PROPOSED RESIDENTIAL DEVELOPMENT
740 TENNANT AVENUE
MORGAN HILL, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Scott Canel
The Canel Companies
1949 St. Johns Avenue, Suite 200
Highland Park, IL 60035

PREPARED BY
ENGEO Incorporated

July 28, 2023

PROJECT NO.
23556.000.001

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23556.000.001

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Mr. Scott Canel
The Canel Companies
1949 St. Johns Avenue, Suite 200
Highland Park, IL 60035

Subject: Proposed Residential Development
740 Tennant Avenue
Morgan Hill, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Canel:

We are pleased to present this preliminary geotechnical exploration for the proposed residential development at 740 Tennant Avenue in Morgan Hill, California. The accompanying report presents our findings and preliminary conclusions and recommendations regarding the proposed development.

Based on our preliminary findings, it is our opinion from a geotechnical viewpoint that the proposed development is feasible from a geotechnical standpoint. The primary geotechnical concerns for the site development include presence of existing undocumented fill, expansive soil, compressibility soft and seismic hazards. This report provides our preliminary conclusions and recommendations for planning.

A design-level geotechnical exploration should be conducted prior to site development once more detailed land plans and structural loads have been prepared.

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

Alvaro Crisanto

Theodore P. Bayham, GE, CEG

Jeanine T. Ruffoni, PE, GE

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical exploration report in support of your evaluation of the subject site located in Morgan Hill, California, for residential development. As outlined in our executed agreement dated June 16, 2023, The Canel Companies authorized us to perform the following scope of services.

- Review available literature and geologic maps for the study area.
- Subsurface exploration
- Data analysis and development of preliminary geotechnical recommendations
- Report preparation

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1.2 SITE LOCATION

The approximately 11.2-acre site is located near the southern city limits of Morgan Hill, California, in an area of mixed residential and commercial development, as shown in the Vicinity Map (Figure 1). The parcel is located at 740 Tennant Avenue and is identified by Assessor's Parcel Number (APN) 817-08-032. The site is generally bounded by Butterfield Boulevard to the west, Tennant Avenue to the north, and undeveloped land to the south and east. Highway 101 on-ramp is located approximately 400 feet to the east. The site generally consists of undeveloped agricultural land consisting of dry desiccated surface soil and tall dry vegetation throughout the site.

1.3 PROPOSED DEVELOPMENT

We understand the proposed development will consist of at-grade multi-family, residential buildings up to four stories tall. Structural loading information is not available at the time of this report writing. We assume the structures will be conventional wood-frame construction. We assume building loads will be typical for the respective construction types. We anticipate associated improvements may include utilities, roadways, and sidewalks, bioretention basins, and other landscaped areas.

2.0 FINDINGS

2.1 SITE HISTORY

We reviewed the following historical topographic maps and aerial photographs that we obtained from Environmental Data Resources during our concurrent phase I environmental site assessment.

TABLE 2.1-1: Historical Review Summary

HISTORICAL MAP/PHOTOGRAPH	YEARS
Topographic Maps	1917, 1939, 1955, 1968, 1971, 1973, 1980, 1981, 1993, 1994, 1996, 2012, 2015, 2018
Aerial Photographs	1939, 1940, 1950, 1956, 1963, 1968, 1970, 1974, 1982, 1993, 2006, 2009, 2012, 2016, 2020

The 1939 to 1993 photographs indicate that the site was primarily occupied by an orchard. The 1974 photograph shows Highway 101 to the east of the site area and shows three to five structures that were present in the central portion of the site until they were demolished as seen in the 2012 photograph. The 1993 to 2012 photographs show construction of the commercial development along the western perimeter of the site has begun, with the construction of a new road separating the two sites. Between the 2016 and 2020 photograph, the site conditions have remained relatively unchanged. During our site visit on June 28, 2023, we observed that the site conditions have remained relatively unchanged compared to the conditions shown in the 2020 aerial photograph.

2.2 FIELD EXPLORATION

We retained the services of Conetec to advance cone penetration test (CPT) soundings at four locations, as shown in the Site Plan (Figure 2). The CPTs were advanced to a maximum depth of approximately 44½ feet below ground surface (bgs). At Sounding 1-CPT4, we encountered shallow refusal at an approximate depth of 8 feet bgs; it was determined in the field that the cone had encountered very hard material during the sounding which damaged the cone. Measurements include the tip resistance to penetration of the cone, resistance of the surface sleeve, and pore pressure (Robertson and Campanella, 1988); CPTs were performed in accordance with ASTM D-5778. Pore pressure dissipation testing was performed in three CPTs to interpret hydrostatic groundwater levels. We present the CPT logs in Appendix A.

2.3 GEOLOGY

2.3.1 Regional Geologic Setting

The site is located within the Coast Ranges geomorphic province. The Coast Ranges have experienced a complex geological history characterized by Late Tertiary folding and faulting that has resulted in a series of northwest-trending mountain ranges and intervening valleys. The site is in the flatland areas of the East Bay, near the western edge of a series of uplifted sedimentary formations that comprise the Diablo Range.

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

The regional geologic mapping is shown in Figure 3.

2.3.2 Site Geology

According to published maps by Dibblee (2005), the site is underlain by Quaternary-age alluvial fan deposits (Qa), consisting of alluvial gravel and sand and clay of valley areas. Published maps by McLoughlin (2001) indicate the site is underlain by Pleistocene-age alluvial fan deposits, comprising unsorted gravel, sand, and silt that was deposited in older alluvial fans. These alluvial deposits include older alluvial fan deposits incised by younger Pleistocene- and Holocene-age deposits.

2.4 FAULTING AND SEISMICITY

Northern California contains numerous active earthquake faults, as shown in Figure 4. The nearby active faults to the site include the Calaveras, San Andreas, Sargent, and Hayward faults. An active fault is defined by the California Geologic Survey (CGS) as one that has had surface displacement within Holocene time (about the last 11,700 years) (CGS, 2018).

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

To evaluate nearby active faults that can generate strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool and disaggregated the hazard at the peak ground acceleration (PGA) for a 2,475-year return period, with the resulting faults listed below in Table 2.4-1.

TABLE 2.4-1: Active Faults Capable of Producing Significant Ground Shaking at the Site (Latitude: 37.1163 Longitude: -121.6304)

SOURCE[a]	R_{RUP}[b]		MOMENT MAGNITUDE[c] M_W
	(KM)	(MILES)	
Calaveras (Central) [2]	8.54	5.31	7.14
San Andreas (Santa Cruz Mts) [3]	15.77	9.80	7.73
Sargent [2]	13.65	8.48	7.34
Hayward (So) extension [0]	14.33	8.90	6.85

Notes: a. Fault System (Fault Section) [Fault Subsection assigned by UCERF3]
b. R_{RUP} = nearest fault-to-site rupture distance
c. Fault-to-site distances and maximum moment magnitude based on USGS Unified Hazard Tool - Edition: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

2.5 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting and liquefaction. Common secondary seismic hazards include ground shaking, liquefaction, and lateral spreading. The following sections present a discussion of these and other hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, or landslides, is considered low to negligible at the site.

2.5.1 Ground Rupture

Since there are no known active faults that transverse the site, and the site is not located within an Alquist-Priolo Earthquake Fault Study Zone, it is our opinion that the risk of ground rupture is low.

2.5.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the Bay Region could cause considerable ground shaking at the site, like that which has occurred in the past. 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, 1996). California Building Code (CBC, 2022) seismic design parameters are presented later in this report.

2.5.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. 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, it 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 because of cyclic loading.

According to published maps under the Seismic Hazard Mapping Act (1990), the site is in an area where the risk of seismic-induced liquefaction has not been evaluated by the California Geologic Survey, as shown in Figure 5. Regional mapping by the United State Geological Survey (Witter et al., 2006) indicates the area is mapped in an area of low liquefaction susceptibility. As previously discussed, the site is in an area underlain by upper Pleistocene deposits that may be incised by younger Holocene-age deposits. These Holocene deposits are typically less consolidated than the older Pleistocene deposits and may be susceptible to seismic-induced shaking.

To evaluate the susceptibility of site-specific layers that may trigger during a seismic event, we performed a preliminary analysis using empirical data recorded during the CPT explorations. We analyzed potential liquefaction based on the CPT data and the computer software CLiq (Version 3.5.2.19) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). For our analysis, we utilized a Peak Ground Acceleration (PGA) of 0.81g consistent with the 2022 California Building Code (CBC) and maximum moment magnitude of 7.7, and a groundwater depth of 13 feet bgs, based on recorded historical high levels in the site vicinity.

Dynamic densification settlement of loose granular soil above the groundwater table, also called dry sand settlement, can cause settlement of the ground surface due to earthquake-induced vibrations. We performed a preliminary CPT-based analysis of potential dynamic densification settlement for sand layers above the groundwater table. Based on the results of our analyses, we estimate the settlement to be negligible. This estimate is preliminary in nature and should not be considered a recommendation for design. The potential for dynamic densification of granular soil above the groundwater table should be further evaluated during design-level exploration based on blow counts and collection of soil samples.

Based on our preliminary liquefaction analysis, sand, and silt mixtures between approximately 13 to 23 feet bgs may be potentially liquefiable. Based on our analysis the total seismic-induced settlement ranges between less than ½ inch to 2½ inches.

2.5.4 Liquefaction-Induced Surface Rupture

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. The risk of surface expression should be assessed based on the findings during the design-level liquefaction hazard evaluation. Based on our preliminary findings, it is our opinion that the risk of surface rupture is low.

2.5.5 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, the effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope. It is our opinion that the risk of lateral spreading at the site is low.

2.6 2022 CBC DESIGN PARAMETERS

The 2022 CBC utilizes design criteria established in the ASCE/SEI Standard" Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16). Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2022 CBC. We provide the 2022 CBC seismic design parameters in Table 2.6-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 2.6-1: 2022 CBC Seismic Design Parameters, Latitude: 37.1163 - **Longitude:** -121.6304

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.624
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.6
Site Coefficient, F_A	1.2
Site Coefficient, F_V	See ASCE Section 11.4.8

PARAMETER	VALUE
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.948
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	See ASCE Section 11.4.8
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.299
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	See ASCE Section 11.4.8
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.677
Site Coefficient, F_{PGA}	1.2
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.813

ASCE 7-16 requires a site-specific ground-motion hazard analysis for Site Class D sites with a mapped S_1 value greater than or equal to 0.2; however, Section 11.4.8 of ASCE 7-16 and Supplement No. 3 provide an exception to this requirement. A site-specific ground-motion hazard analysis is not required where the value of the parameter S_{M1} determined by Equation 11.4-2 and shown in Table 2.6-1 is increased by 50 percent for developing the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response, calculating S_{D1}, and evaluating C_s in accordance with Chapter 12 of ASCE 7-16.

During design-level studies, we recommend that we collaborate with the structural engineer of record to further evaluate the effects of taking the exception on the structural design and identify the need for performing a site-specific ground-motion hazard analysis. We can prepare a proposal for a site-specific ground-motion hazard analysis, if requested.

2.7 SURFACE CONDITIONS

At the time of our field exploration in June 2023, the surface conditions at the site generally consisted of undeveloped agricultural land consisting of dry desiccated surface soil and tall dry vegetation throughout the site (Photo 2.7-1).

We observed the following features during our field exploration.

- An off-site drainage canal trends along Butterfield Boulevard with a headwall on the northern end that allows water conveyance underneath Tennant Avenue
- A plastic water tank located in the central portion of the site (Photo 2.7-2)
- A utility meter is in the central portion of the site
- A remnant concrete foundation is in the central portion of the site
- A sealed water well is in the central portion of the site
- An active water quality monitoring well is in the western portion of the site

Please refer to the Site Plan, Figure 2, for more information on site features

PHOTO 2.7-1: Northeast Corner Looking West

PHOTO 2.7-2: Plastic Water Tank

Based on our review of Google Earth Imagery, the current topography of the site is relatively level and ranges from Elevation 330 feet in the south to Elevation 333 feet (WGS-84) in the north.

2.8 SUBSURFACE CONDITIONS

Based on the results of the CPT interpretations, the subsurface conditions generally consist of medium dense to dense gravelly sands that extend to depths ranging between 11 feet below ground surface (bgs) and 17 feet bgs. Beneath the gravelly sands, we encountered very stiff to hard silts and to depths ranging between 11 feet and 29 feet bgs. We encountered discontinuous layers of very stiff to hard clay and dense to very dense sands; these layers ranged between 2 and 5 feet thick and were generally encountered below 30 feet.

The subsurface conditions encountered are generally consistent with the mapped regional geology.

2.9 GROUNDWATER CONDITIONS

As discussed in Section 2.2, we performed pore pressure dissipation tests at three CPT locations to estimate static groundwater conditions. We were able to reach equilibrium for the pore pressure dissipation test readings at two CPT locations. We summarize our findings in Table 2.9-1.

TABLE 2.9-1: Pore Pressure Dissipation Test Results

EXPLORATION ID	INTERPRETED GROUNDWATER DEPTH (FEET, BGS)
1-CPT2	13¼
1-CPT3	19¾

Our review of groundwater data also included publicly available resources, including compiled well data presented in the State Water Data Library (California Department of Water Resources) and county groundwater database (Valley Water). The findings from the publicly available resources indicated that groundwater generally ranges between 20 feet to 50 feet below ground surface.

For the purposes of our analyses and recommendations, we consider an appropriate groundwater depth of 13 feet bgs to be appropriate to use 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, and changes in irrigation and drainage patterns.

3.0 PRELIMINARY CONCLUSIONS

Based on our preliminary findings, it is our opinion from a geotechnical viewpoint that the proposed development is feasible from a geotechnical standpoint. The primary geotechnical concerns for the site development include presence of existing undocumented fill, expansive soil, compressibility soft and seismic hazards.

3.1 SEISMIC SETTLEMENT - PRELIMINARY LIQUEFACTION ANALYSIS

As previously discussed, it is our preliminary evaluation that sand and silt layers between approximately 13 to 23 feet bgs may be potentially liquefiable. Based on our preliminary analysis the total seismic-induced settlement ranges between less than ½ inch to 2½ inches. 1-CPT4 was advanced to approximately 8 feet bgs due to shallow refusal; therefore, we did not calculate the seismically induced settlement for this CPT.

Depending on the location of proposed buildings, the structures may need to be designed for total seismic-induced settlement of up to 2½ inches and differential settlement taken as one-half the total estimate over the width of the building, or over a lateral distance of 40 feet, whichever is less.

