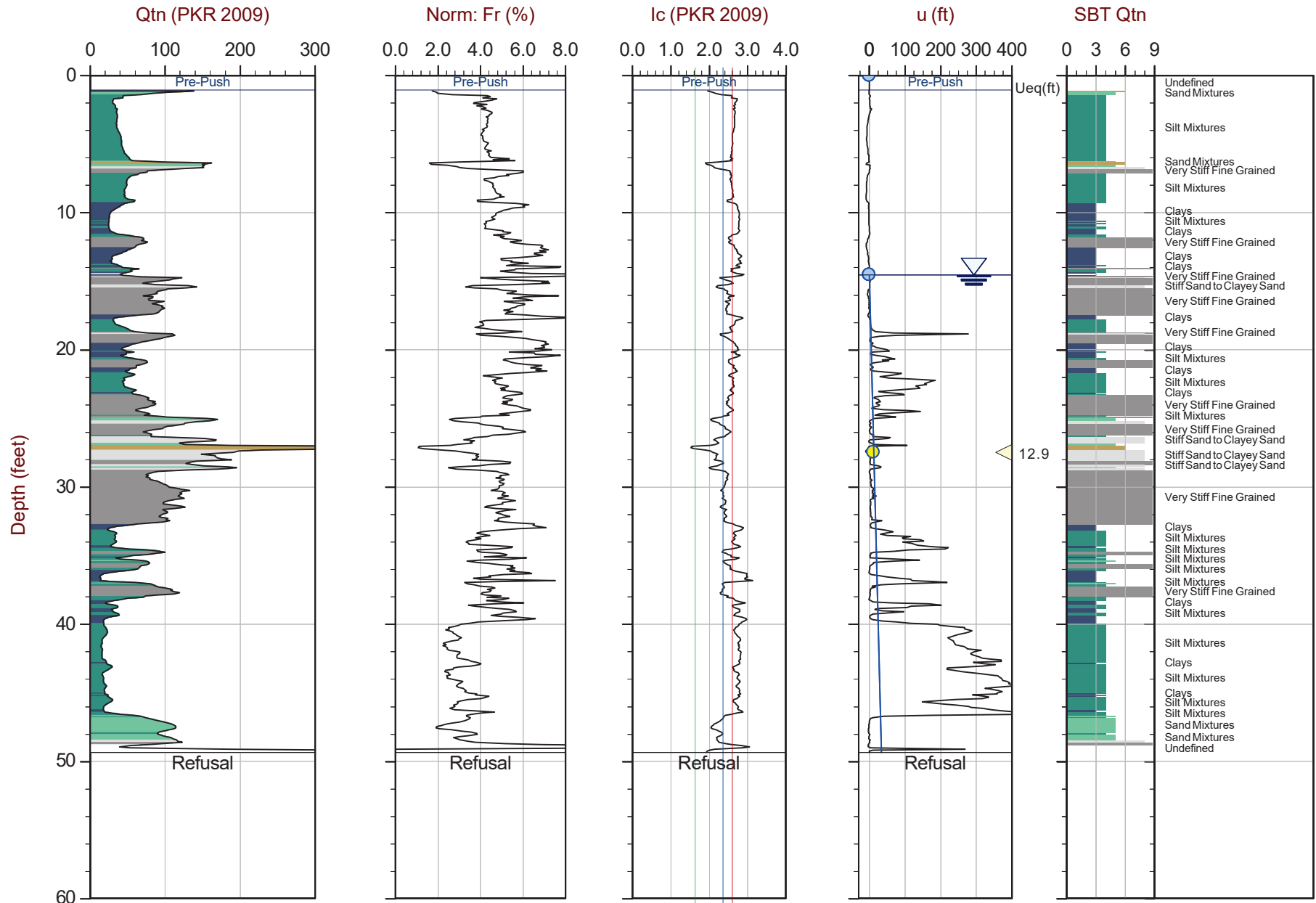
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ENGEO

Job No: 21-56-22652
Date: 2021-06-29 10:05
Site: 1430 Chapin Avenue

Sounding: ECPT-04
Cone: 448:T1500F15U500



Max Depth: 15.050 m / 49.38 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

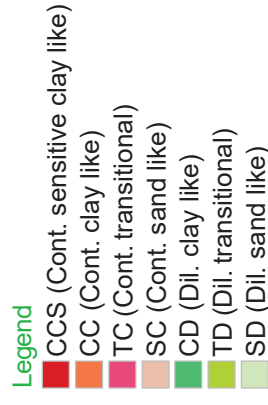
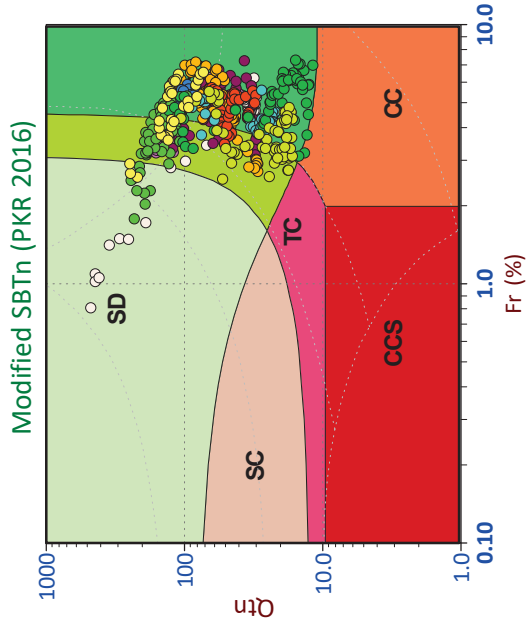
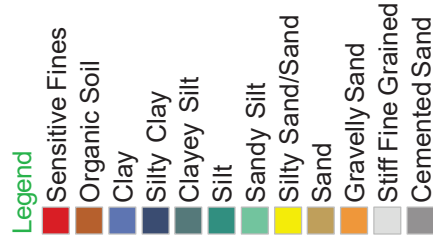
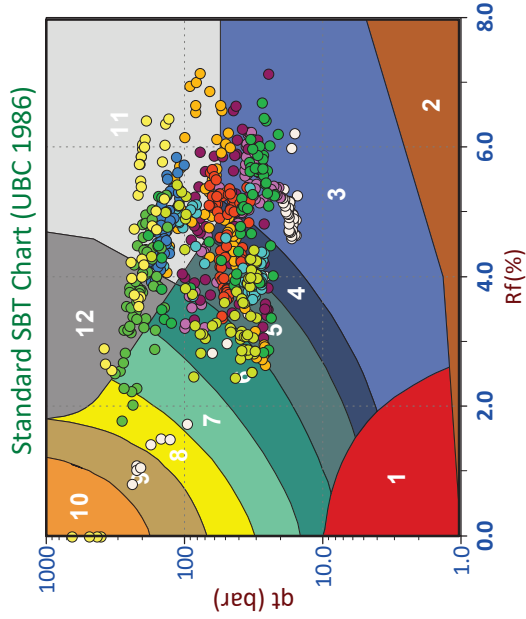
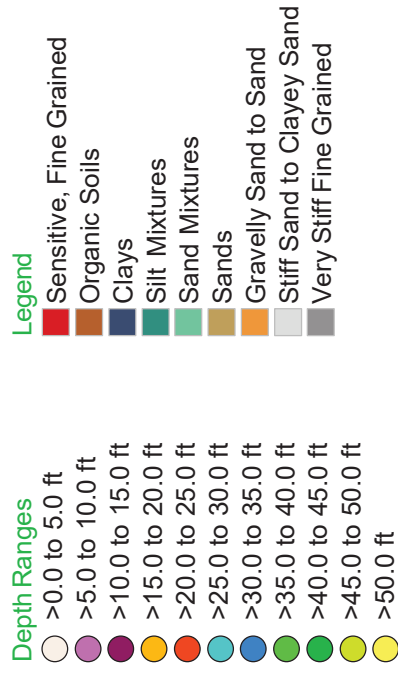
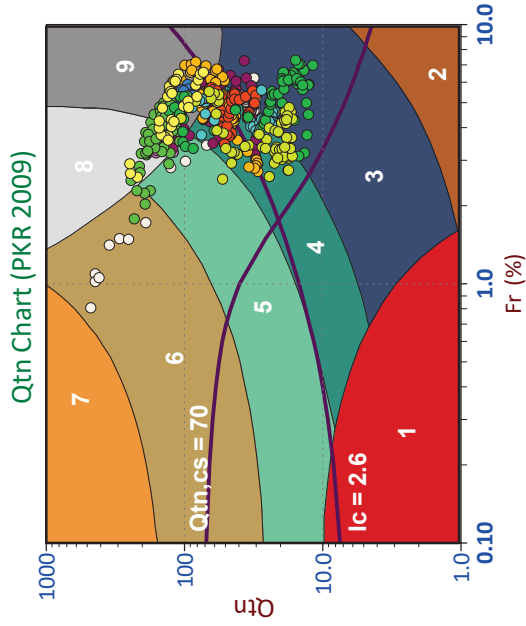
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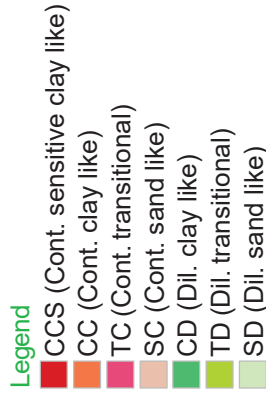
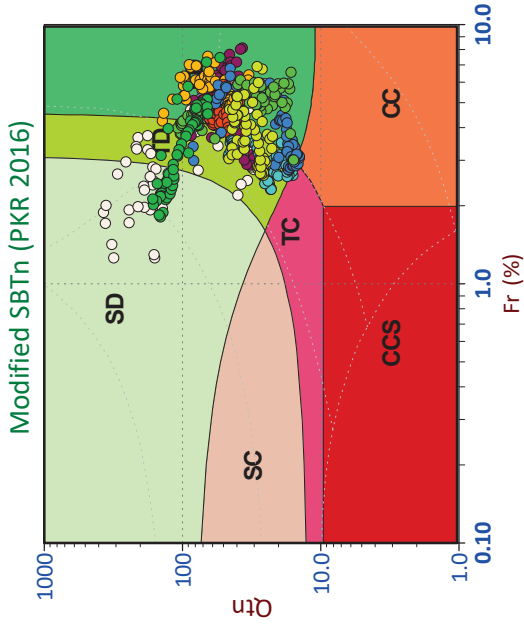
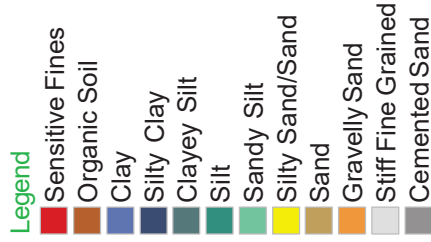
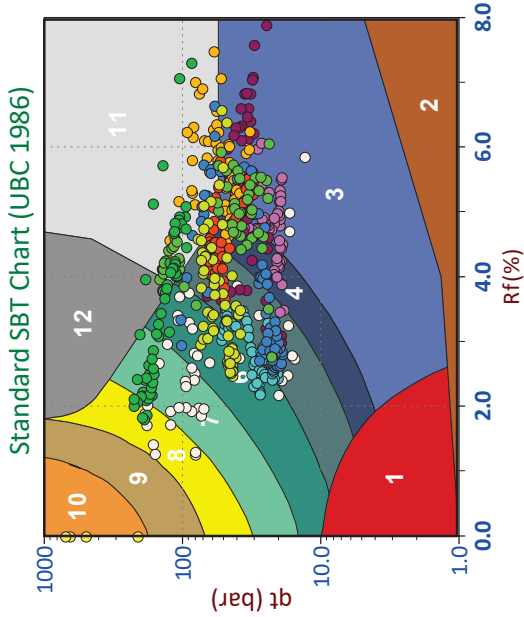
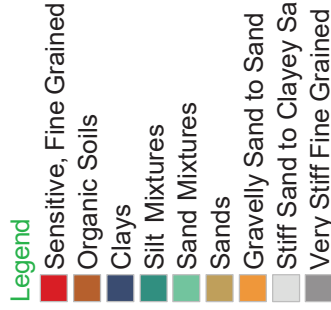
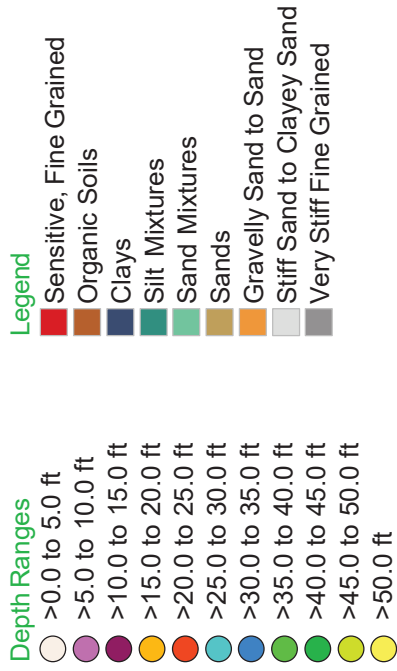
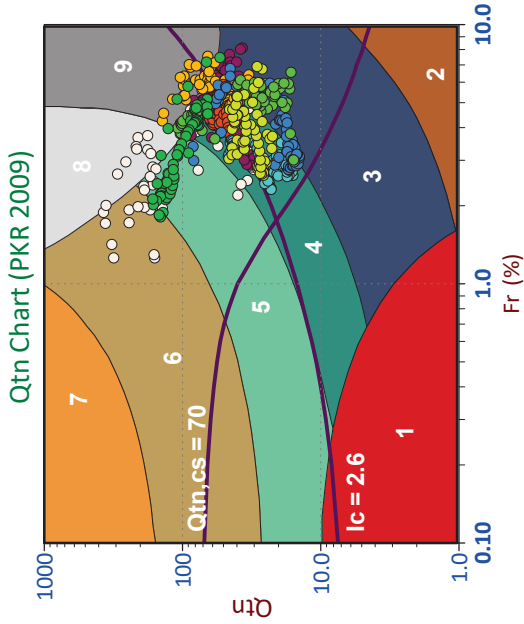
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159192m E: 557343m