TABLE 3.1-1: Liquefaction Analysis Results

CPT LOCATION	CALCULATION METHOD		
	NCEER (2001)	MOSS et al. (2006)	BOULANGER & IDRISS (2014)
1-CPT1	1½	1½	2½
1-CPT2	¼	½	¾
1-CPT3	½	¾	1¼

3.2 EXISTING UNDOCUMENTED FILL AND LOOSE SURFICIAL SOIL

As previously discussed, the site has been primarily used for agricultural purposes. Due to this past usage, the upper 1 to 1½ feet of site soil may be disturbed and loose from seasonal tilling during agricultural operations; however, this depth of disturbed soil may be greater depending on depth of seasonal tilling. The upper disturbed soil layer and areas with undocumented fill may be subject to settlement that is not easily characterized.

3.3 COMPRESSIBLE SOIL

Soil may be subject to settlement when loaded with a new structure or additional fill. This settlement may occur as elastic or consolidation settlement. Elastic settlement is a function of soil stiffness while consolidation settlement is highly dependent on the number of water-filled voids within the soil. The rate of settlement is highly dependent on the permeability of the soil and the presence of water. Consequently, sandy soil will settle almost immediately, whereas clayey soil below the water table will settle much more slowly. Based on the CPT data, we generally encountered medium dense to dense coarse-grained soil below the interpreted groundwater table. It is our opinion that most of the static-induced settlement will occur as immediate settlement in the coarse-grained soil and will predominantly occur during construction. Additional analysis should be performed during the design-level study, in addition to incorporating anticipated building loads when they are provided by the project Structural Engineer.

3.4 EXPANSIVE SOIL

Based on our experience in the site vicinity, we anticipate the near-surface soil to have low to moderate expansive potential. It is our opinion that the potential for expansive behavior can be controlled by incorporating the preliminary fill placement recommendations provided in Section 4.3. In addition, the surface samples obtained during this study were intended to generally characterize the site conditions. During the design-level investigation, additional laboratory testing should be performed to develop foundation criteria.

4.0 PRELIMINARY RECOMMENDATIONS

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding site grading and foundation construction will be provided after future site-specific, design-level geotechnical exploration has been undertaken.

4.1 GENERAL SITE CLEARING

After demolition of the existing buildings, paving, and associated improvements, the site should be cleared of all obstructions, including existing foundations, and debris. Any existing underground utilities within the proposed development area should be identified and removed entirely, including pipes and their backfill. Depressions resulting from the removal of underground obstructions extending below the proposed finish grades should be cleared and backfilled with suitable material compacted to the recommendations presented in Section 4.4.

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 the geotechnical engineer 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.

Stripping and demolition below design grades should be cleaned to a firm undisturbed soil surface determined by the geotechnical engineer. This surface should then be cleaned, scarified, moisture conditioned, and backfilled with suitable material compacted to the recommendations presented in the Fill Compaction section. No loose or uncontrolled backfilling of depressions resulting from demolition and stripping should be permitted.

4.2 UNDOCUMENTED FILL

As described previously, undocumented fill is expected to be present at the site. Existing fill, existing utility trench backfill, existing foundation backfill, and existing landscape materials are considered undocumented and should be subexcavated to expose underlying competent native soils that are approved by the geotechnical engineer. If in a fill area, the base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the recommendations for engineered fill.

4.3 EXPANSIVE SOIL MITIGATION

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil or constructing the upper 18 inches of the building pad with non-expansive fill, can reduce the risk of structural damage associated with the expansive soil conditions. Generally, these recommendations include compaction control to reduce over-compaction of the soil and moisture conditioning the soil to well above the optimum moisture content.

The soil expansion potential of the site soil should be evaluated further at the time of design-level study and mitigated during grading activities and through appropriate improvement design.

4.4 SELECTION OF MATERIALS

Except for construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, high organic content soil (soil which contains more than 3 percent organic content by weight), and environmentally impacted soil (if any), we anticipate the site soil is suitable for use as engineered fill. Other material and debris, including trees with their root balls, should be removed from the project site.

We recommend removal of existing fill (if encountered during grading), stripping of organics, scarification, moisture conditioning, and compaction of the soil prior to fill placement. For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas:

- Test Procedures: ASTM D-1557.
- Required Moisture Content: Not less than 3 percentage points above optimum moisture content for PI > 12.
Not less than 2 percentage points above optimum moisture content for PI ≤ 12.
- Minimum Relative Compaction: Not less than 90 percent for PI > 12.
Not less than 95 percent for PI ≤ 12.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. If imported fill material is characterized and following the design-level geotechnical report, the recommendations may change with respect to the soil type.

4.5 SURFACE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. Regarding 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 of 5 percent away from foundations. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Consider the use of surface drainage collection system to reduce ponding of water at the ground surface near the foundation, pavements, or exterior flatwork.
3. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
4. 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 Slope Gradients

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter. The contractor is responsible for constructing temporary construction slopes in accordance with CALOSHA requirements.

4.7 PRELIMINARY FOUNDATION RECOMMENDATIONS

Based on the potential for seismically induced settlement resulting from dynamic densification of sandy layers during a seismic event, we recommend use of post-tensioned (PT) mat foundation to support the proposed residential single-family structures. For planning purposes, the structural engineer should consider up to 2½ inches of total seismically induced settlement. Based on our preliminary analysis, we anticipate long-term static consolidation settlement will be negligible.

The differential settlement can be taken as one-half the total estimate over the width of the building, or a lateral distance of 40 feet, whichever is less. Settlement estimates are preliminary and should be refined during design-level study. The proposed development may be supported by a structural mat foundation, conventional footing system with slab-on-grade flooring, or a post-tension mat slab.

4.7.1 Post-Tension Mat Slab

Based on cost and constructability, the team may consider using post-tensioned mat foundations for the ground surface foundation elements. We recommend that structures supported on post-tensioned mat foundations be founded on properly moisture conditioned subgrades. An allowable bearing pressure of 1,000 psf can be used for this foundation type.

4.7.2 Conventionally Reinforced Structural Mat Foundation

The proposed development may be supported by a structural mat foundation designed to tolerate minor differential settlement and expansive soils. We anticipate a minimum mat thickness of 18 inches and the structural mat should be stiff enough to accommodate differential movements due to shrinking/swelling of expansive soil. For planning purposes, an average allowable bearing pressure of 1,000 psf can be considered for dead-plus-live loads on engineered fill.

4.7.3 Slab Moisture Vapor Reduction

When buildings are constructed with mats, water vapor from beneath the mat will migrate through the foundation and into the building. This water vapor can be reduced but not eliminated. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. Where water vapor migrating through the mat would be undesirable, we recommend the following measures to reduce water vapor transmission upward through the mat foundations.

1. Install a vapor retarder membrane directly beneath the mat. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E1745 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."
2. Concrete should have a concrete water-cement ratio of no more than 0.5.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
4. Consider and implement adequate moist cure procedures for mat foundations.
5. Protect foundation subgrade soil from seepage by providing impermeable plugs within utility trenches.

The structural engineer may consider use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed below the vapor retarder membrane to assist in concrete curing.

4.8 BIORETENTION IMPROVEMENTS CONSIDERATIONS

If bioretention areas are incorporated in design, the infiltration features should be planned a minimum of 5 feet away from surface improvements and structures, including pavement, flatwork, and slab-on-grade. When this setback distance is not practical, at a minimum, the designer of the infiltration improvements may consider the following.

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

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

4.9 PRELIMINARY PAVEMENT RECOMMENDATION

4.9.1 Flexible Pavements

Based on the site soil, we estimate a Resistance (R-Value) of 5 is appropriate for preliminary design. The design sections may be reduced based on R-Value testing of site-specific samples collected from areas that are planned for pavement improvements. Using estimated traffic indices, we developed the following recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in Table 4.9.1-1 below. During a design-level study, the project civil engineer should provide suitable traffic indices for planned roadways.

TABLE 4.9.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION BASED ON R-VALUE 5	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	10
6	3½	13
7	4	16

Note: AB is Class 2 aggregate base material with a minimum R-value of 78

Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, civil engineer, and appropriate public agency.

4.9.2 Rigid Pavements

Concrete pavement sections may be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. 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.
- Concrete pavement should have a minimum 28-day compressive strength of 3,000 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

4.9.3 Subgrade and Aggregate Base Compaction

The contractor should compact finish subgrade and aggregate base in accordance with the design-level geotechnical report. In general, aggregate base (AB) materials should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

5.0 DESIGN-LEVEL GEOTECHNICAL STUDY

As previously discussed, this report presents our preliminary geotechnical findings, conclusions and recommendations intended for planning purposes only. To support future proposed development, a design-level geotechnical exploration and assessment should be performed when development plans are available. The exploration will also allow for more detailed evaluations of the geotechnical issues discussed in this report and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

We recommend the design-level exploration and reporting include the following scope items.

- Borings, including matched-pair borings located immediately adjacent to CPTs performed as part of this study.

- Laboratory testing including but not limited to, moisture content, unit weight, gradation, Atterberg Limits, strength, and corrosivity testing.
- Design-level assessment of geologic and geotechnical hazards, including, but not limited to:
    - Characterization of subsurface conditions.
    - Static load-induced settlement based on structural loading.
- Design recommendations for foundation system design
- Foundation constructability recommendations
- Design-level earthwork and improvement design and construction recommendations

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents our preliminary geotechnical exploration for the site discussed in Section 1.2. 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 strive to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, either express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

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

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

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

SELECTED REFERENCES

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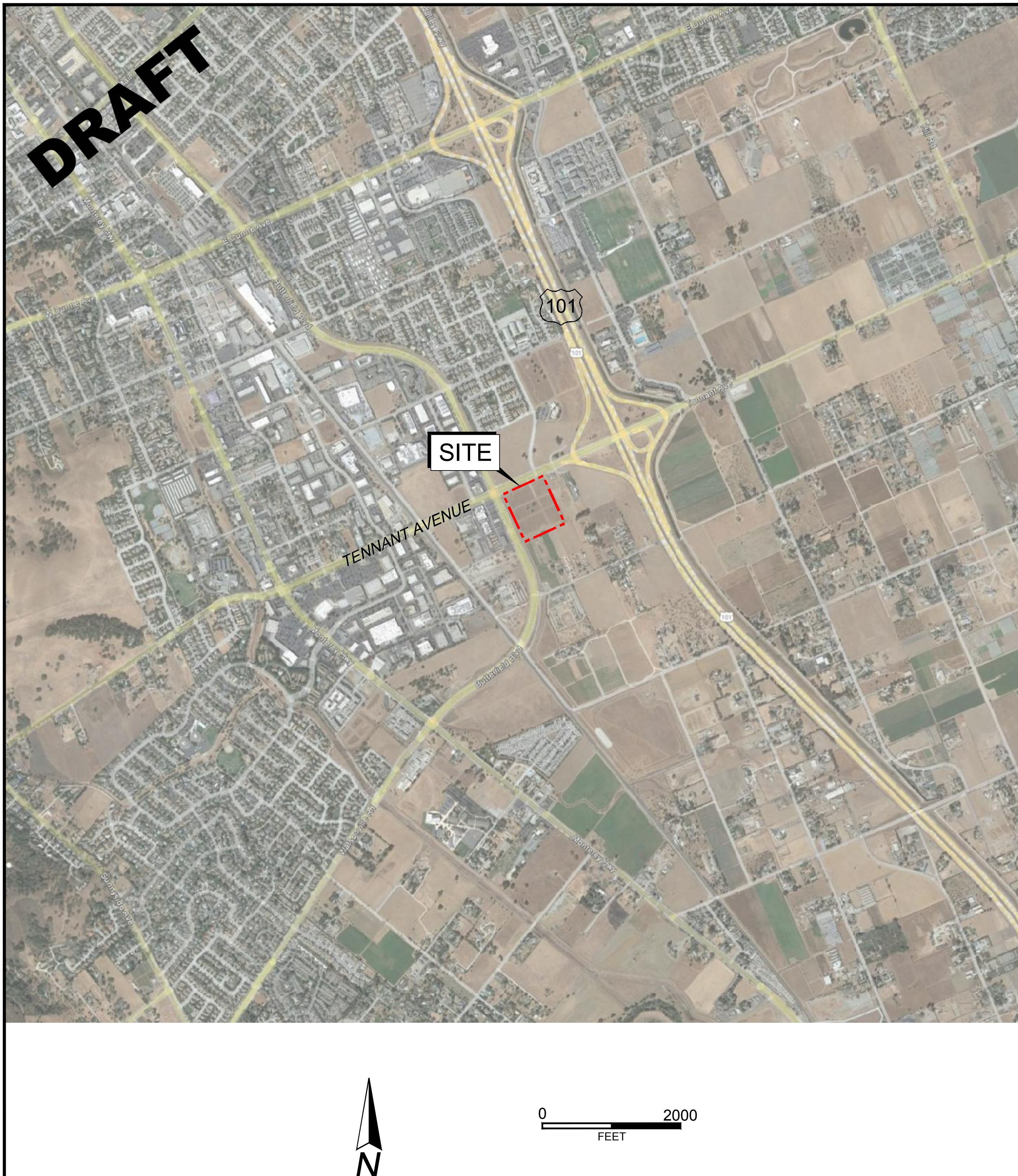
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP 740 TENNANT AVENUE MORGAN HILL, CALIFORNIA	PROJECT NO.: 23556.000.001 SCALE: AS SHOWN DRAWN BY: CC CHECKED BY: JTR	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