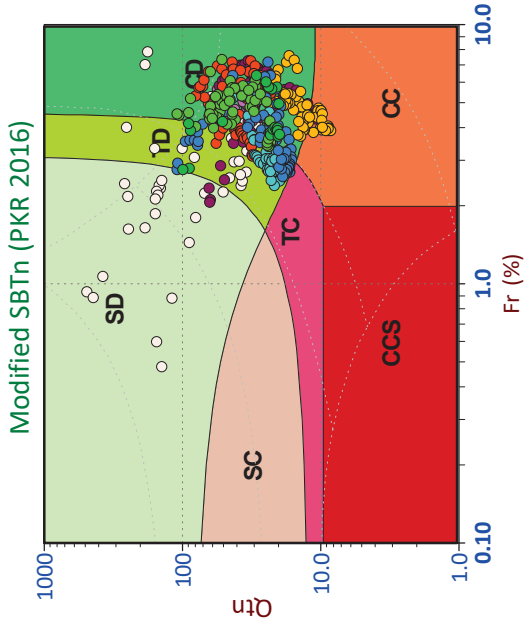
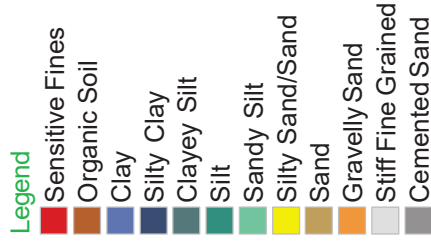
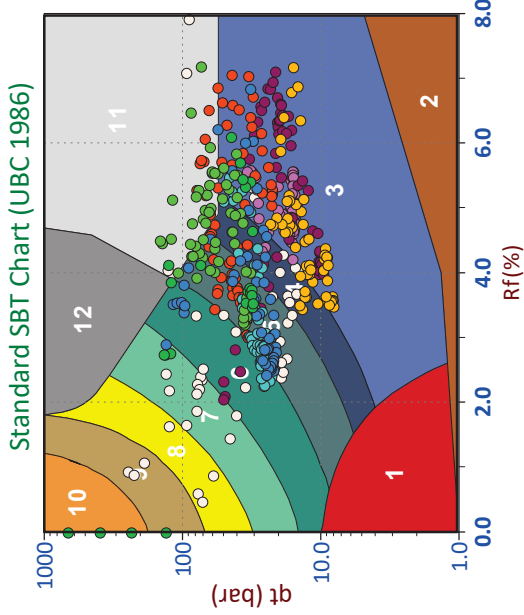
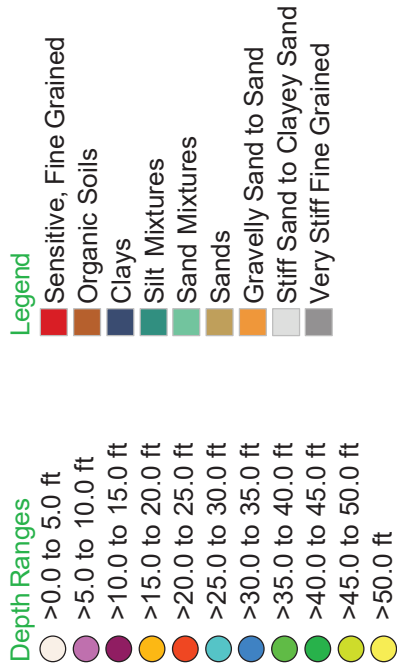
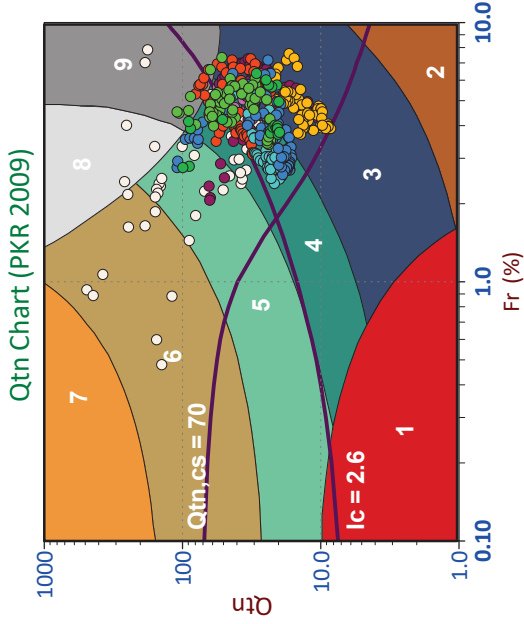
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

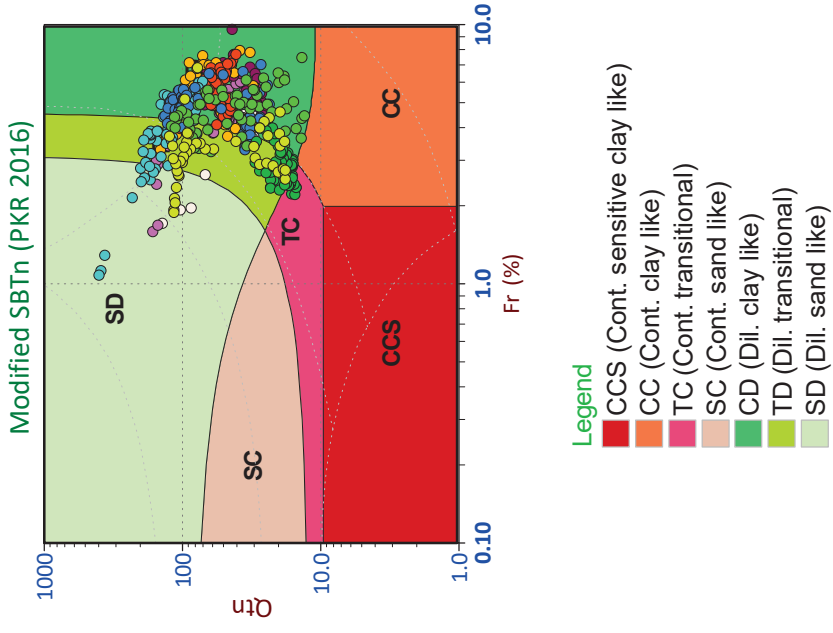
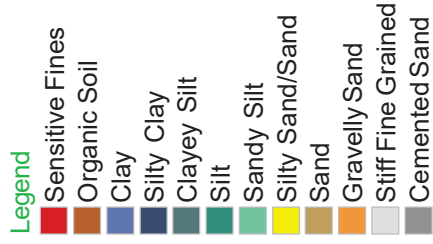
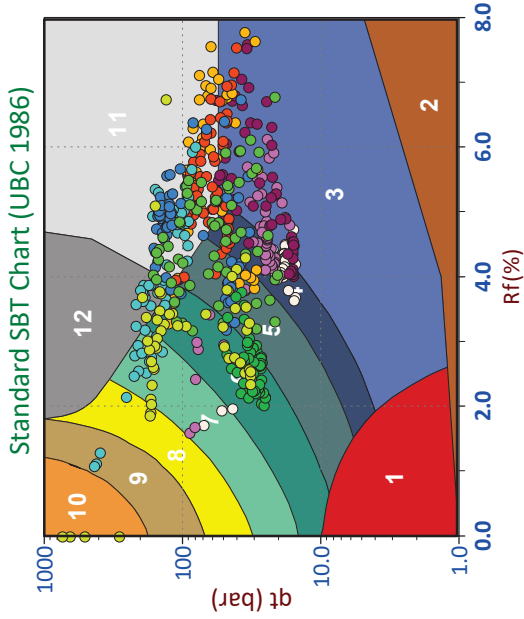
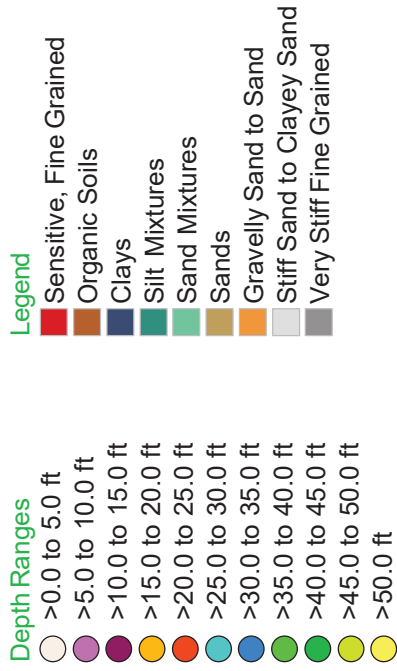
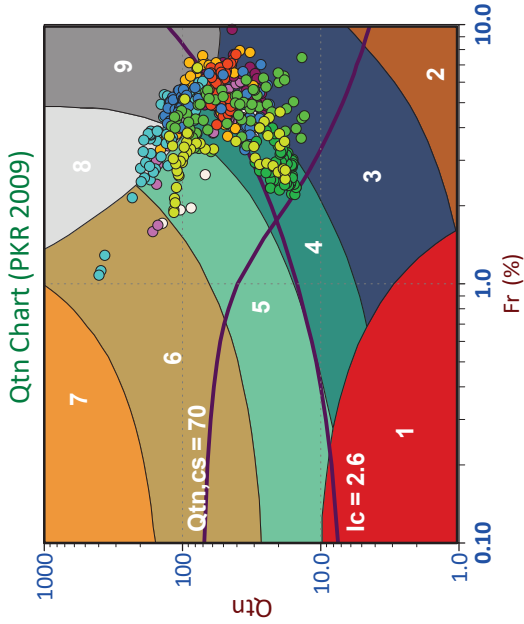
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots









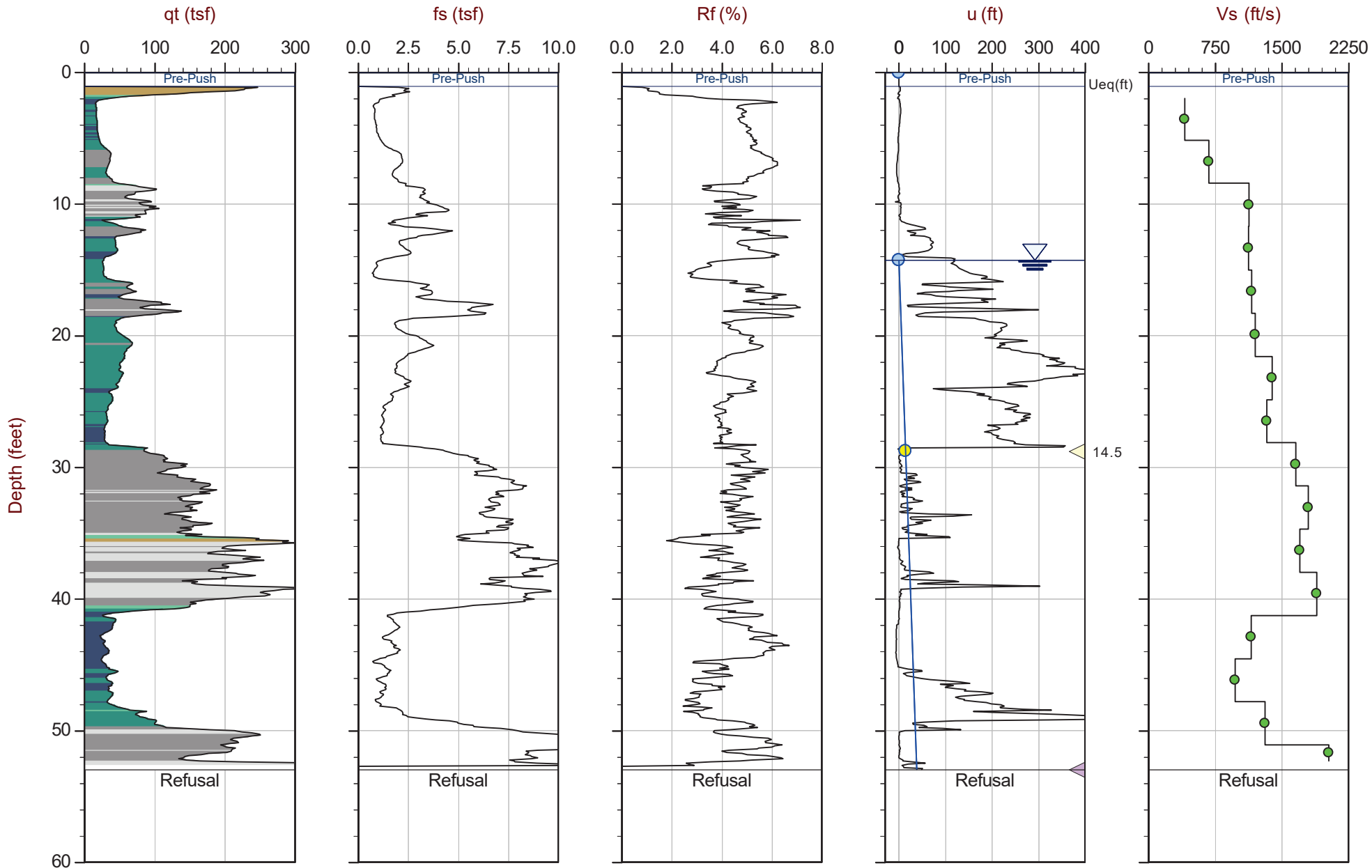
Seismic Cone Penetration Test Plots



ENGEO

Job No: 21-56-22652
Date: 2021-06-29 07:25
Site: 1430 Chapin Avenue

Sounding: ECPT-01
Cone: 448:T1500F15U500



Max Depth: 16.150 m / 52.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22652_SP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4159217m E: 557361m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results



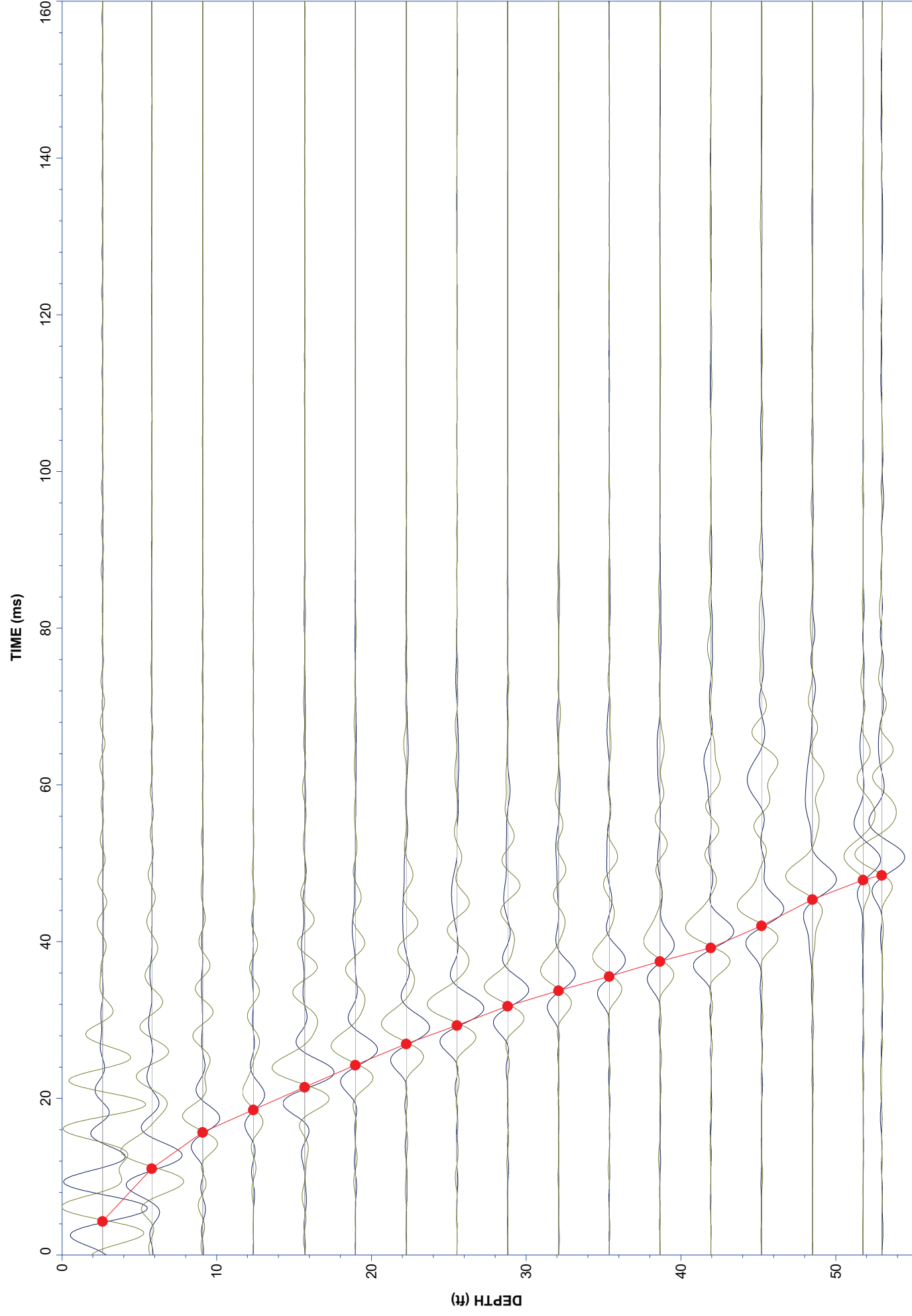
Job No: 21-56-22652
Client: ENGEO
Project: 1430 Chapin Avenue
Sounding ID: ECPT-01
Date: 06:29:21 07:25

Seismic Source: Beam
Seismic Offset (ft): 1.87
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.62	1.97	2.71			
5.81	5.15	5.48	2.77	6.75	410
9.09	8.43	8.64	3.16	4.64	680
12.37	11.71	11.86	3.22	2.85	1131
15.68	15.03	15.14	3.28	2.91	1128
18.96	18.31	18.40	3.26	2.80	1164
22.24	21.59	21.67	3.27	2.72	1201
25.53	24.87	24.94	3.27	2.35	1392
28.81	28.15	28.21	3.27	2.45	1336
32.09	31.43	31.49	3.28	1.97	1662
35.37	34.71	34.76	3.28	1.82	1800
38.65	37.99	38.04	3.28	1.92	1707
41.93	41.27	41.32	3.28	1.73	1894
45.21	44.55	44.59	3.28	2.83	1159
48.49	47.83	47.87	3.28	3.35	978
51.77	51.12	51.15	3.28	2.50	1314
52.99	52.33	52.36	1.21	0.60	2031

Seismic Cone Penetration Test Shear Wave (V_s) Traces



Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 21-56-22652
Client: ENGEO Incorporated
Project: 1430 Chapin Avenue
Start Date: 29-Jun-2021
End Date: 29-Jun-2002

CPT_u PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U _{eq} (ft)	Calculated Phreatic Surface (ft)
ECPT-01	21-56-22652_SP01	15	360	28.79	14.5	14.3
ECPT-01	21-56-22652_SP01	15	300	52.98	Not Achieved	
ECPT-02	21-56-22652_CP02	15	780	35.43	21.1	14.3
ECPT-04	21-56-22652_CP04	15	420	27.48	12.9	14.5