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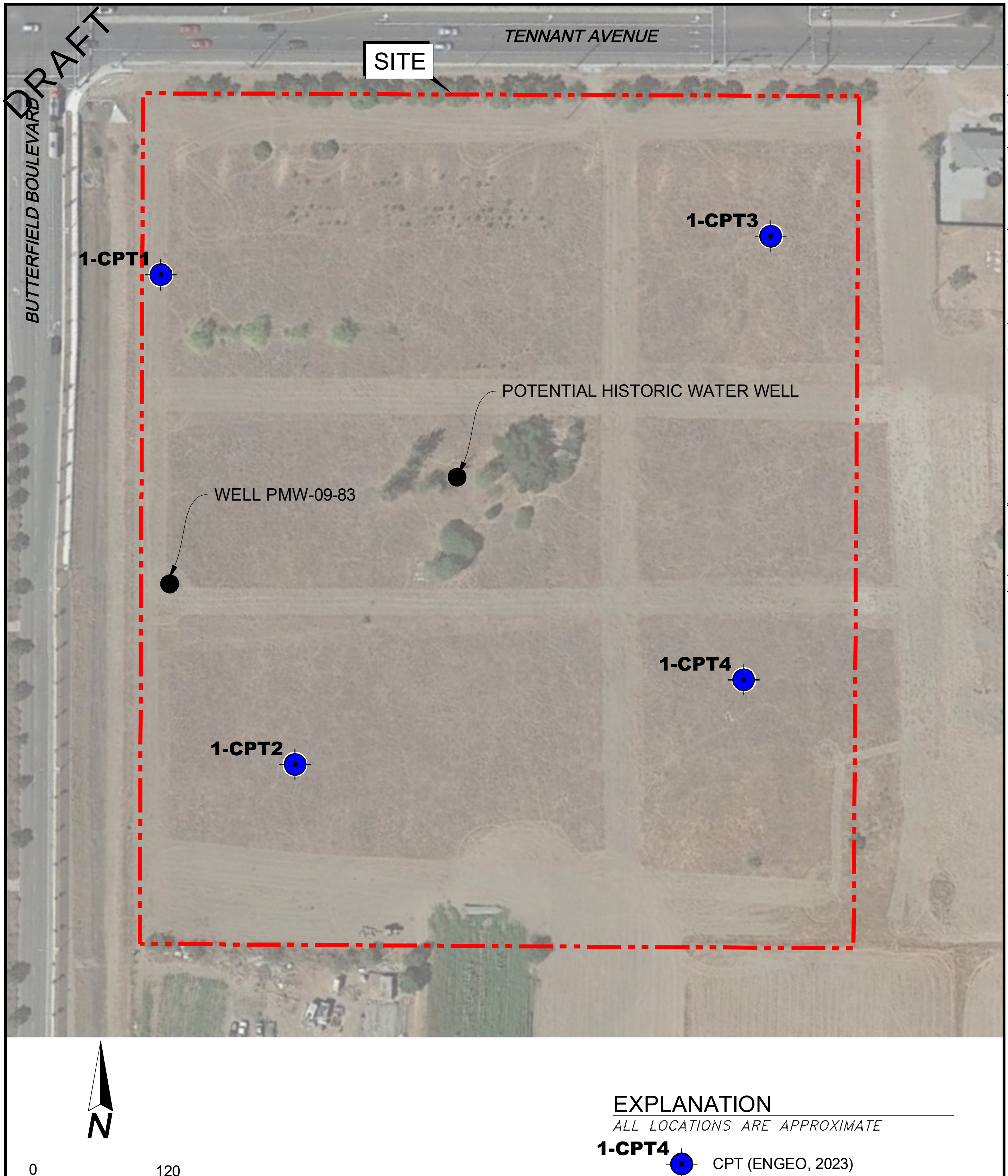

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT4 CPT (ENGEO, 2023)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	SITE PLAN 740 TENNANT AVENUE MORGAN HILL, CALIFORNIA	PROJECT NO.: 23556.000.001 SCALE: AS SHOWN DRAWN BY: CC CHECKED BY: JTR	FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR

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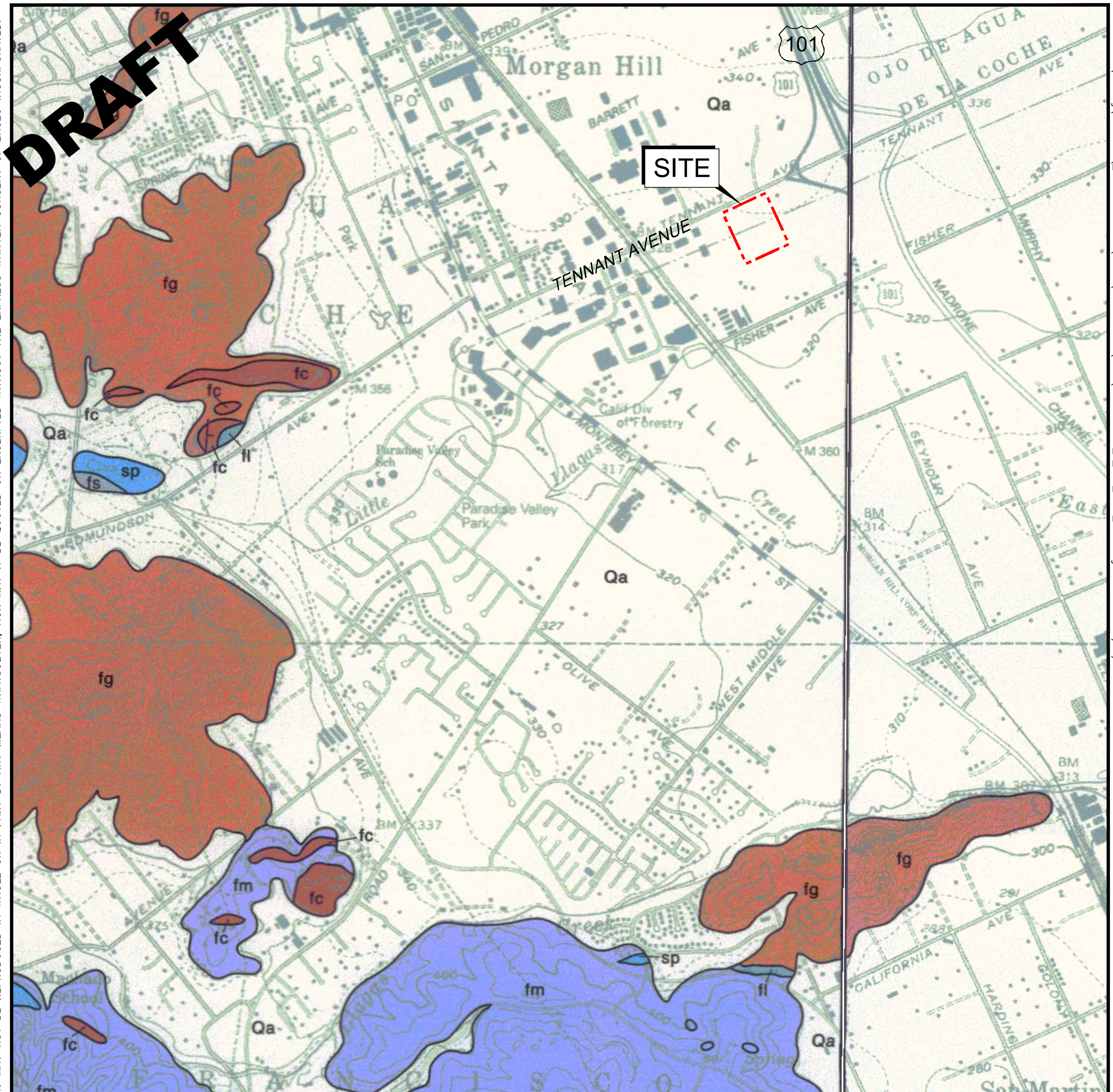


EXPLANATION

——— – – –	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED	Qa	ALLUVIUM
		sp	SERPENTINITE
—▲—▲– – – · ·	FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT	fm	MELANGE
		fs	CHERT
		fg	GREENSTONE

N

0 2000
FEET

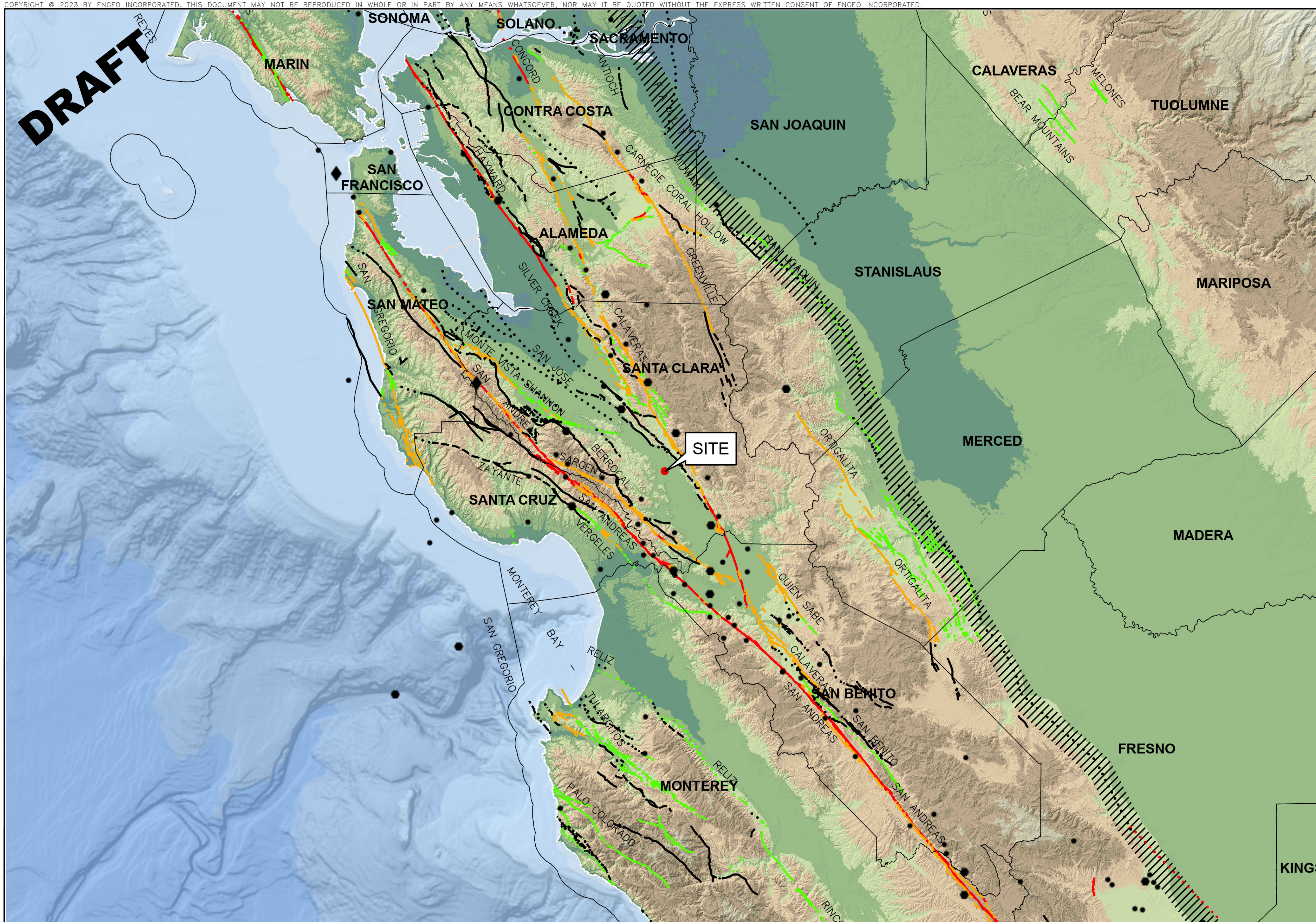
BASE MAP SOURCE: DIBBLEE, 2005

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
740 TENNANT AVENUE
MORGAN HILL, CALIFORNIA

PROJECT NO.: 23556.000.001	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: CC	CHECKED BY: JTR

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

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

QUATERNARY FAULTS 2020

BASED ON TIME OF MOST RECENT SURFACE DEFORMATION

- HISTORICAL (<150 YEARS), WELL CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), MODERATELY CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), INFERRED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), WELL CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), INFERRED LOCATION
- LATE QUATERNARY (<130,000 YEARS), WELL CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), INFERRED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), WELL CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), MODERATELY CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), INFERRED LOCATION
- GREAT VALLEY FAULT ZONE

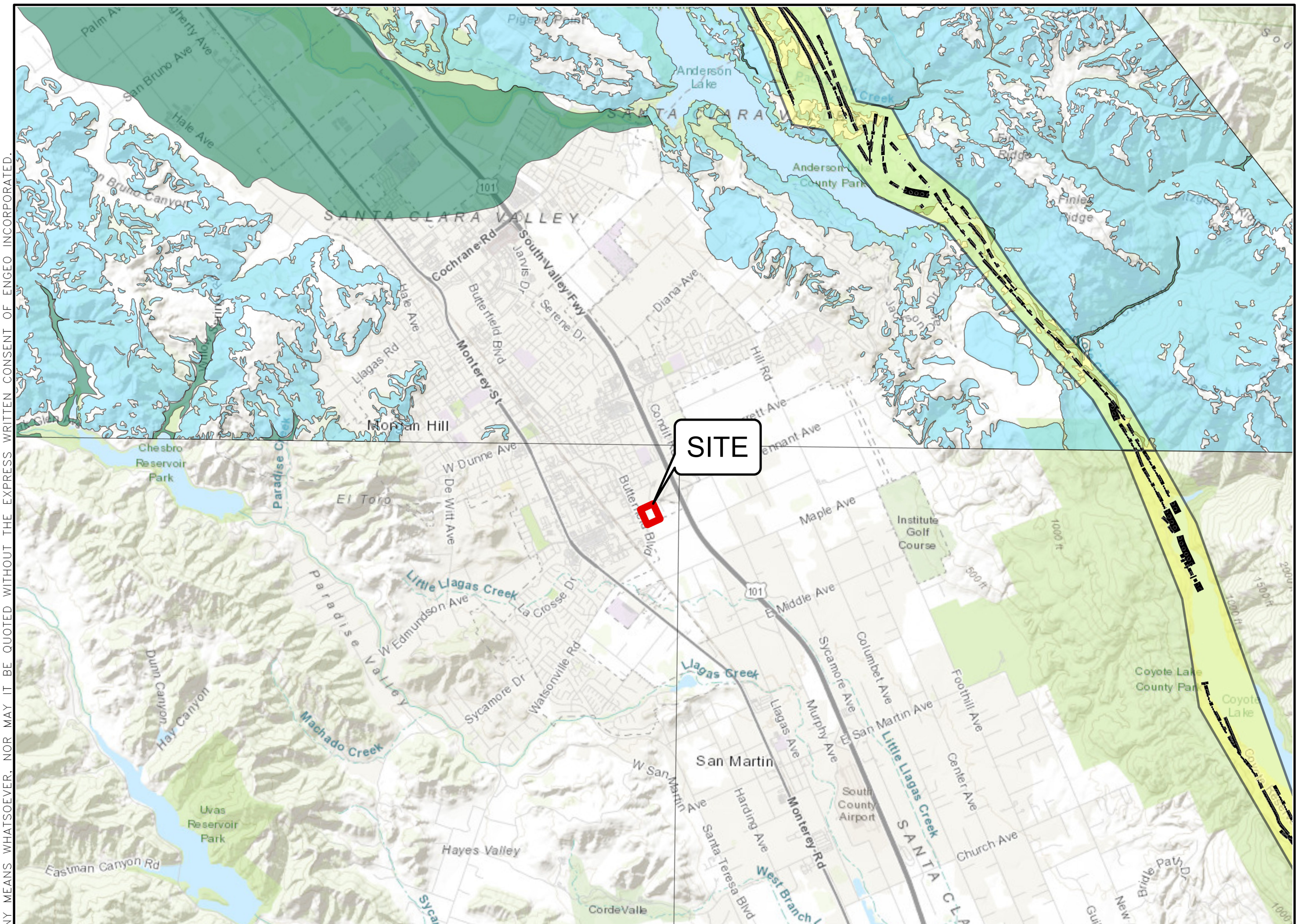
BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)
U.S.G.S OPEN-FILE REPORT 96-705

ENGEO Expect Excellence

REGIONAL FAULTING AND SEISMICITY MAP 740 TENNANT AVENUE MORGAN HILL, CALIFORNIA	PROJECT NO.: 23556.000.001	FIGURE NO. 4
	SCALE: AS SHOWN	
	DRAWN BY: CC / CHECKED BY: JTR	

PATH: G:\DRAFTING\PROJECTS_22000 TO 23999\23556\23556000001\PGEX\23556000001_PGEX_0723.APRX
LAYOUT: FAULT&SEISMICITY - 2020 USER: CCULBERTSON

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

ACCURATELY LOCATED

INFERRED

APPROXIMATELY LOCATED

CONCEALED

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

EARTHQUAKE-INDUCED LANDSLIDE ZONE
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 GROUNDWATER CONDITIONS INDICATE A POTENTIAL L FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

N

0 4,000 8,000

FEET

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

ENGEO
Expect Excellence

SEISMIC HAZARDS ZONE MAP

740 TENNANT AVENUE

MORGAN HILL MORGAN HILL, CALIFORNIA

PROJECT NO. : 23556.000.001	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: JTR

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LAYOUT: SEISMIC HAZARD USER: NLAMOTTEKERR

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

CPT EXPLORATION LOGS

PRESENTATION OF SITE INVESTIGATION RESULTS

740 Tennant Ave

Prepared for:

ENGEO

ConeTec Job No: 23-56-26119

Project Start Date: 2023-Jun-28
Project End Date: 2023-Jun-28
Report Date: 2023-Jun-29

Prepared by:

ConeTec Inc.