ENGEO

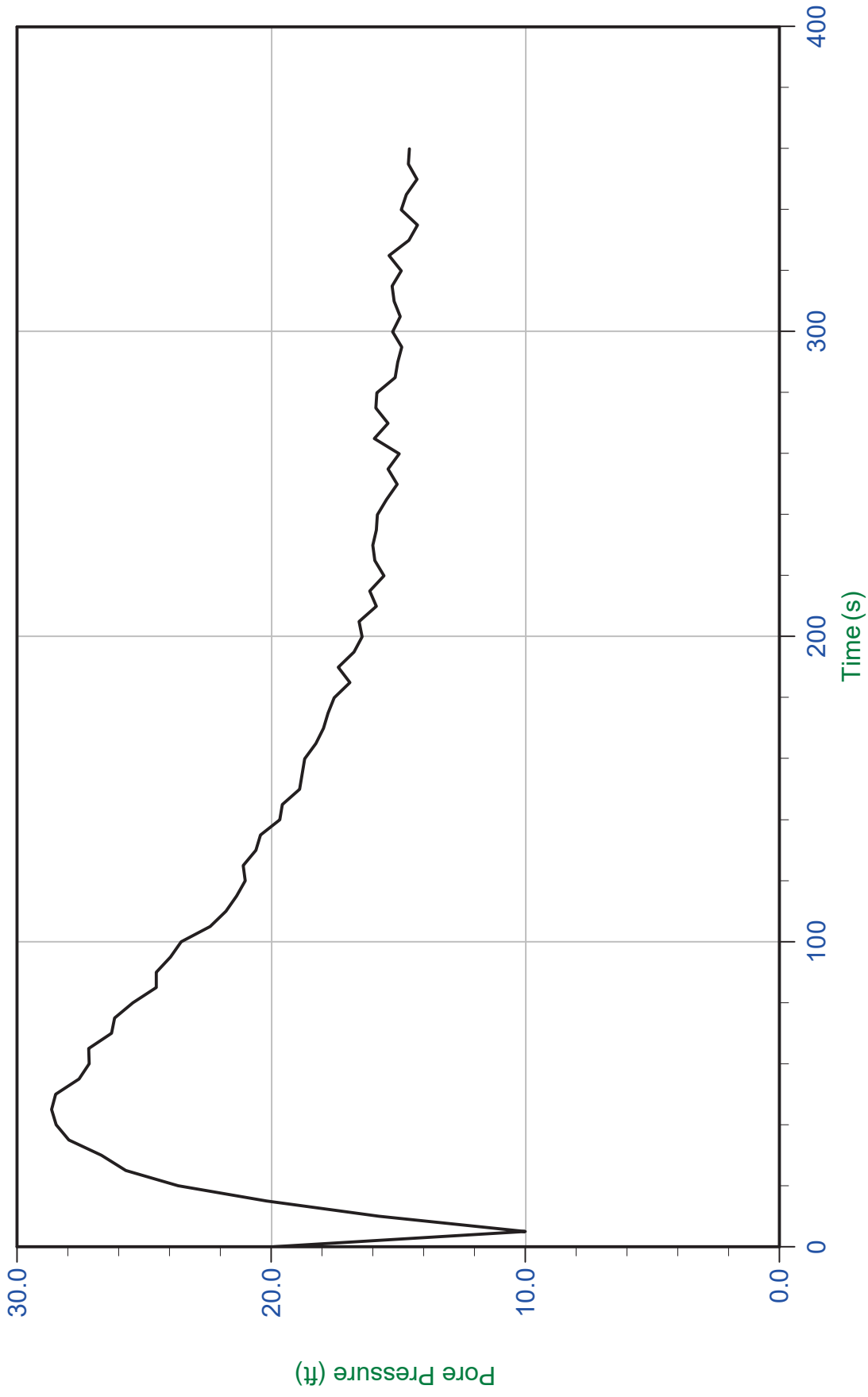
Job No: 21-56-22652

Date: 06/29/2021 07:25

Site: 1430 Chapin Avenue

Sounding: ECPT-01

Cone: 448:T1500F15U500 Area=15 cm²



Filename: 21-56-22652_SP01.PPF

Depth: 8.775 m / 28.789 ft

Duration: 360.0 s

WT: 4.349 m / 14.267 ft

Ueq: 14.5 ft

u Min: 10.0 ft

u Max: 28.7 ft

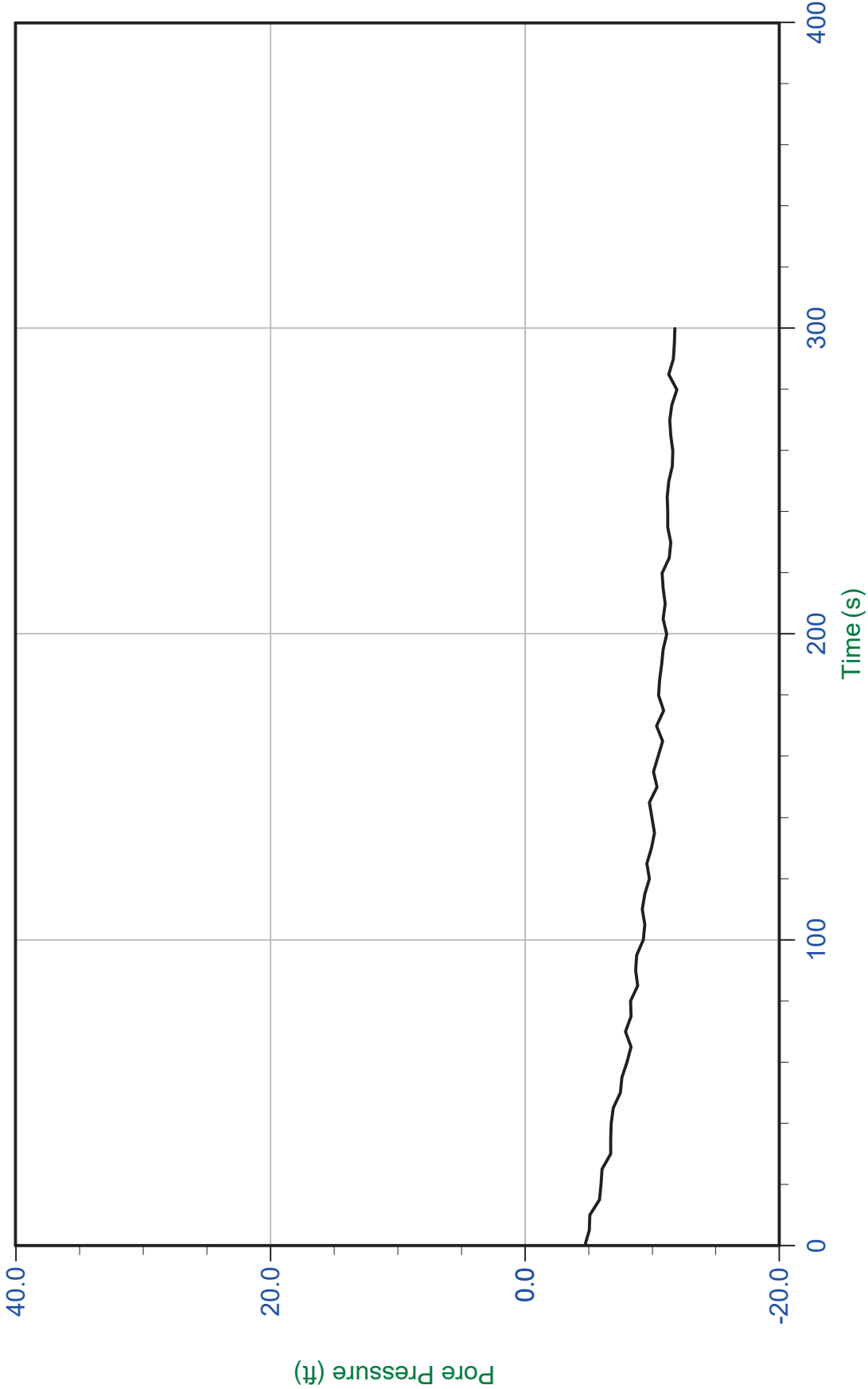
u Final: 14.6 ft

Trace Summary:



ENGEO

Job No: 21-56-22652
Date: 06/29/2021 07:25
Site: 1430 Chapin Avenue
Sounding: ECPT-01
Cone: 448:T1500F15U500 Area=15 cm²



Filename: 21-56-22652_SP01.PPF
Depth: 16.150 m / 52.985 ft
Duration: 300.0 s

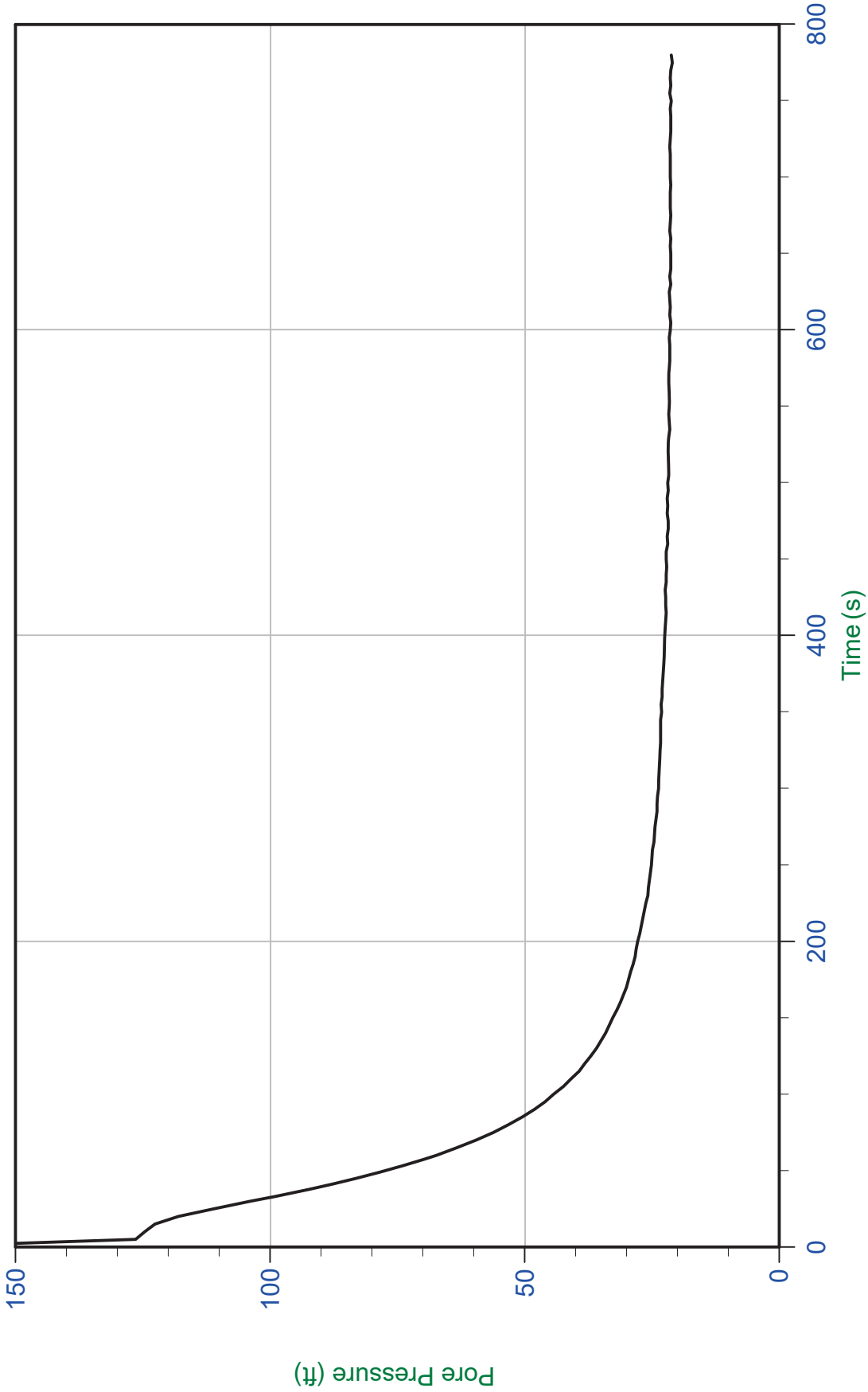
u Min: -11.9 ft
u Max: -4.7 ft
u Final: -11.8 ft

Trace Summary:



ENGEO

Job No: 21-56-22652
Date: 06/29/2021 08:25
Site: 1430 Chapin Avenue
Sounding: ECPT-02
Cone: 448:T1500F15U500 Area=15 cm²



Filename: 21-56-22652_CP02.PPF
Depth: 10.800 m / 35.433 ft
Duration: 780.0 s
u Min: 21.1 ft
u Max: 171.3 ft
u Final: 21.2 ft
WT: 4.355 m / 14.286 ft
Ueq: 21.1 ft

Trace Summary:



ENGEO

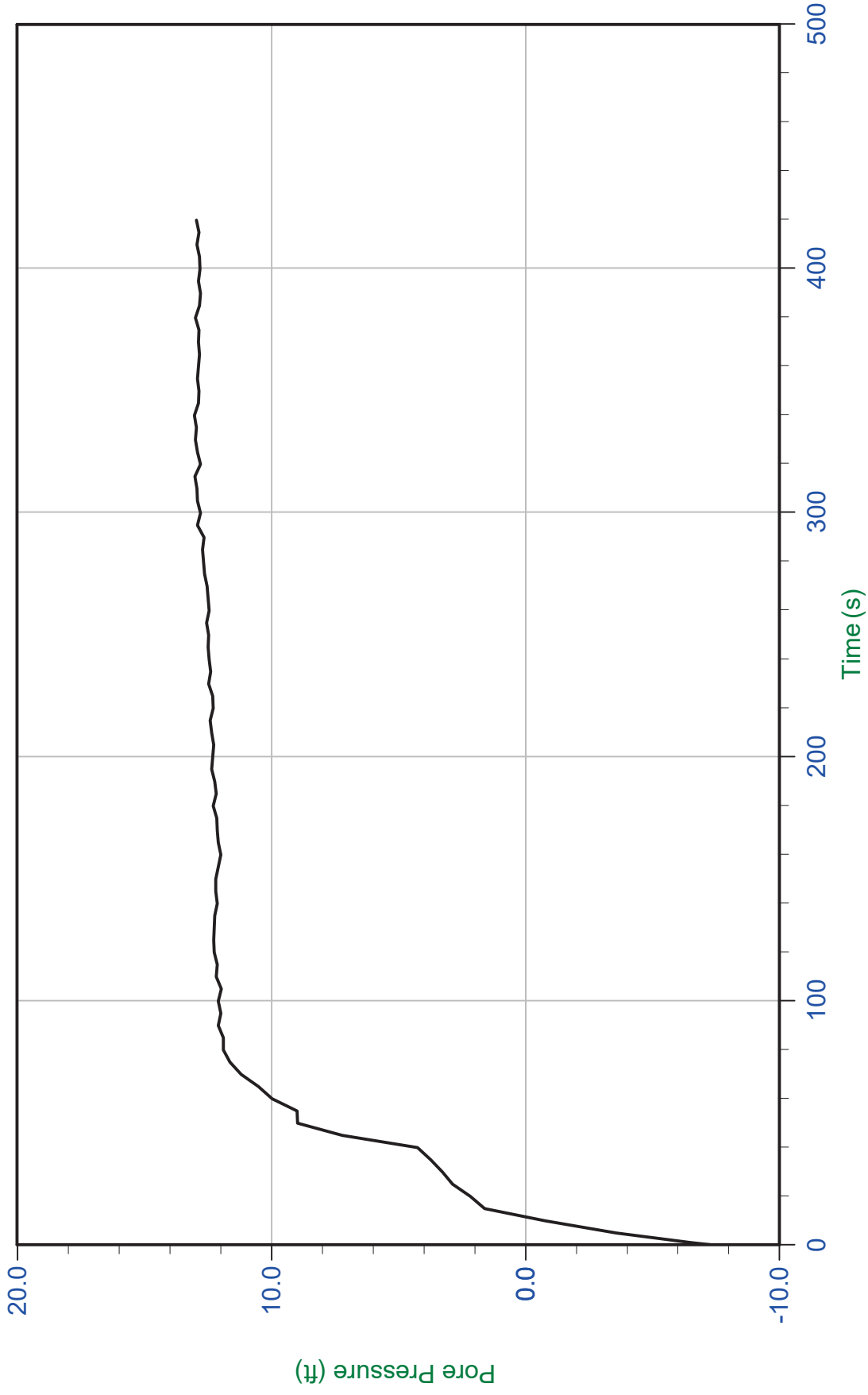
Job No: 21-56-22652

Date: 06/29/2021 10:05

Site: 1430 Chapin Avenue

Sounding: ECPT-04

Cone: 448:T1500F15U500 Area=15 cm²



Filename: 21-56-22652_CP04.PPF

Depth: 8.375 m / 27.477 ft

Duration: 420.0 s

WT: 4.434 m / 14.547 ft

Ueq: 12.9 ft

u Min: -7.3 ft

u Max: 13.0 ft

u Final: 13.0 ft

Trace Summary:



APPENDIX B

PRELIMINARY CPT-BASED LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS REPORT

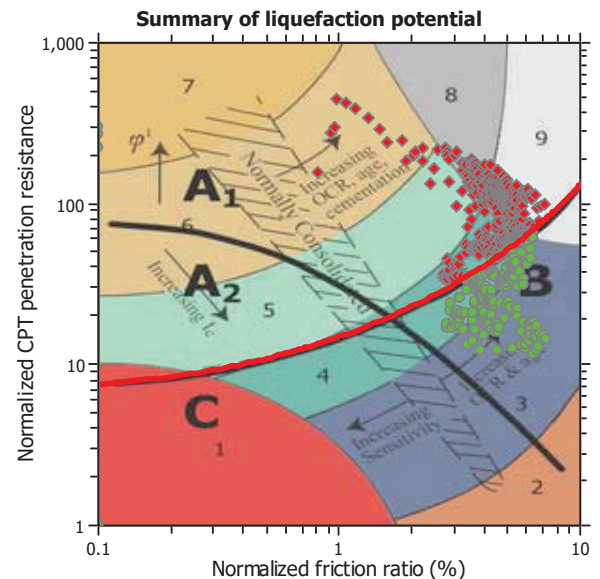
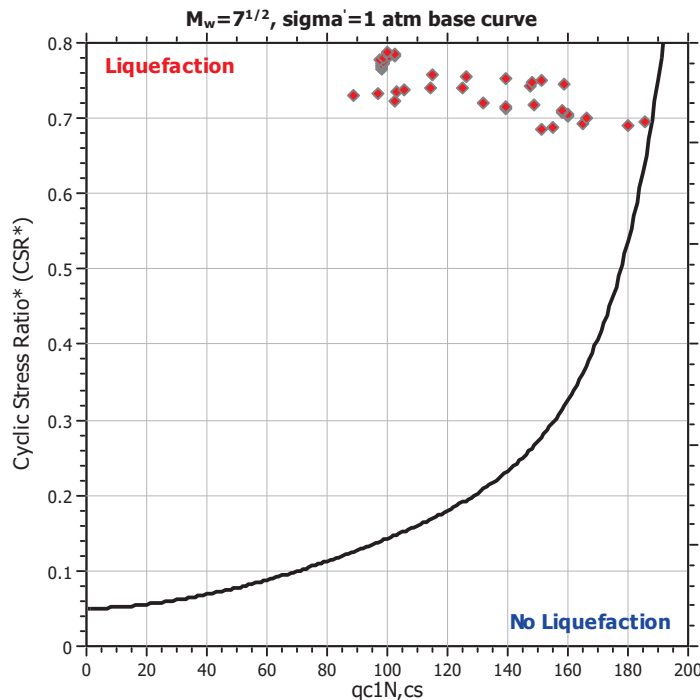
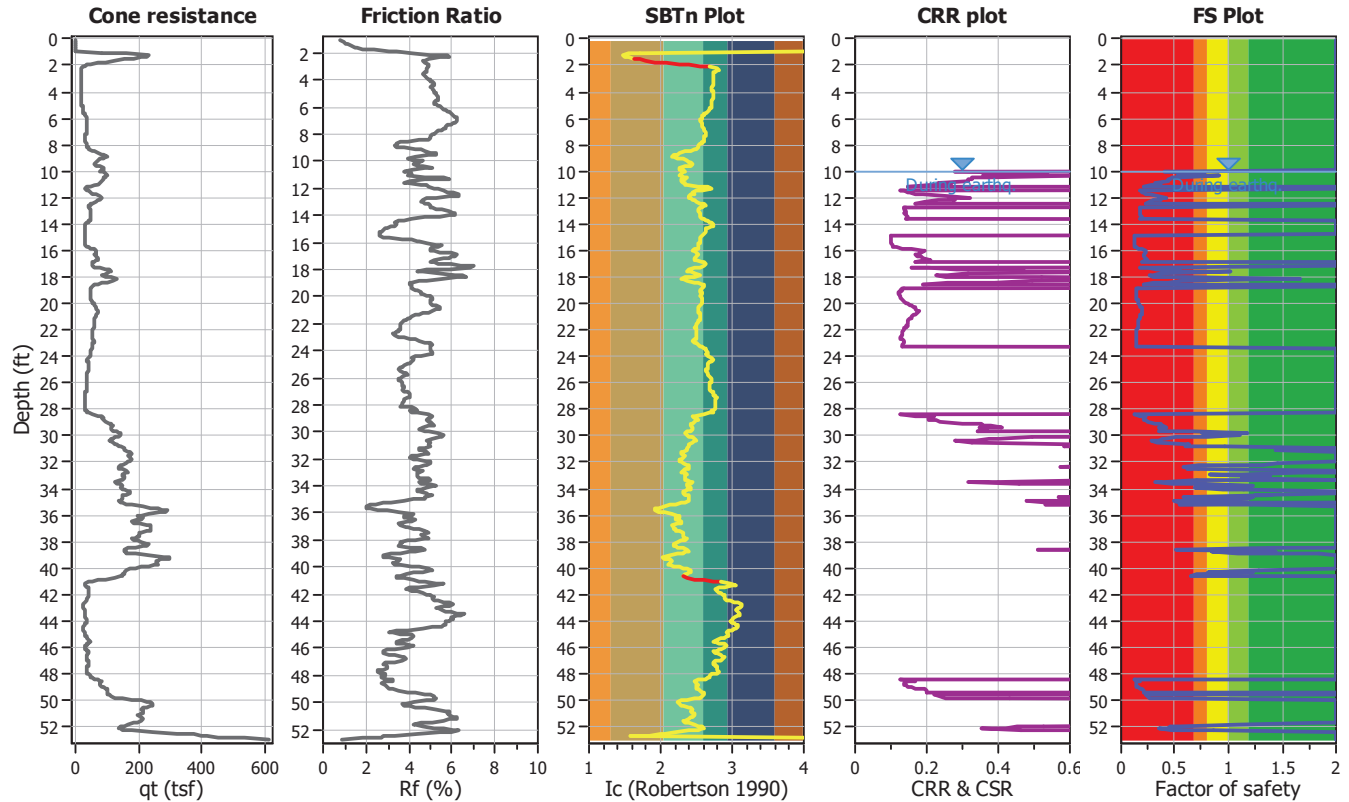
Project title : 1430 Chapin

Location : 1430 Chapin, Burlingame

CPT file : ECPT-1

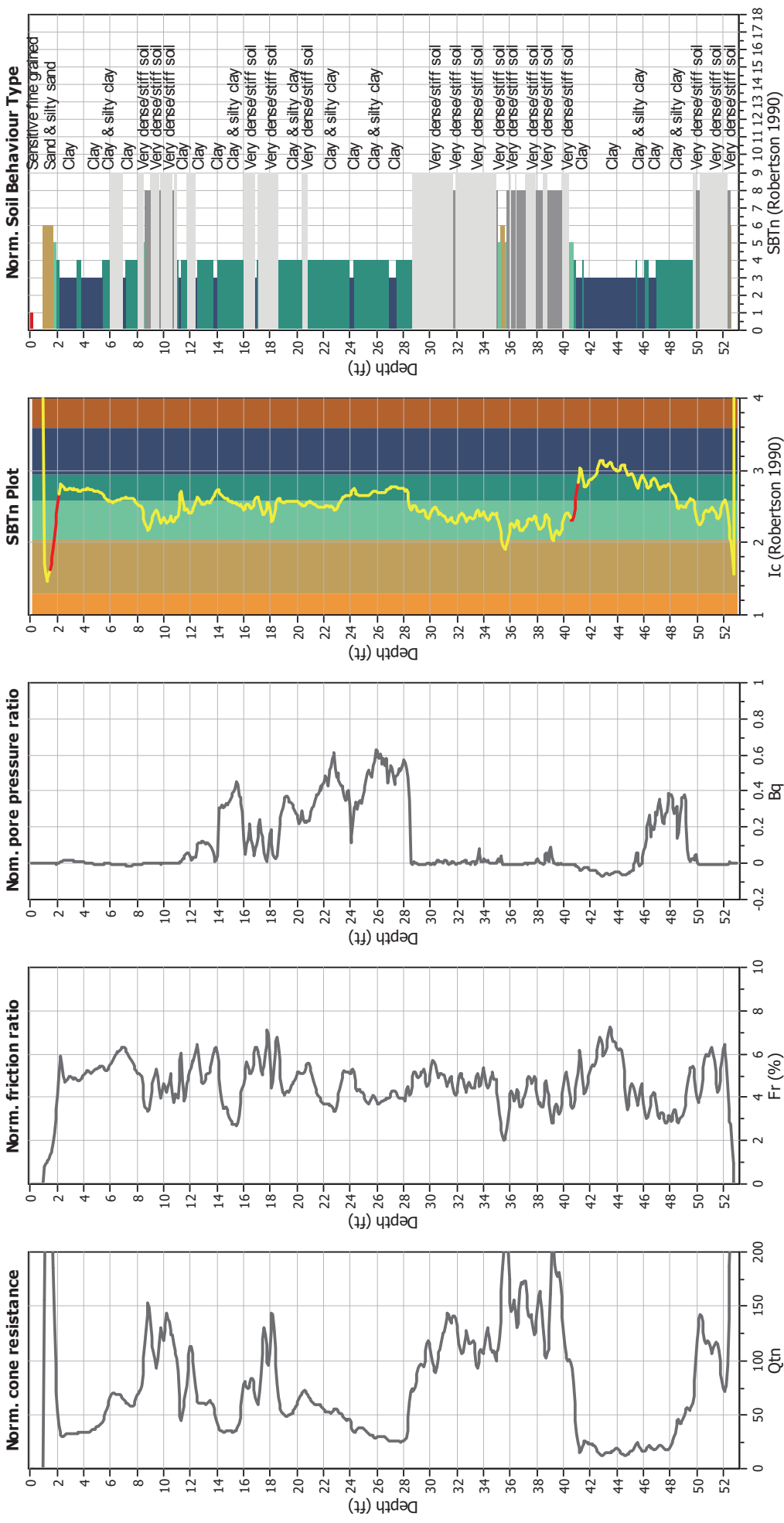
Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.96	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check soil softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



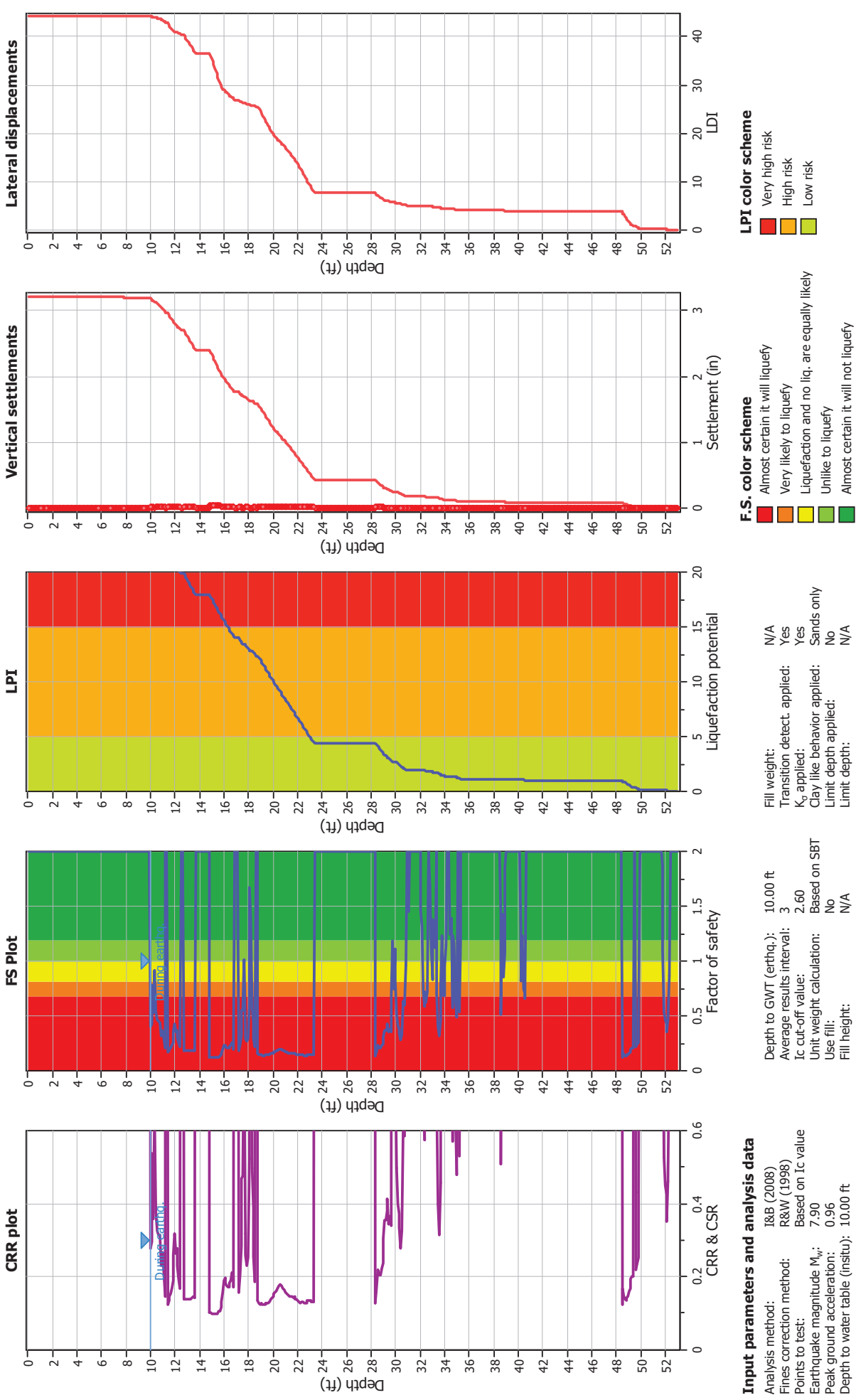
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Sands only
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	No
Peak ground acceleration:	0.96	Use fill:	No	Limit depth applied:	N/A
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