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Tel: (510) 357-3677

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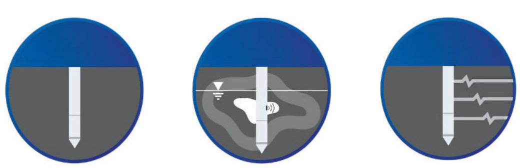
www.conetec.com
www.conetecdataservices.com



ABOUT THIS REPORT

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

Project Information	
Client	ENGEO
Project	740 Tennant Ave
ConeTec Project Number	23-56-26119
Rig Description	30-ton Truck CPT Rig (C-17)

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM 10S)

Cone Penetration Test (CPTu)	
Depth Reference	Existing ground surface at the time of the investigation
Sleeve data offset	0.1 Meters

Calculated Geotechnical Parameters 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).

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

LIMITATIONS

3rd Party Disclaimer

- The "Report" refers to this report titled 740 Tennant Ave
- The Report was prepared by ConeTec for ENGEO

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

Client Disclaimer

- ConeTec was retained by ENGEO
- The "Report" refers to this report titled 740 Tennant Ave
- ConeTec was retained to collect and provide the raw data ("Data") which is included in the Report.

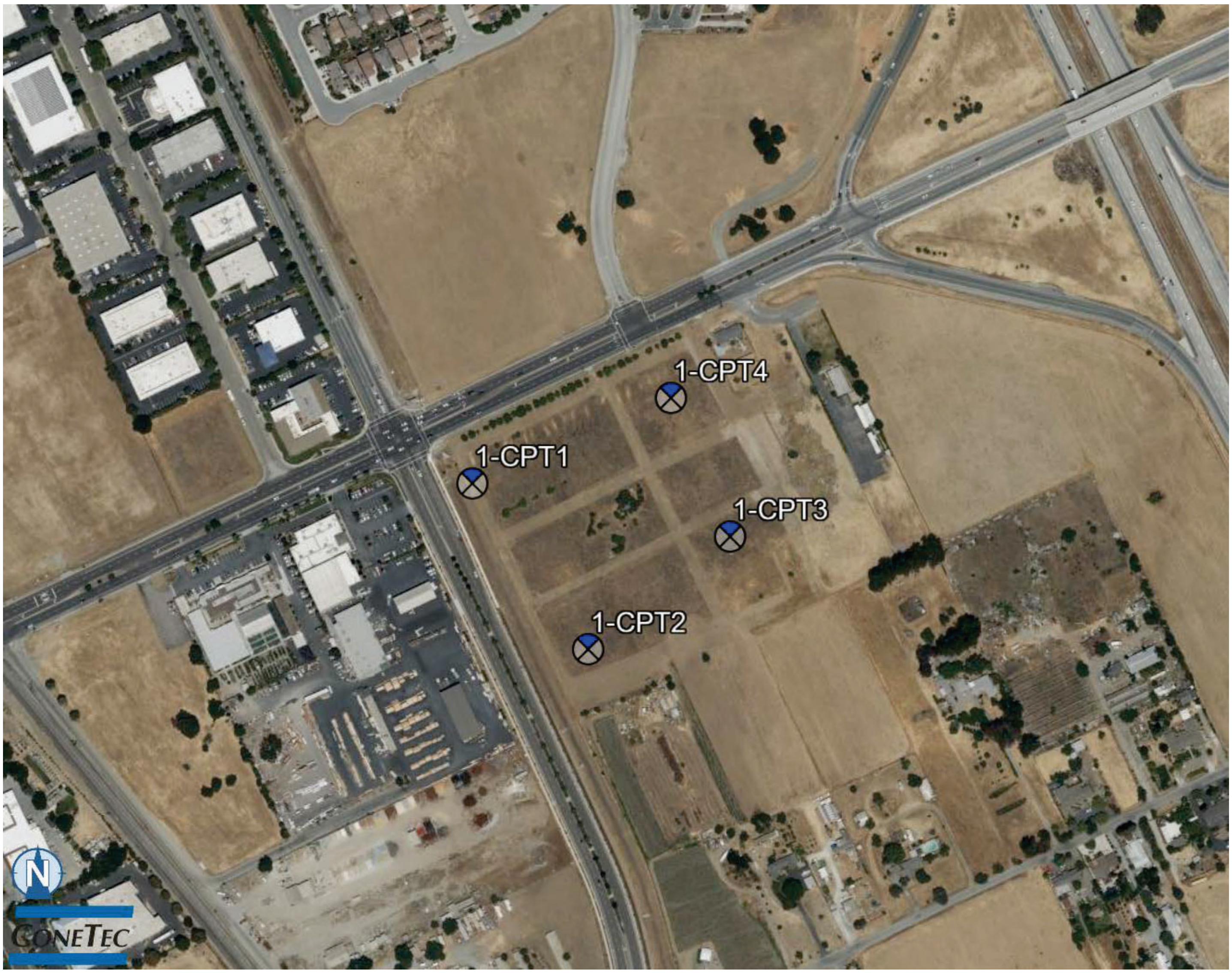
ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other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

CONTENTS

The following listed below are included in the report:

- **Site Map**
- **Piezocone Penetration Test (CPTu) Sounding Summary**
- **CPTu Standard Plots and Advanced Plots**
- **Pore Pressure Dissipation (PPD) Test Summary**
- **PPD Test Plots**
- **Methodology Statements**
- **Data File Formats**
- **Description of Methods for Calculated CPT Geotechnical Parameters**

SITE MAP



ConeTec Job Number: 23-56-26119

Client: ENGEO

Project: 740 Tennant Ave

Report Date: 2023-Jun-29

Sounding Location

All sounding locations are approximate

CONETEC

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

ConeTec

Job No:	23-56-26119
Client:	ENGEO
Project:	740 Tennant Ave
Start Date:	28-Jun-2023
End Date:	28-Jun-2023

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Rig	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
1-CPT1	23-56-26119_CP01	28-Jun-2023	EC795:T1500F15U35	C-17	15	17.0	44.45	4108686	621551	333	4
1-CPT2	23-56-26119_CP02	28-Jun-2023	EC795:T1500F15U35	C-17	15	13.3	44.45	4108560	621642	330	
1-CPT3	23-56-26119_Cp03	28-Jun-2023	EC795:T1500F15U35	C-17	15	19.8	44.45	4108648	621750	331	
1-CPT4	23-56-26119_CP04	28-Jun-2023	EC795:T1500F15U35	C-17	15	>7.9	7.87	4108754	621703	332	4,5

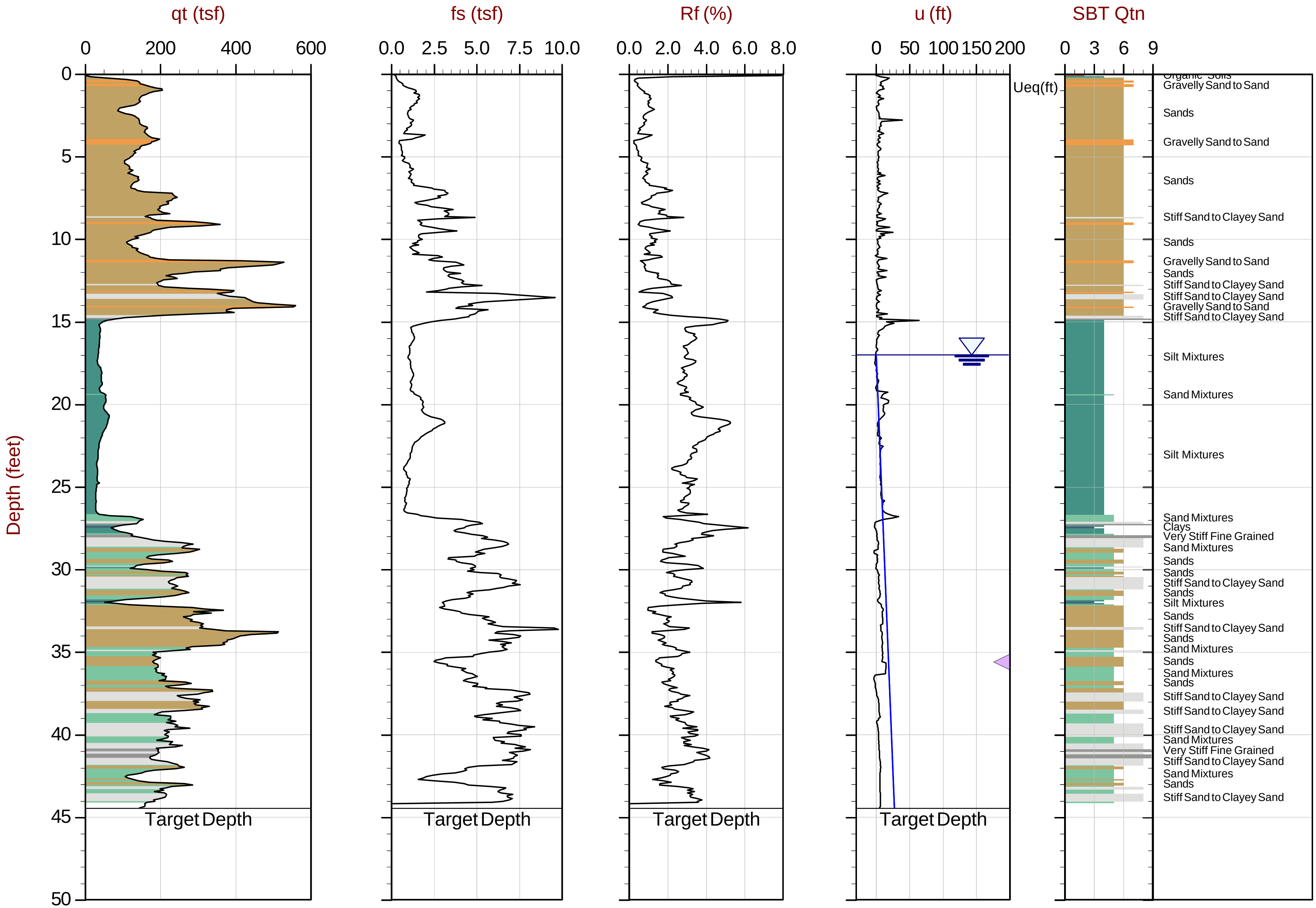
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 10S).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The phreatic surface is based on the pore pressure dissipation test at the nearby soundings, and the dynamic pore pressure within the sounding.
5. The cone broke while performing the sounding.

ConeTec

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 08:34
Site: 740 Tennant Ave

Sounding: 1-CPT1
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108686m E: 621551m

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

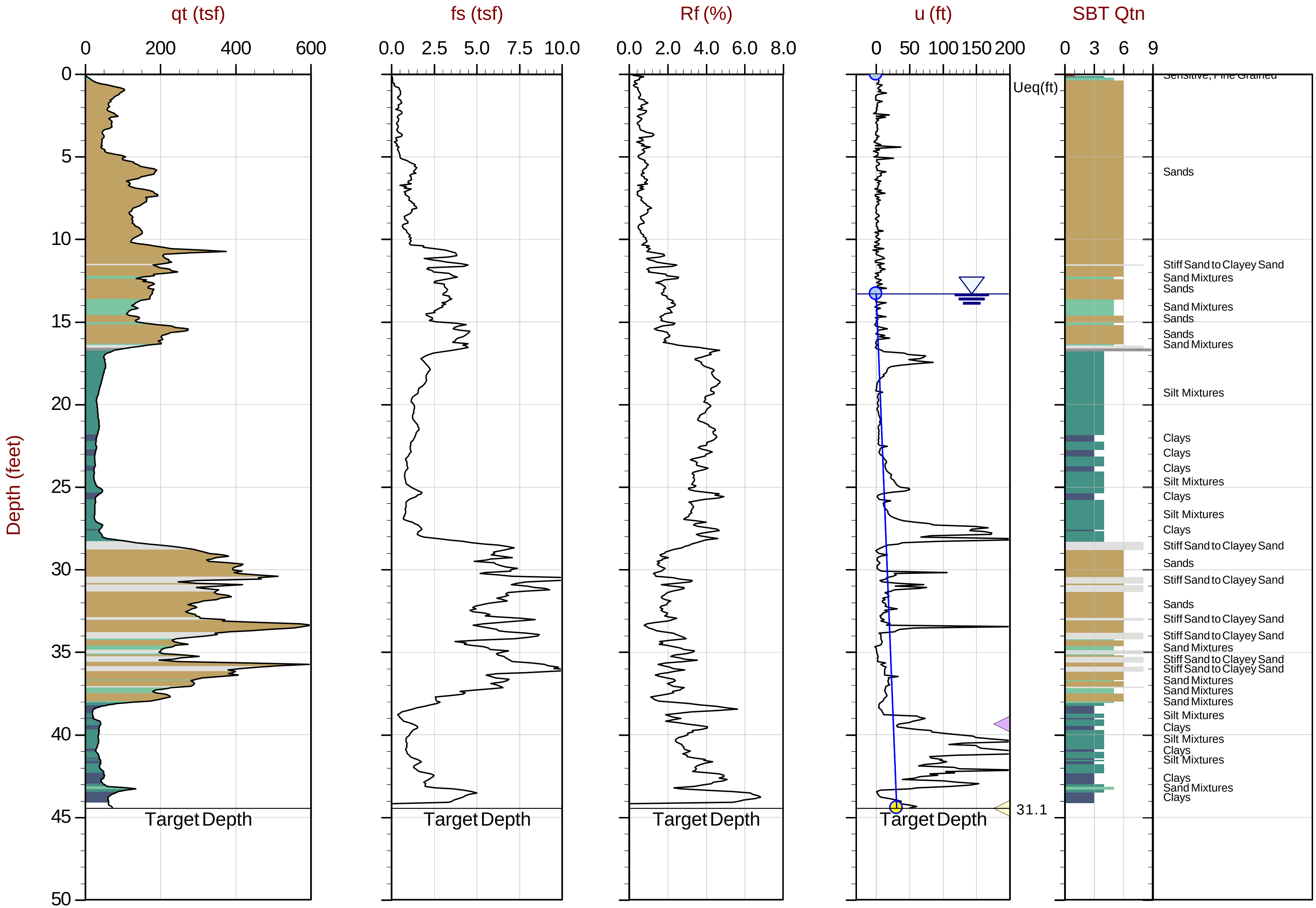
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 09:54
Site: 740 Tennant Ave