LIQUEFACTION ANALYSIS REPORT

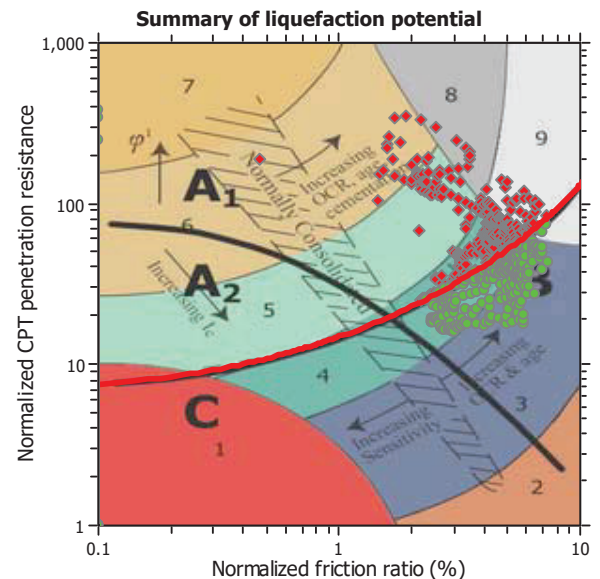
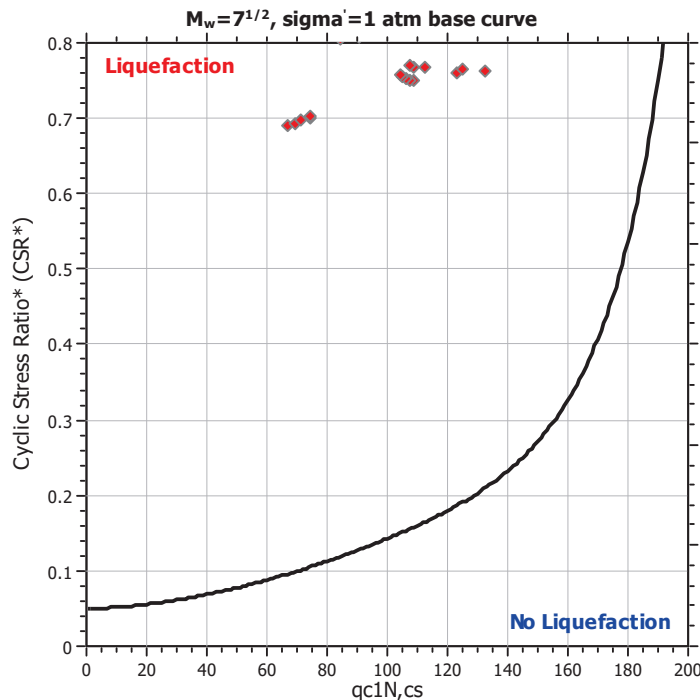
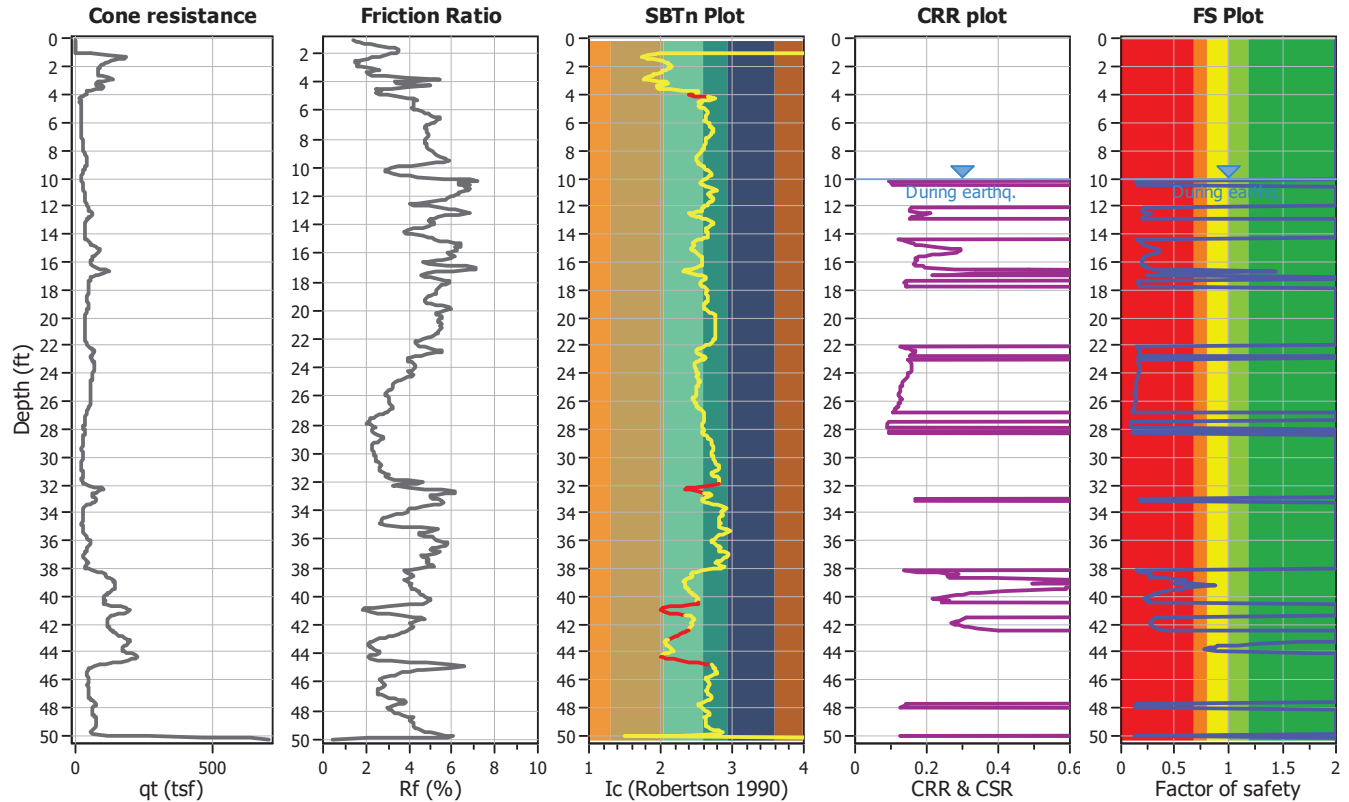
Project title : 1430 Chapin

Location : 1430 Chapin, Burlingame

CPT file : ECPT-2

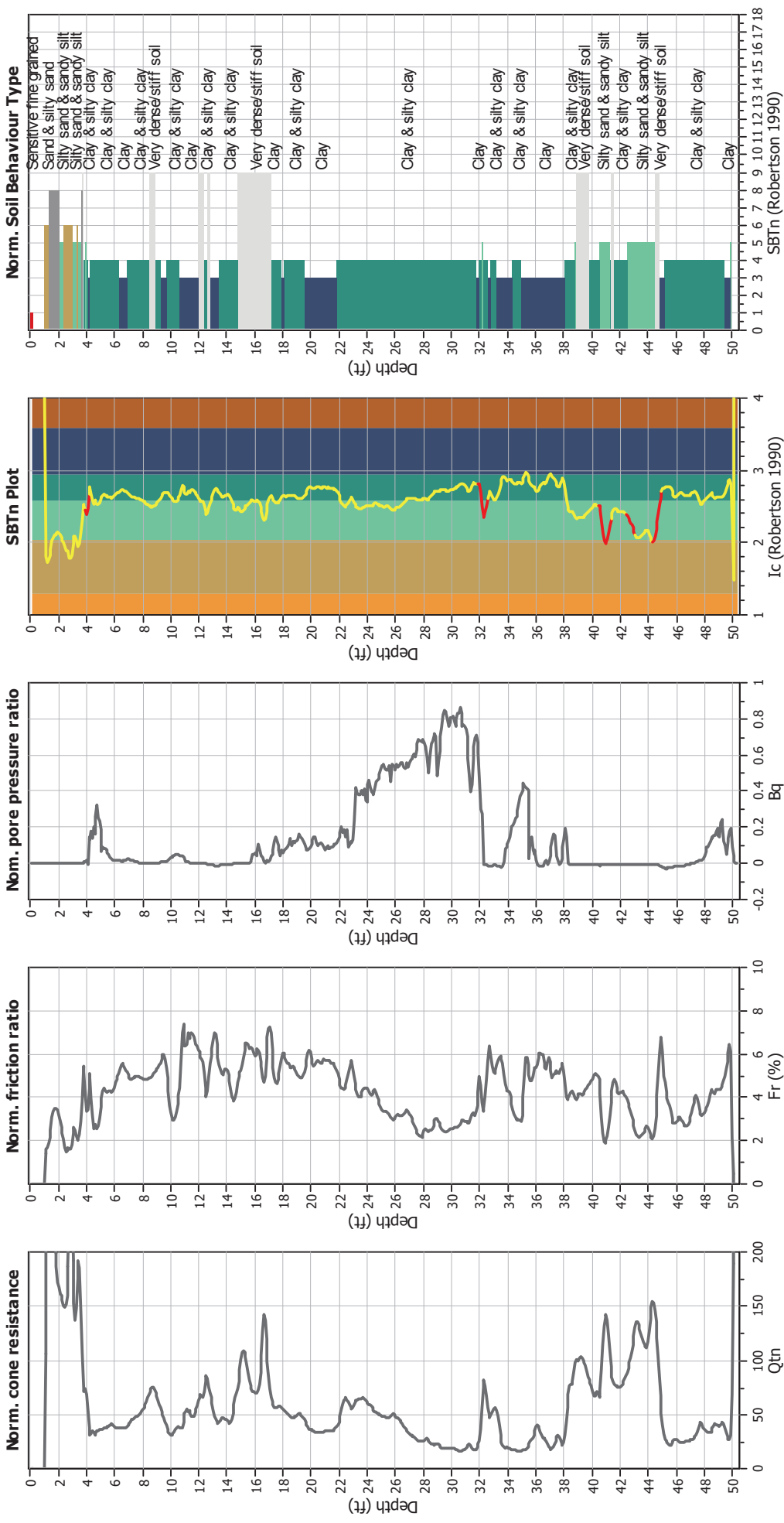
Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.96	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



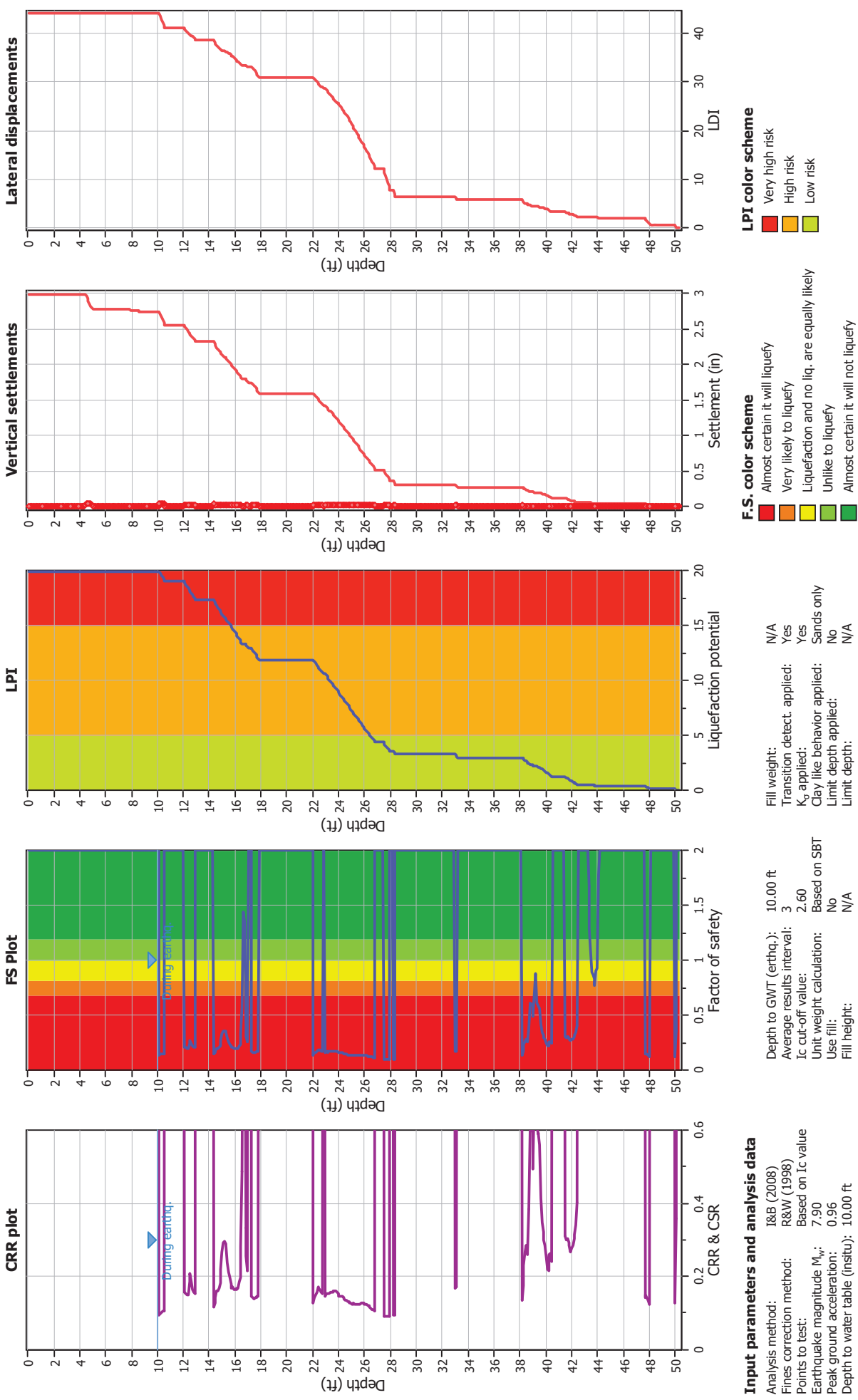
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	Sands only
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	No
Peak ground acceleration:	0.96	Use fill:	No	Limit depth applied:	N/A
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



LIQUEFACTION ANALYSIS REPORT

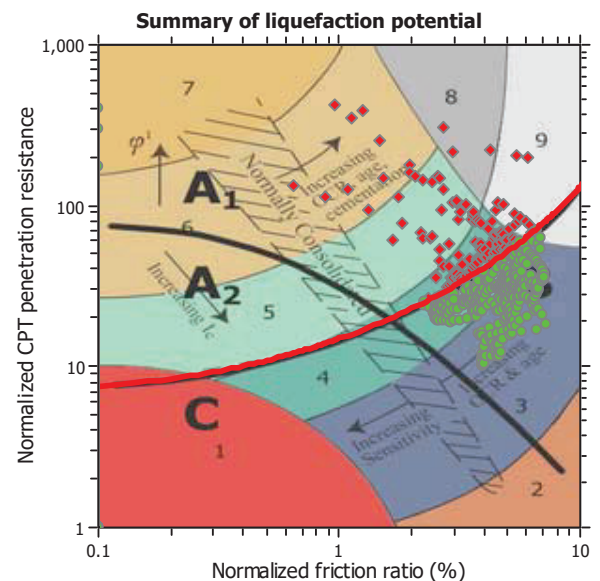
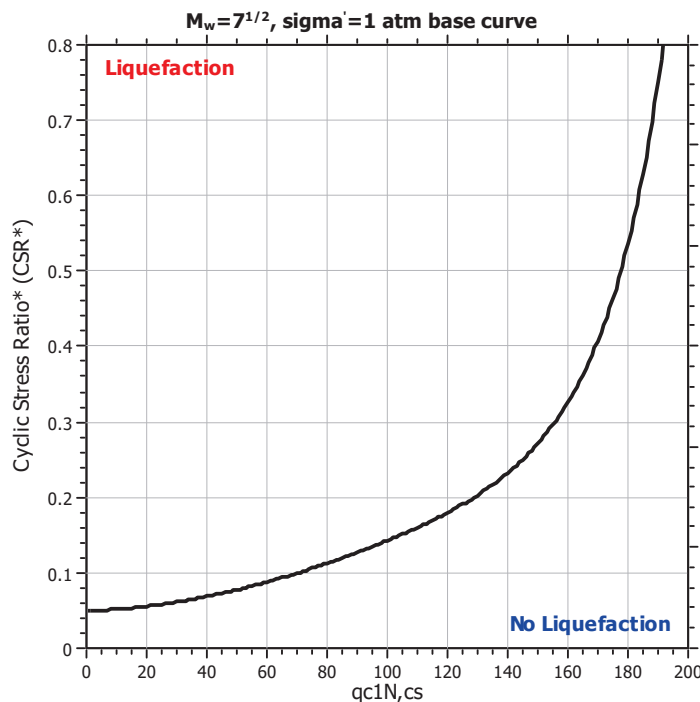
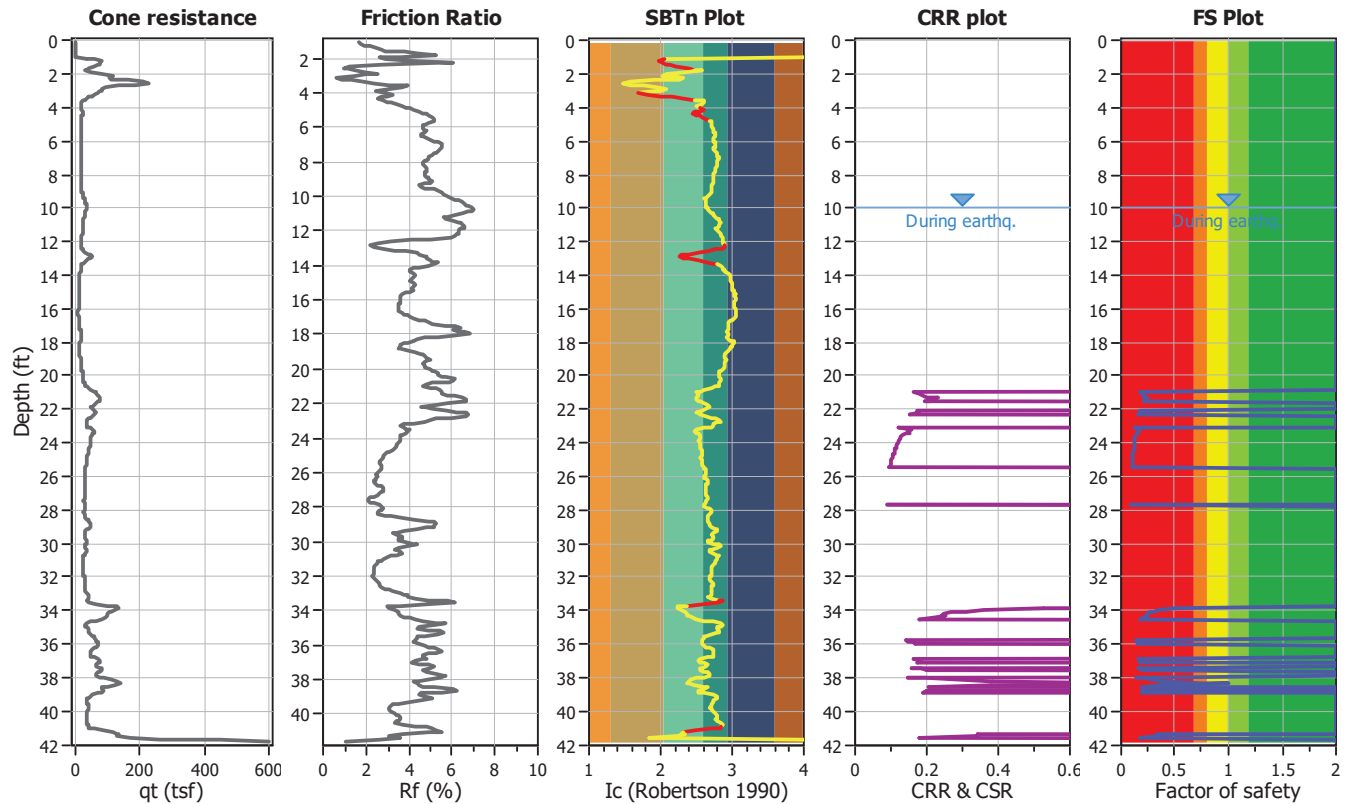
Project title : 1430 Chapin

Location : 1430 Chapin, Burlingame

CPT file : ECPT-3

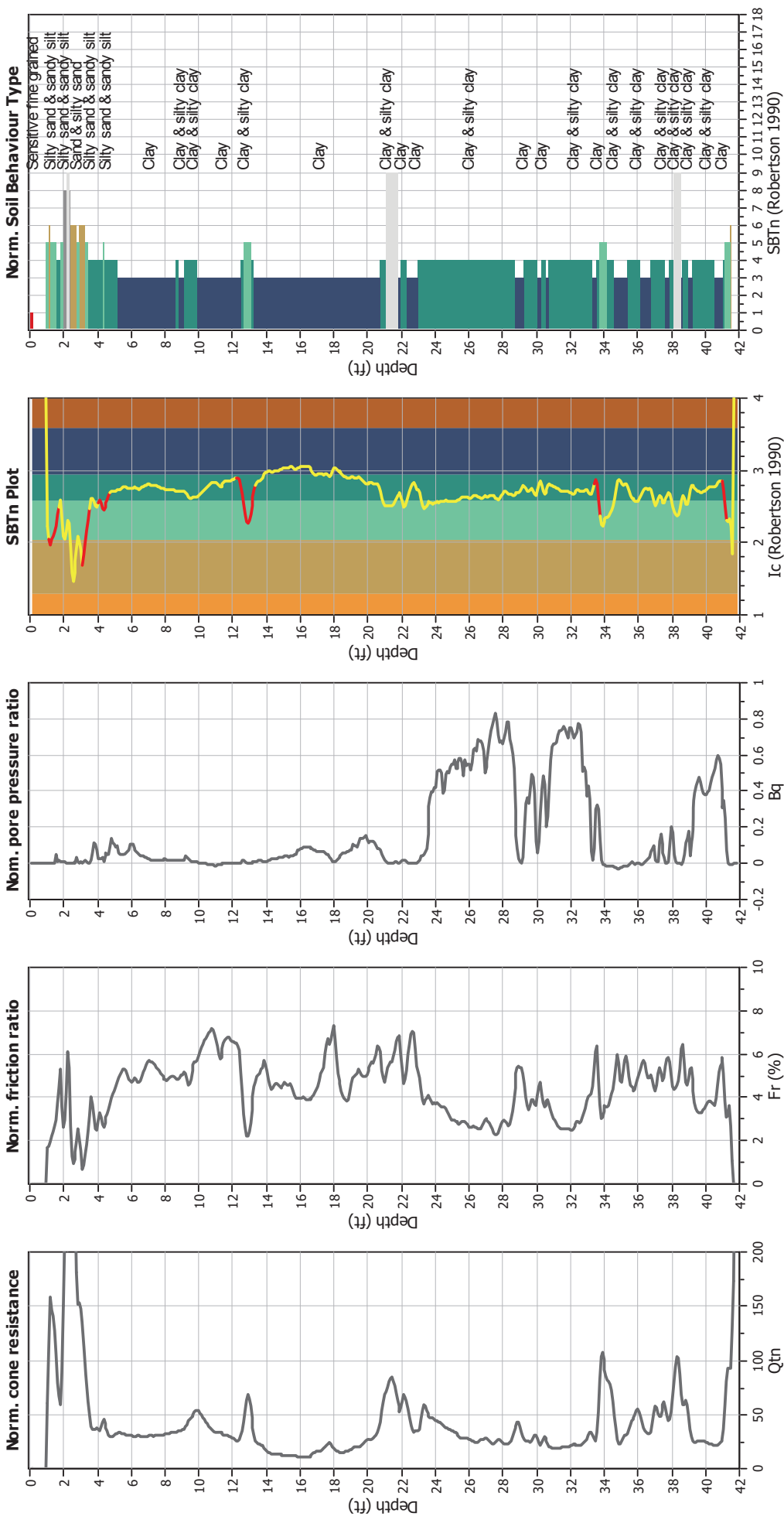
Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.96	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