Sounding: 1-CPT2
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108560m E: 621642m

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

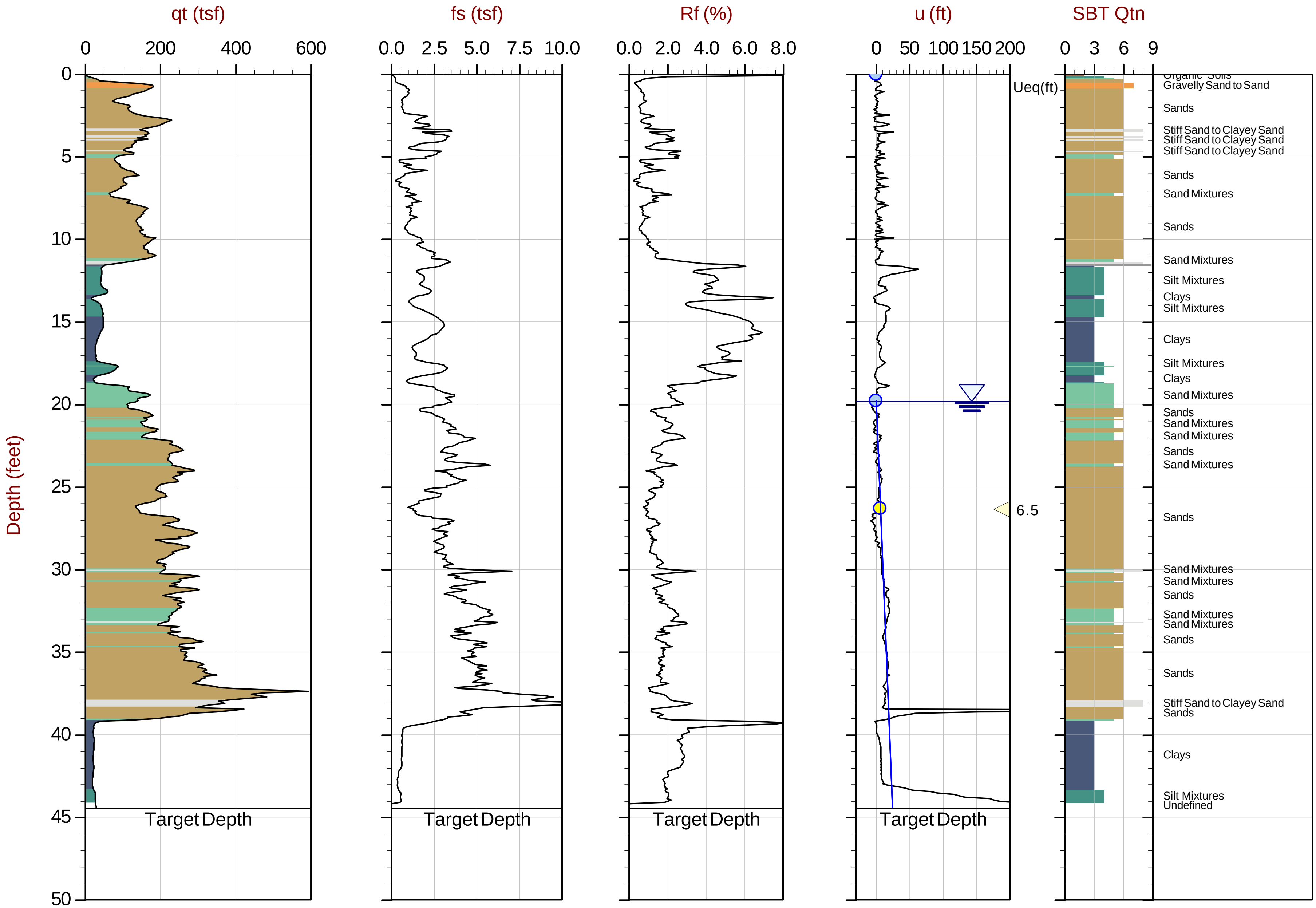
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 11:11
Site: 740 Tennant Ave

Sounding: 1-CPT3
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108648m E: 621750m

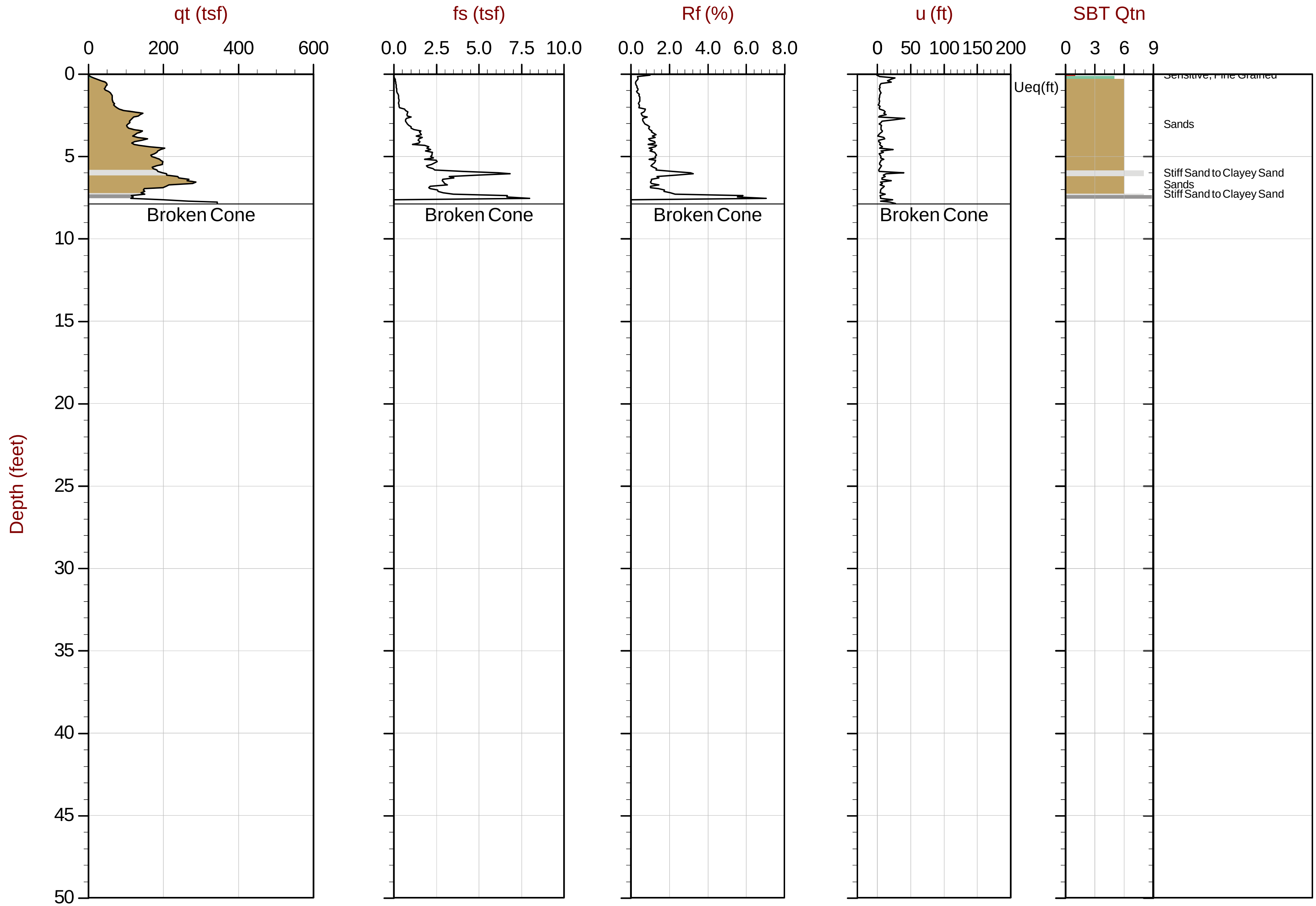
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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: 23-56-26119
Date: 2023-06-28 12:11
Site: 740 Tennant Ave

Sounding: 1-CPT4
Cone: 795:T1500F15U35



Max Depth: 2.400 m / 7.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108754m E: 621703m

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

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

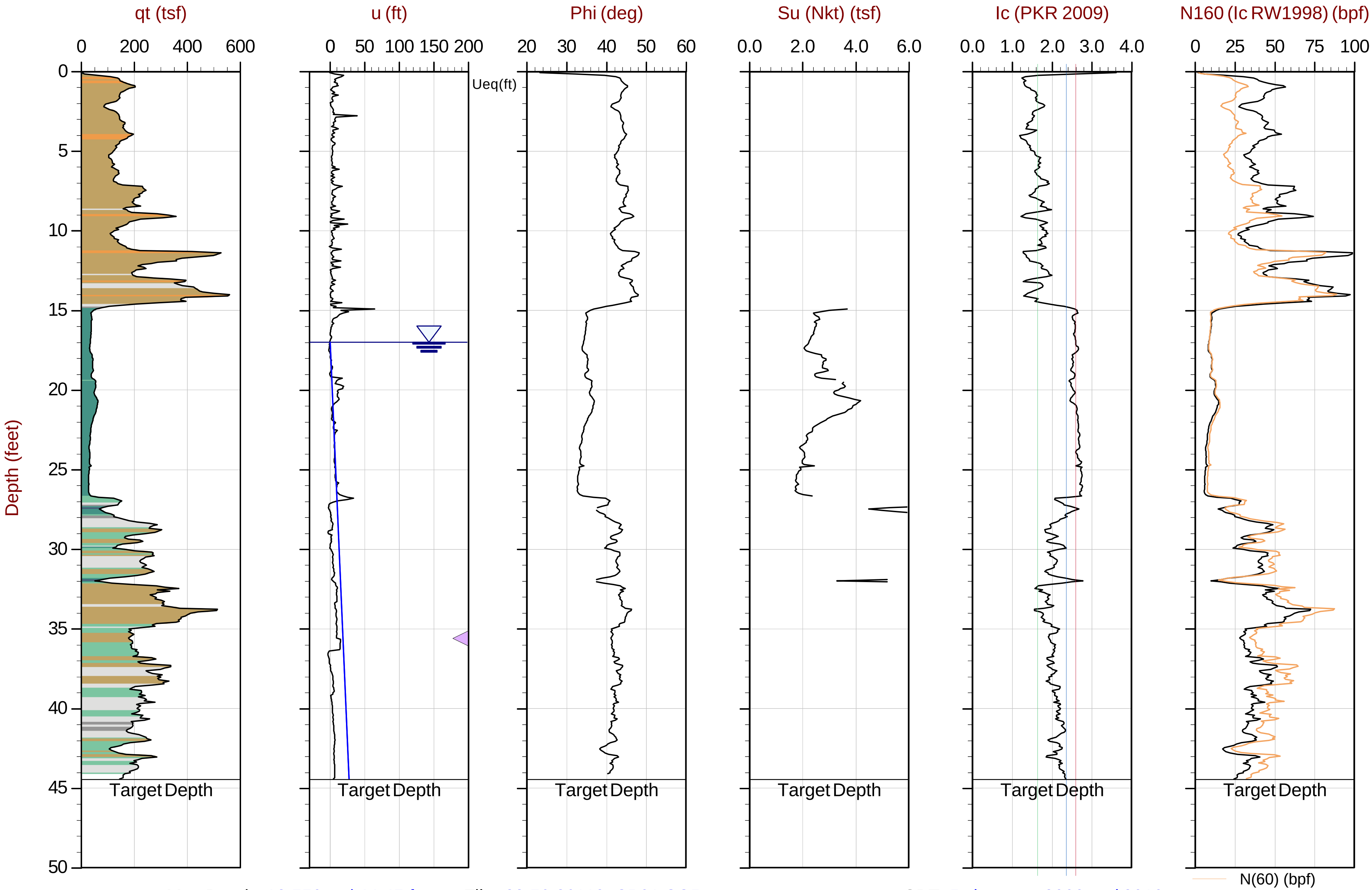
Advanced Cone Penetration Test Plots

CONETEC

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 08:34
Site: 740 Tennant Ave

Sounding: 1-CPT1
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108686m E: 621551m