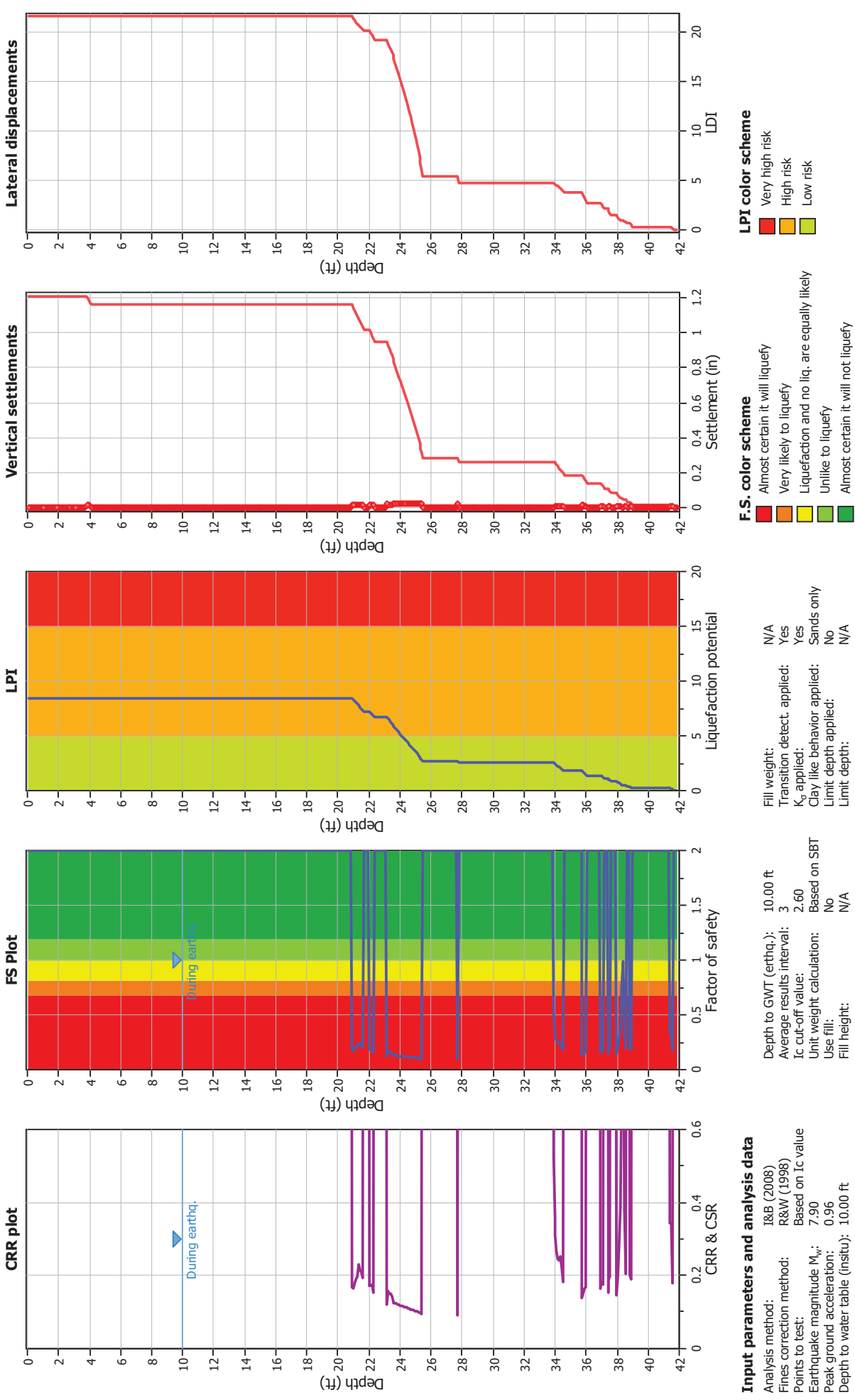
CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.96	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



LIQUEFACTION ANALYSIS REPORT

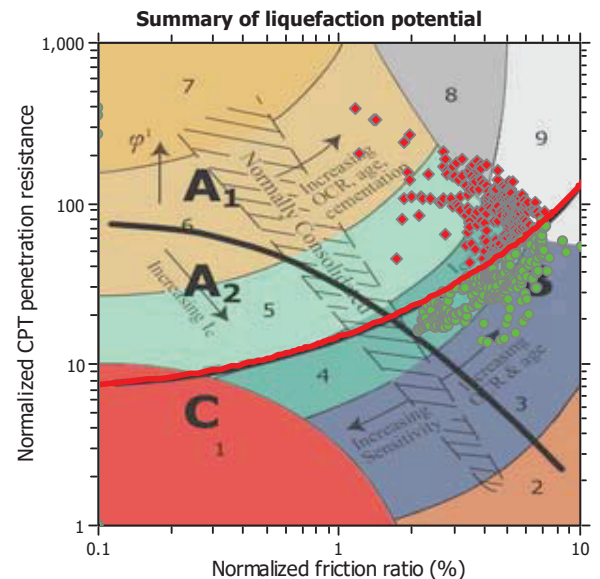
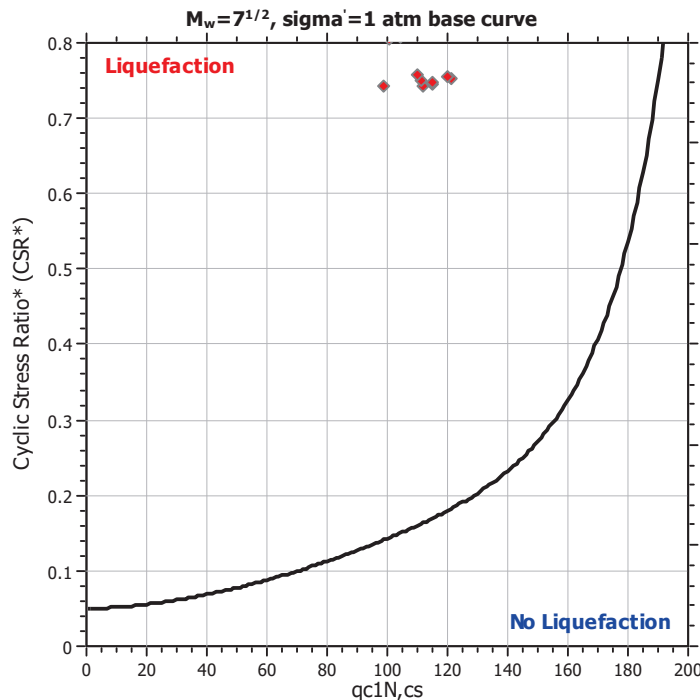
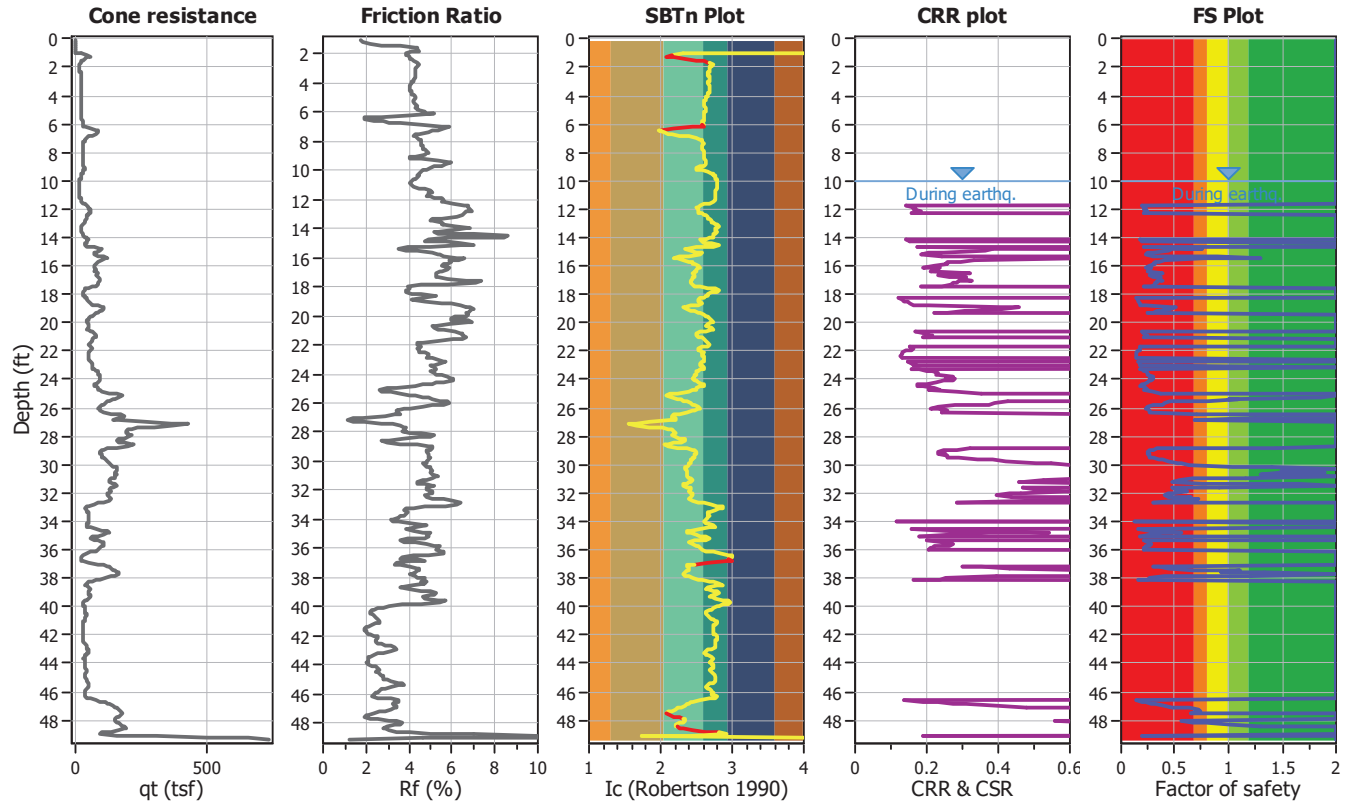
Project title : 1430 Chapin

Location : 1430 Chapin, Burlingame

CPT file : ECPT-4

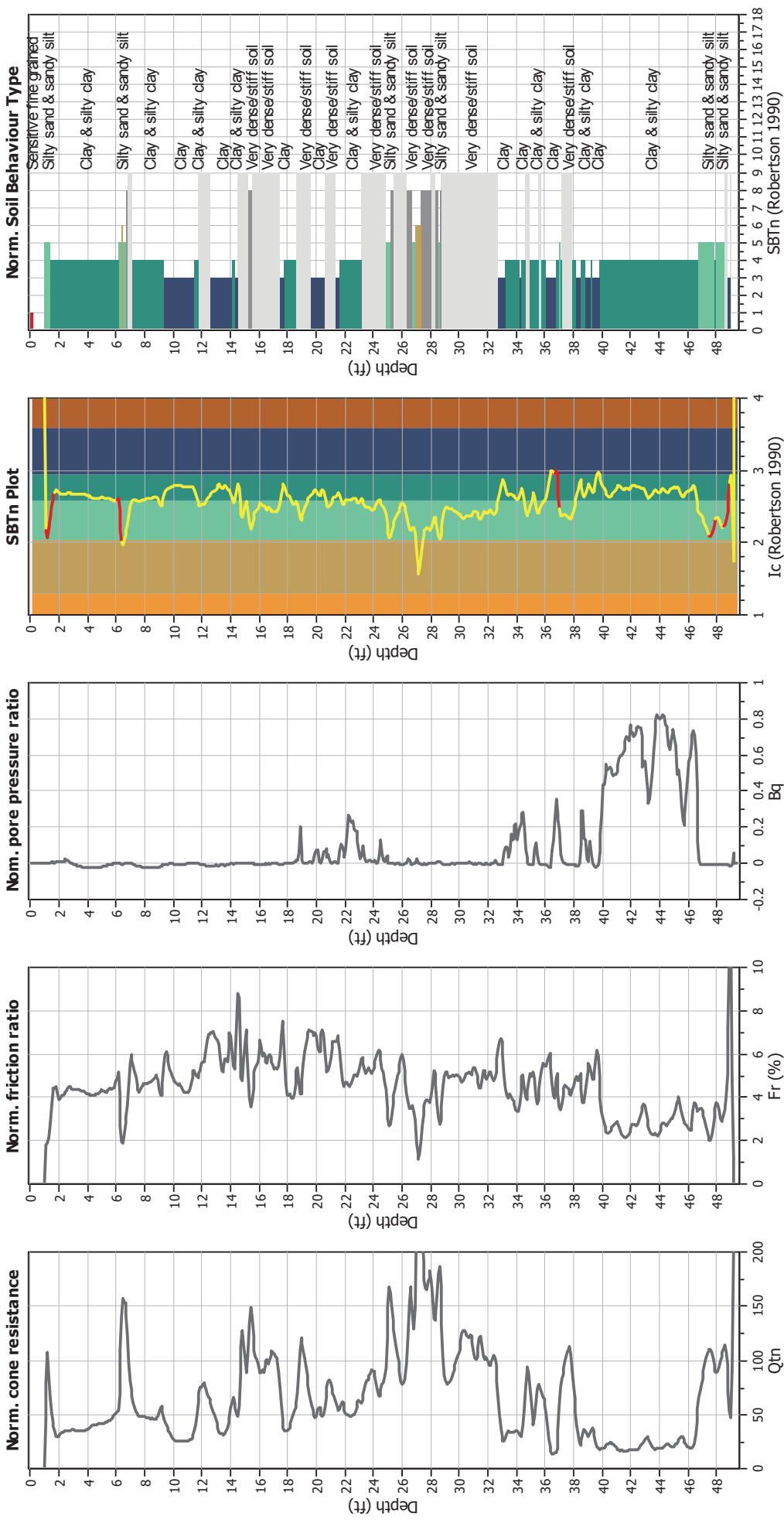
Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.96	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.96	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots