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

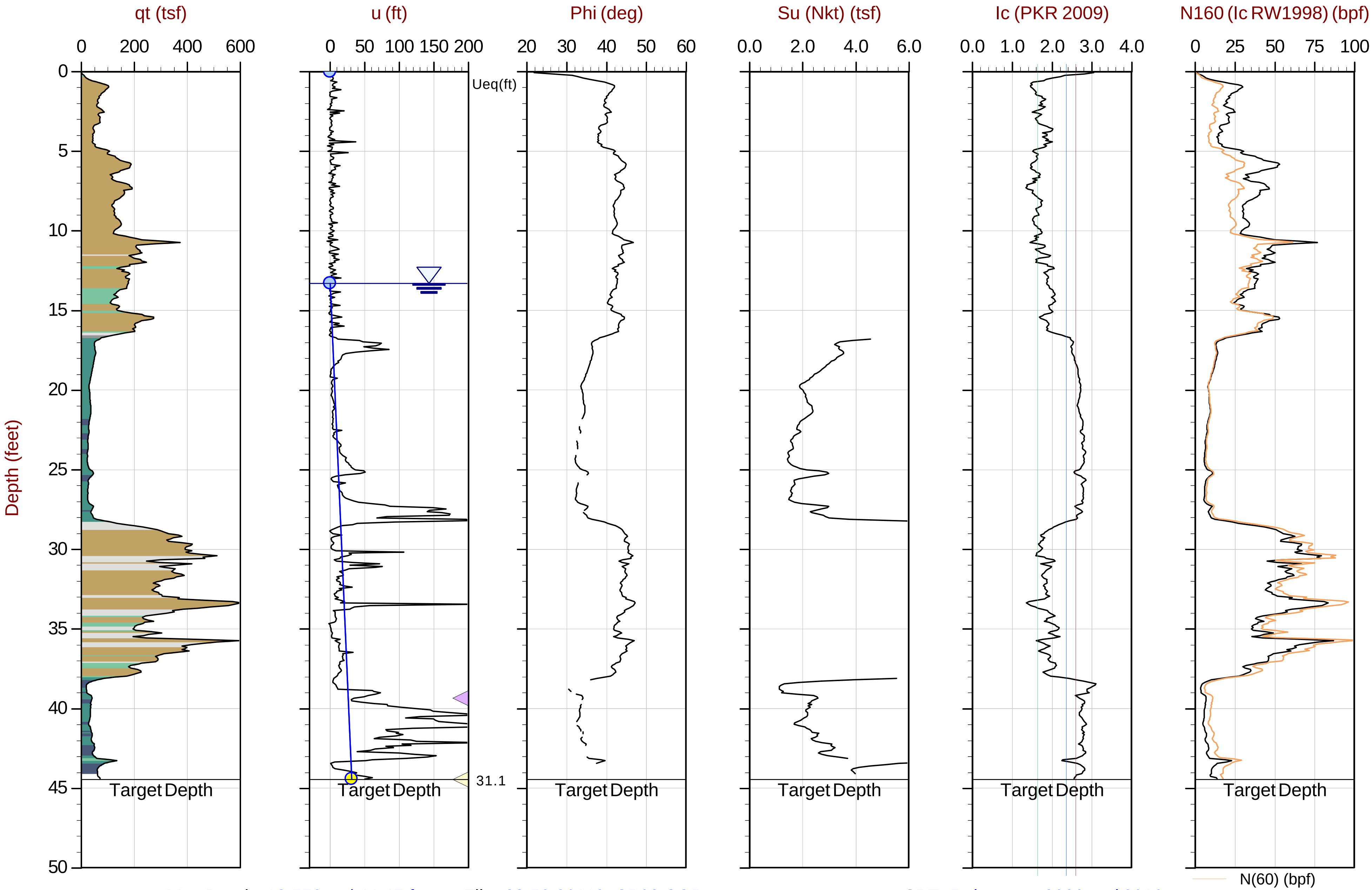
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 09:54
Site: 740 Tennant Ave

Sounding: 1-CPT2
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108560m E: 621642m

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

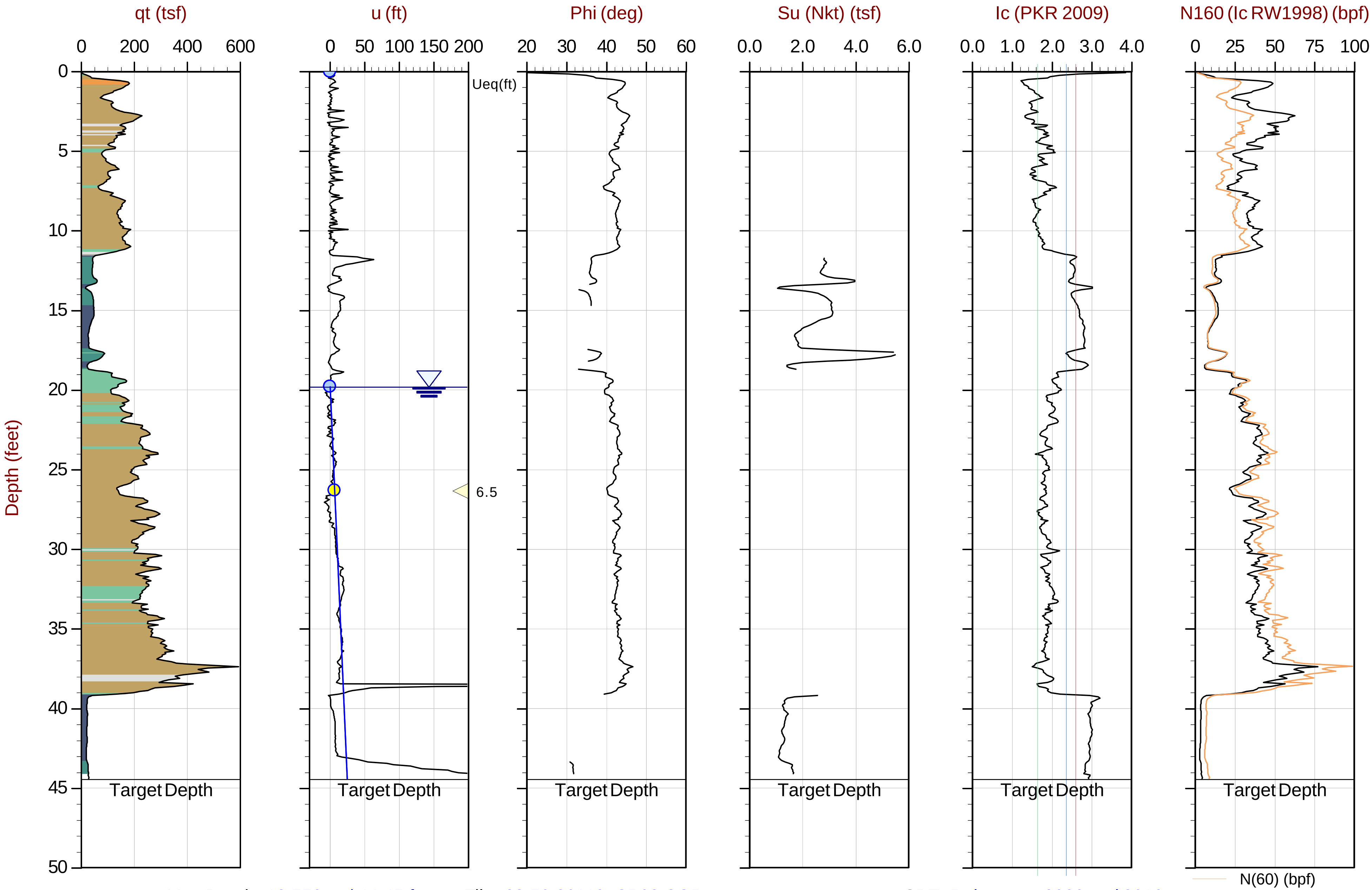
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 11:11
Site: 740 Tennant Ave

Sounding: 1-CPT3
Cone: 795:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108648m E: 621750m

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

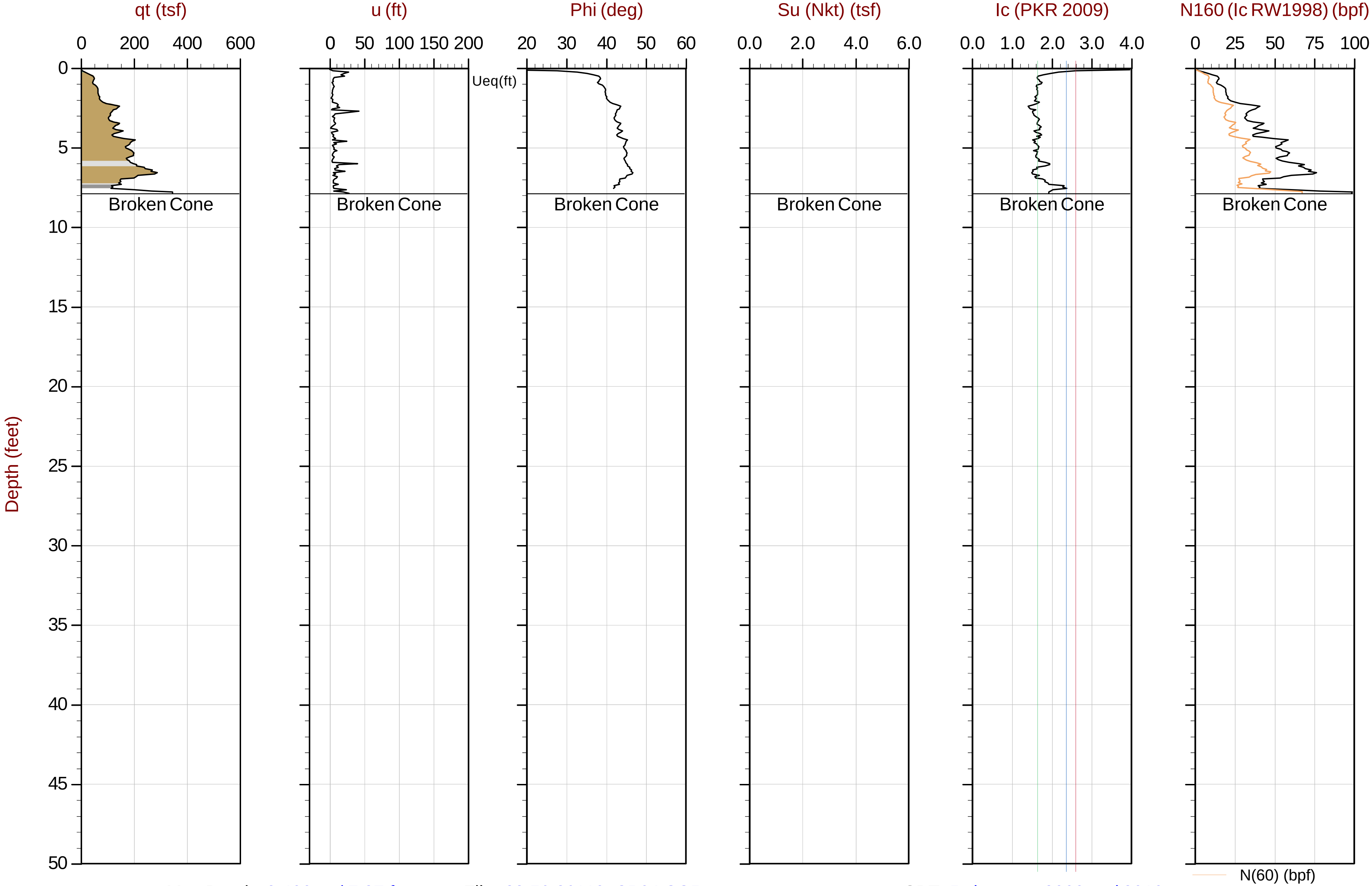
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 23-56-26119
Date: 2023-06-28 12:11
Site: 740 Tennant Ave

Sounding: 1-CPT4
Cone: 795:T1500F15U35



Max Depth: 2.400 m / 7.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-26119_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4108754m E: 621703m

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.

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	23-56-26119
Client:	ENGEO
Project:	740 Tennant Ave
Start Date:	28-Jun-2023
End Date:	28-Jun-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

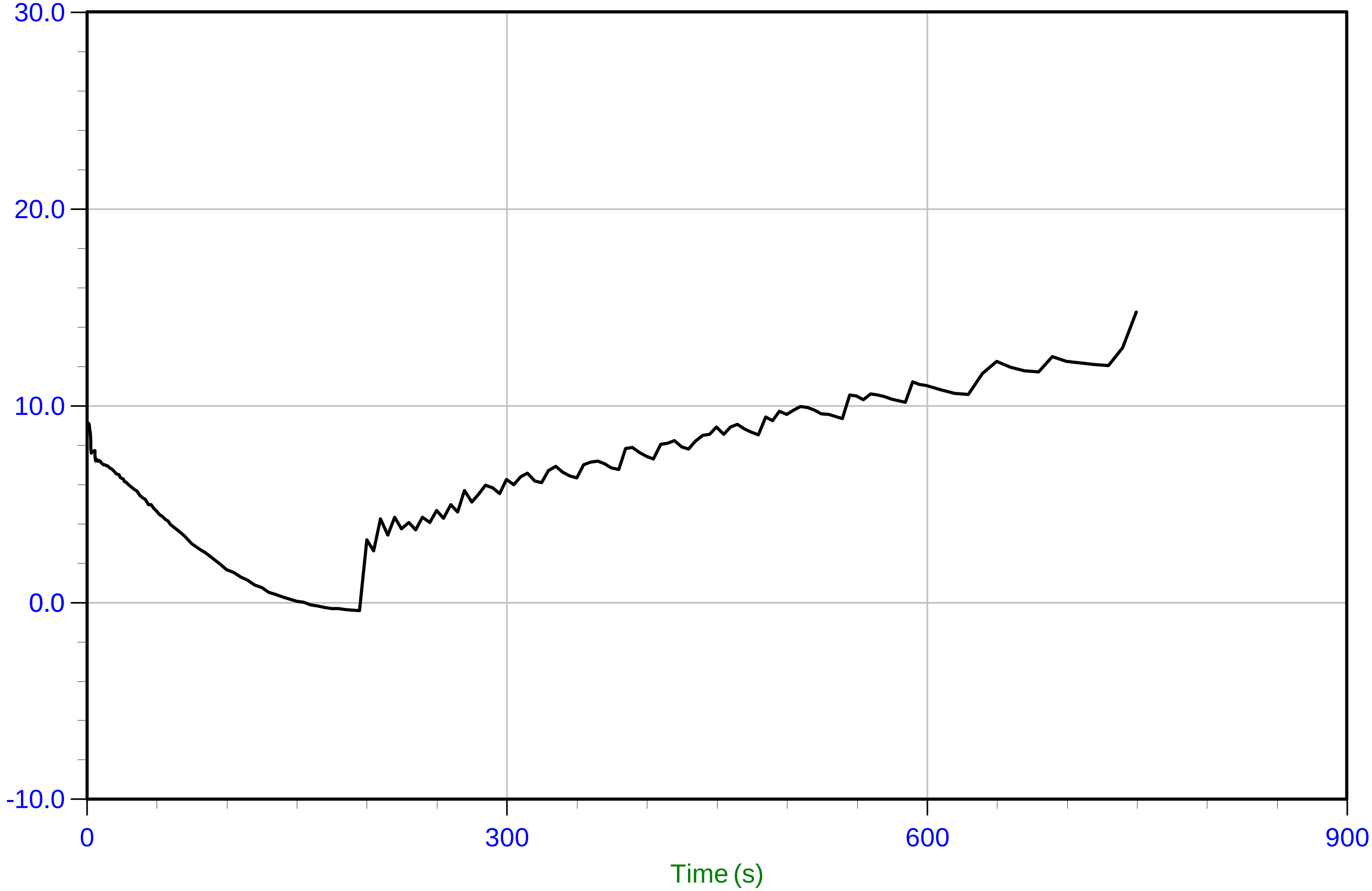
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-CPT1	23-56-26119_CP01	15	750	35.60	Not Achieved	
1-CPT2	23-56-26119_CP02	15	520	39.37	Not Achieved	
1-CPT2	23-56-26119_CP02	15	1020	44.45	31.2	13.3
1-CPT3	23-56-26119_CP03	15	810	26.33	6.5	19.8

CONETEC

ENGEO

Job No: 23-56-26119
Date: 06/28/2023 08:34
Site: 740 Tennant Ave

Sounding: 1-CPT1
Cone: 795:T1500F15U35 Area=15 cm²

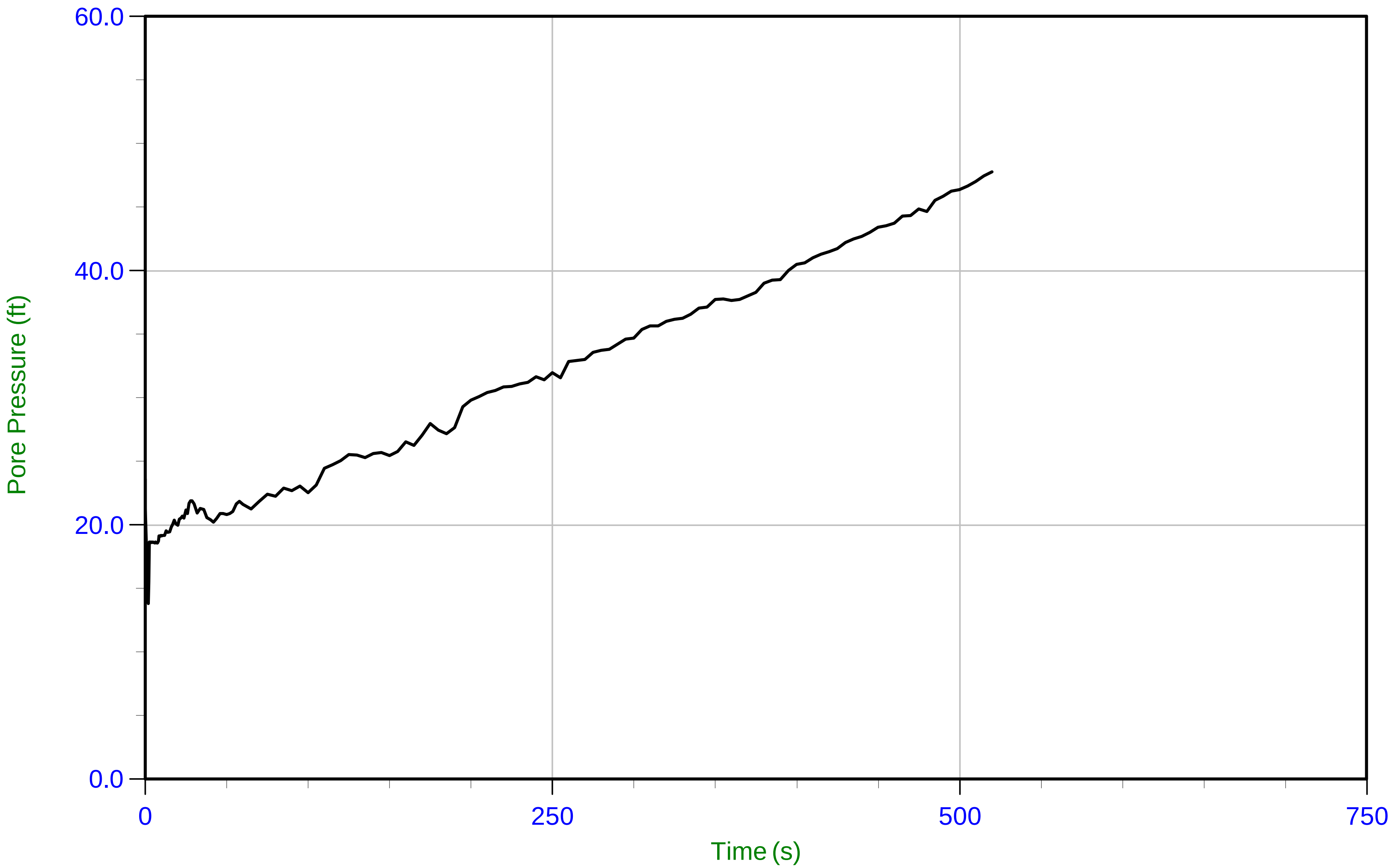


Trace Summary:

Filename: 23-56-26119_CP01.ppf2
Depth: 10.850 m / 35.597 ft
Duration: 750.0 s

u Min: -0.4 ft
u Max: 14.8 ft
u Final: 14.8 ft

ENGEO	Job No: 23-56-26119 Date: 06/28/2023 09:54 Site: 740 Tennant Ave	Sounding: 1-CPT2 Cone: 795:T1500F15U35 Area=15 cm²

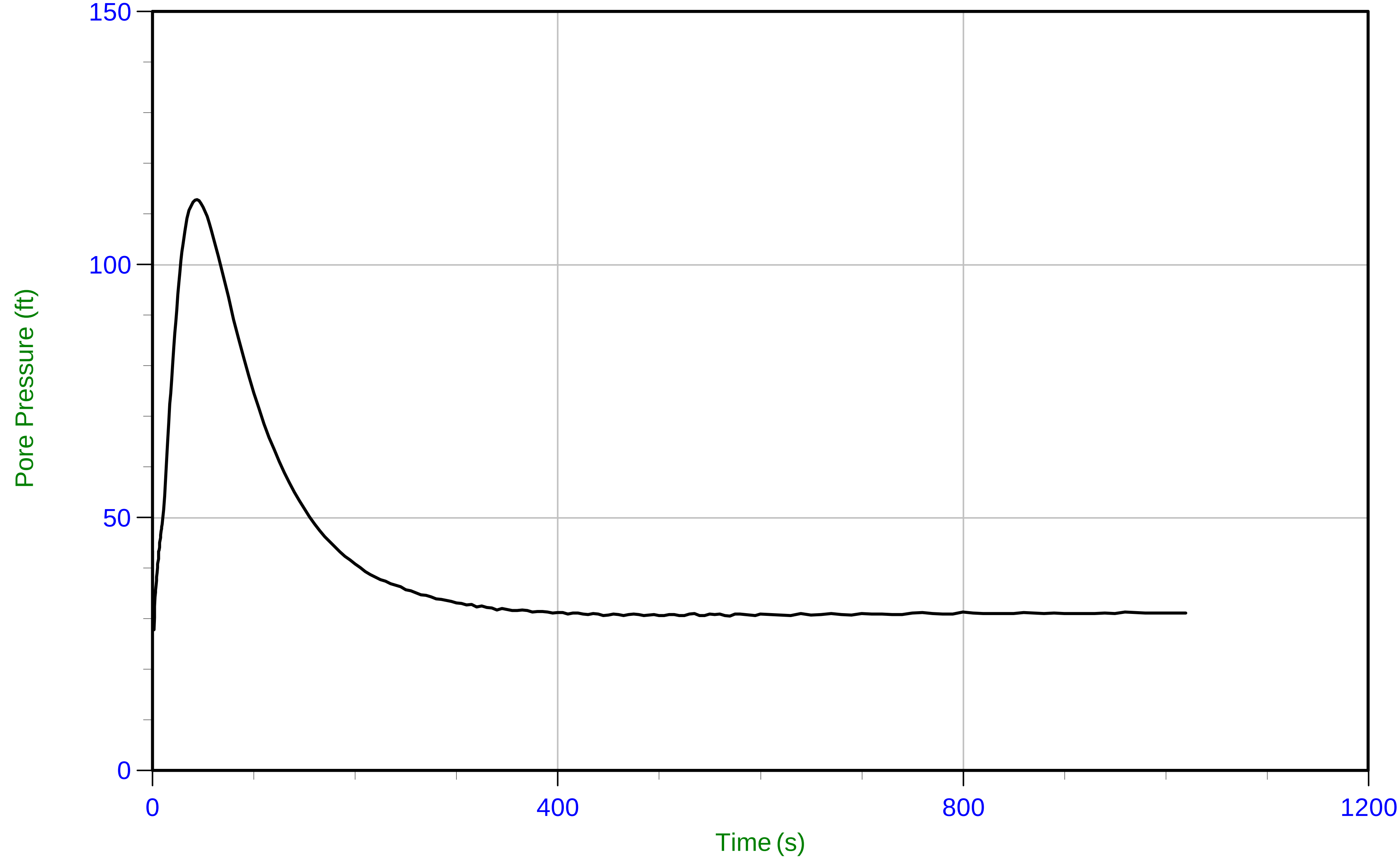


Trace Summary:

Filename: 23-56-26119_CP02.ppf2
Depth: 12.000 m / 39.370 ft
Duration: 520.0 s

u Min: 13.8 ft
u Max: 47.8 ft
u Final: 47.8 ft

CONETEC	ENGEO	Job No: 23-56-26119 Date: 06/28/2023 09:54 Site: 740 Tennant Ave	Sounding: 1-CPT2 Cone: 795:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-56-26119_CP02.ppf2
Depth: 13.550 m / 44.455 ft
Duration: 1020.0 s

u Min: 27.8 ft
u Max: 112.9 ft
u Final: 31.2 ft

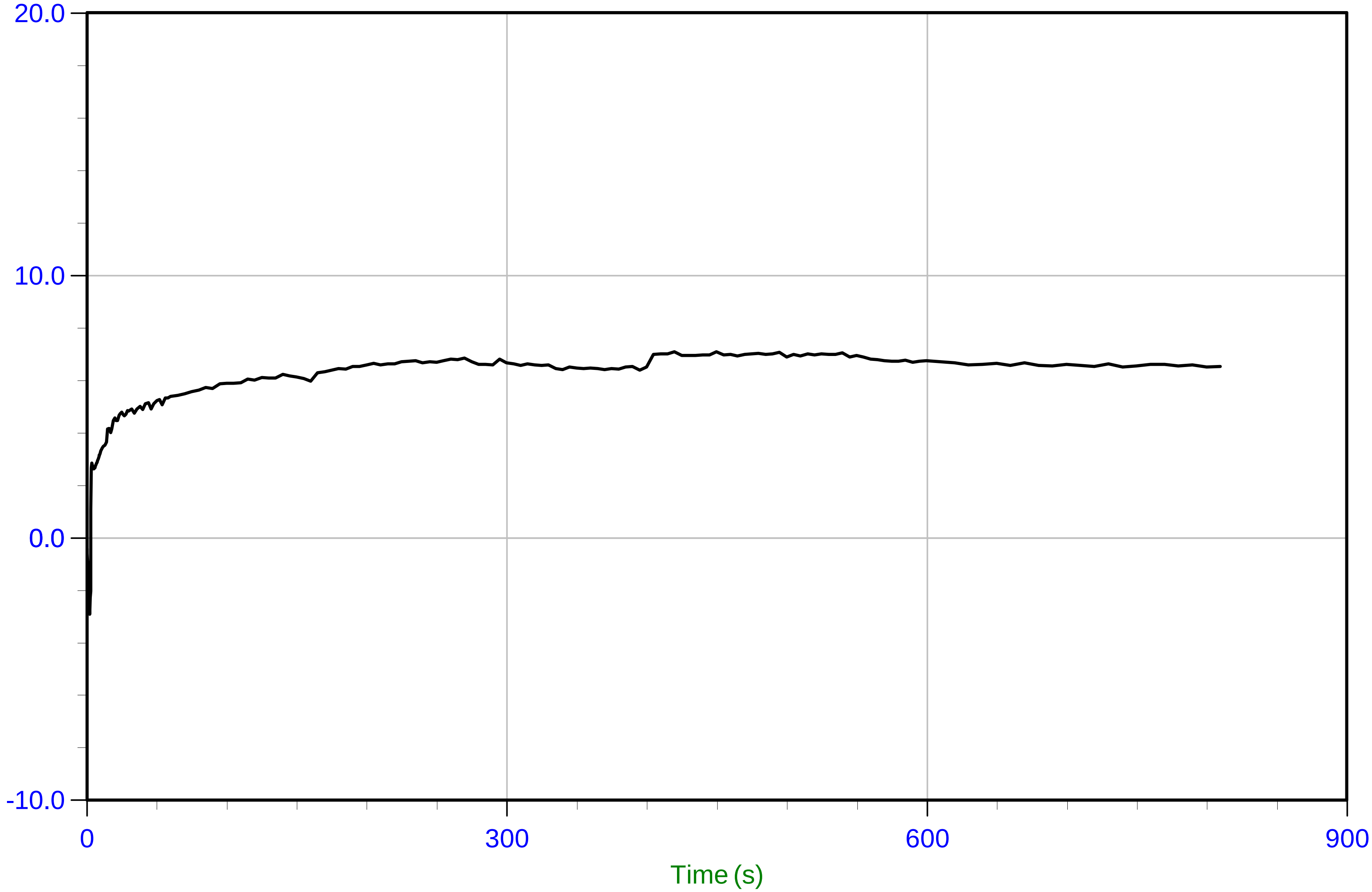
WT: 4.053 m / 13.299 ft
Ueq: 31.2 ft

CONETEC

ENGEO

Job No: 23-56-26119
Date: 06/28/2023 11:11
Site: 740 Tennant Ave

Sounding: 1-CPT3
Cone: 795:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-56-26119_CP03.ppf2
Depth: 8.025 m / 26.328 ft
Duration: 810.0 s

u Min: -2.9 ft
u Max: 7.1 ft
u Final: 6.5 ft

WT: 6.039 m / 19.813 ft
Ueq: 6.5 ft

Methodology Statements and Data File Formats

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

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

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

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

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

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

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

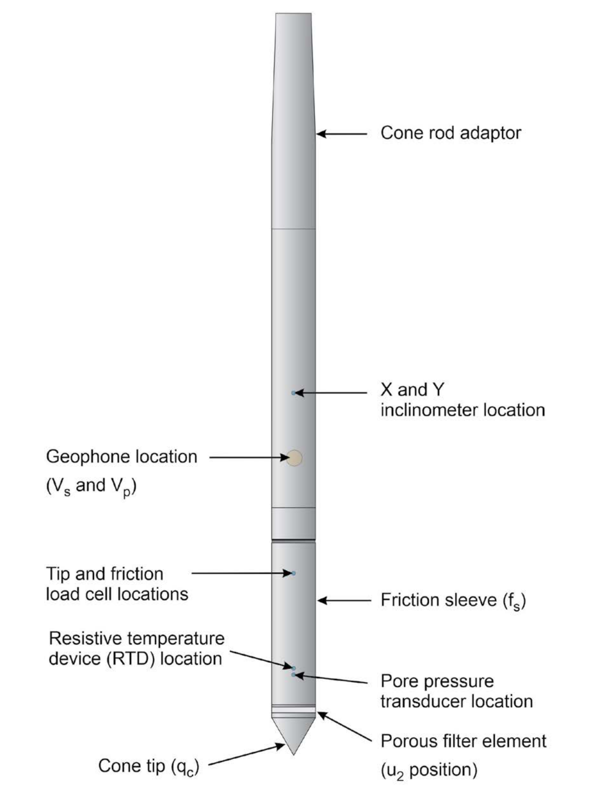


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

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

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

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

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

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

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

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

Non-normalized Classification Chart - Robertson 2010

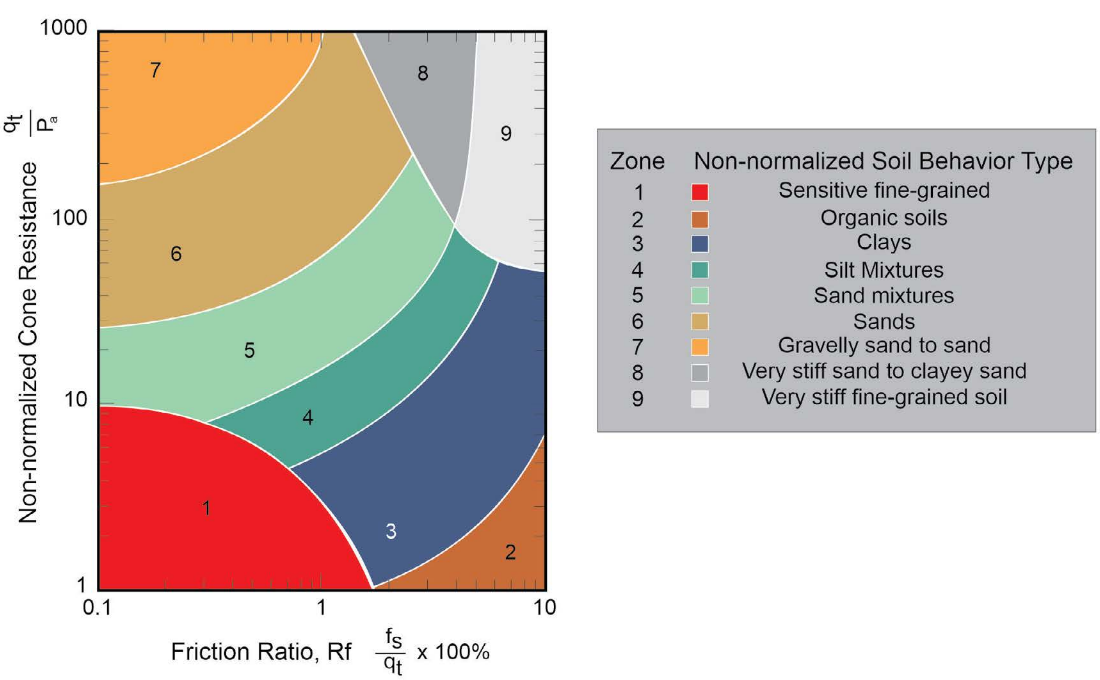


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

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

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

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

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

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

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

REFERENCES

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

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

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

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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

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

PORE PRESSURE DISSIPATION TEST

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

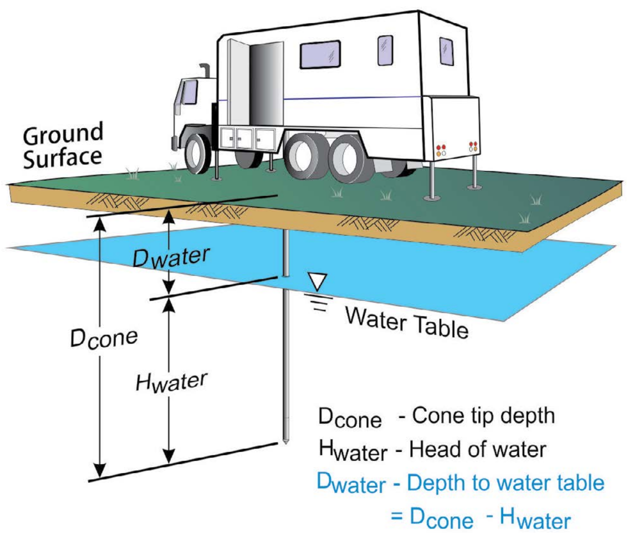


Figure PPD-1. Pore pressure dissipation test setup

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

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

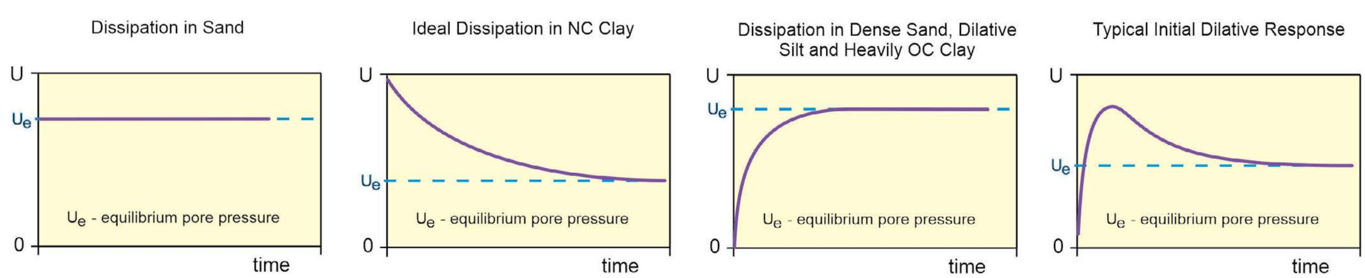


Figure PPD-2. Pore pressure dissipation curve examples

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

CPT Data Files (COR Extension)

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

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

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

Header Lines

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

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

Data Records

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

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

End of Data Marker

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

Units Information

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

CPT Data Files (XLS Extension)

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

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

CPT Dissipation Files (XLS Extension)

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

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

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

Data Records

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

Cone Type Designations

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

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023

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

Limitations

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

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

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

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

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

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

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

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

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

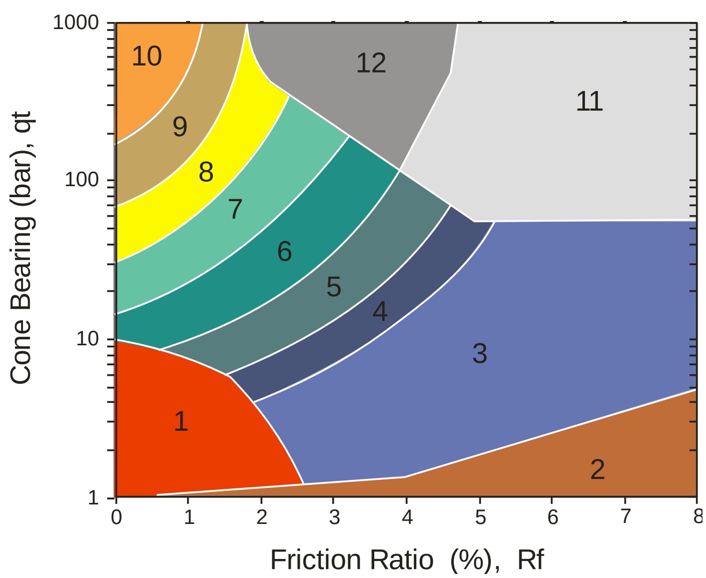
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from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

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

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

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

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



$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

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

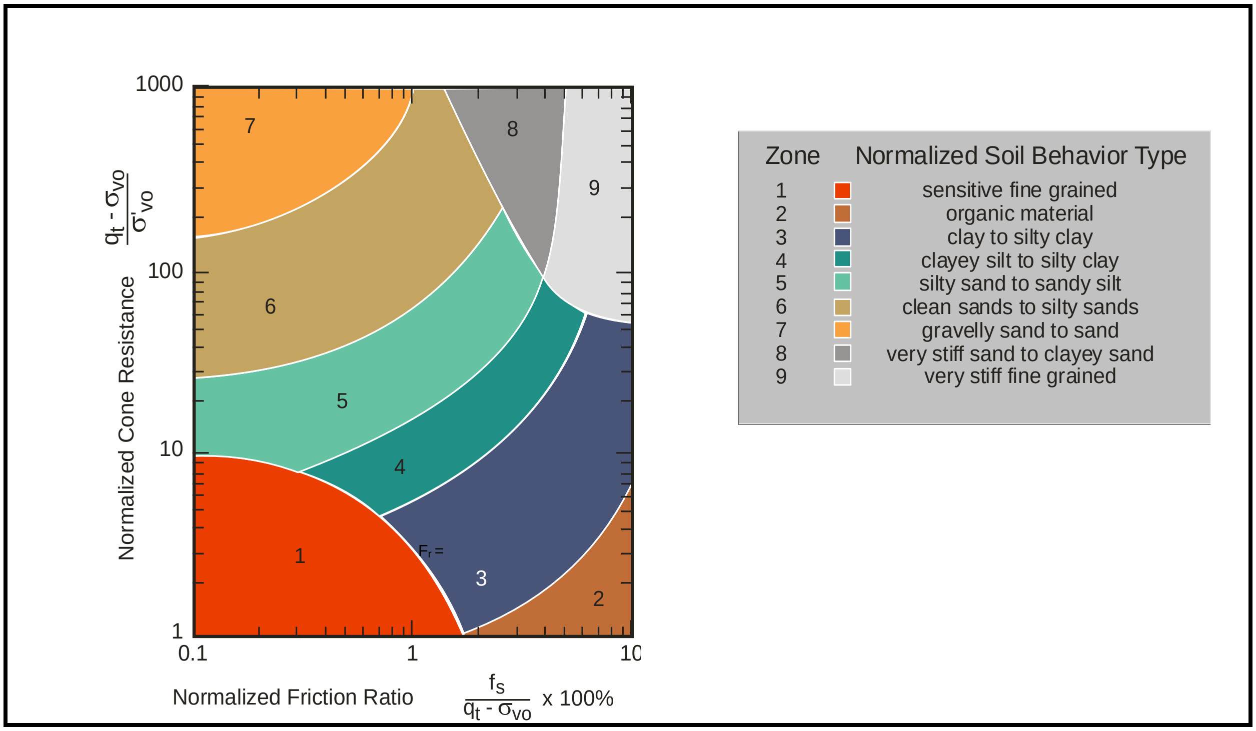


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

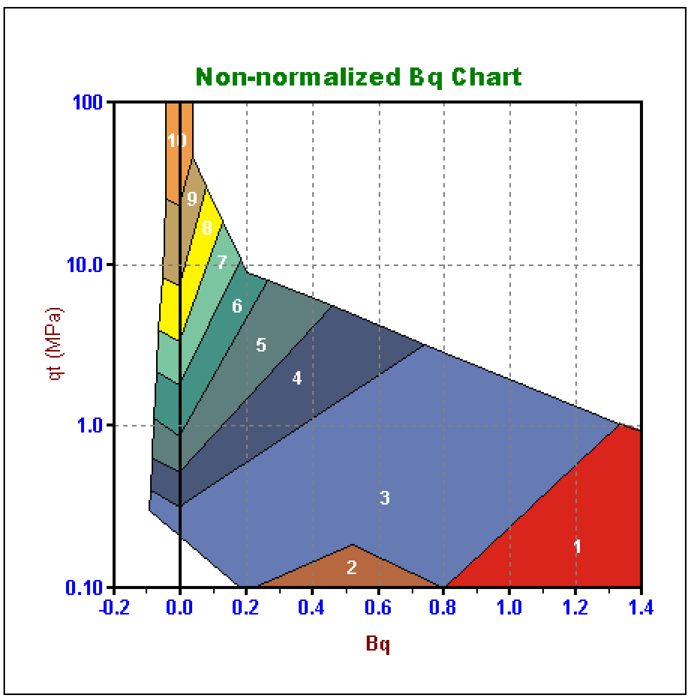


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

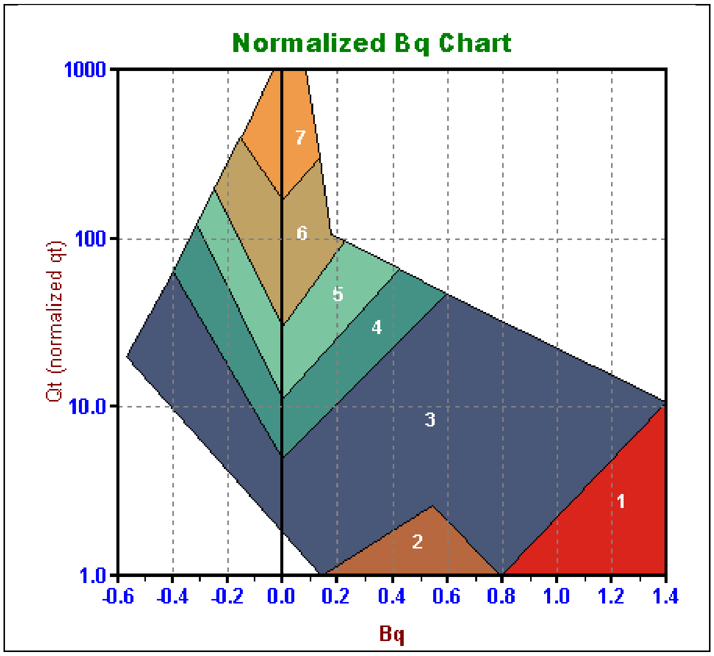


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

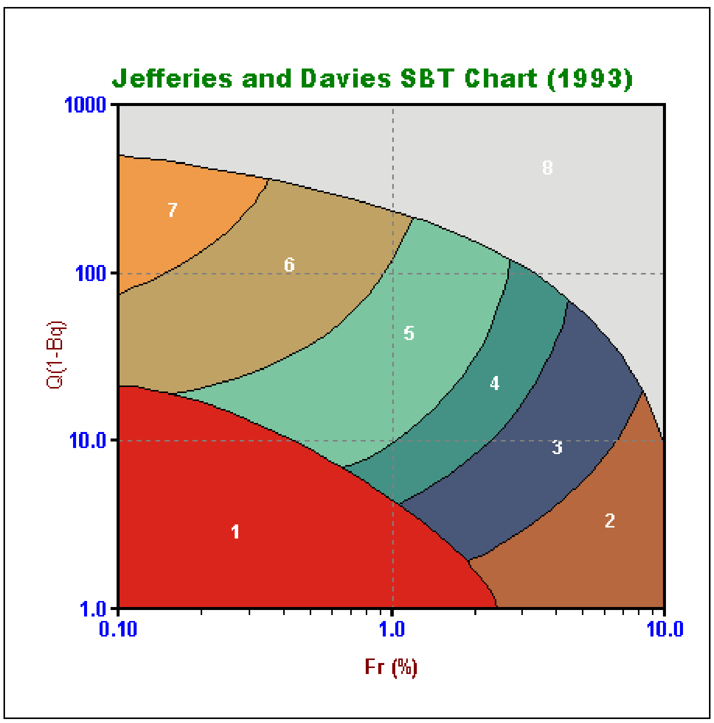


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

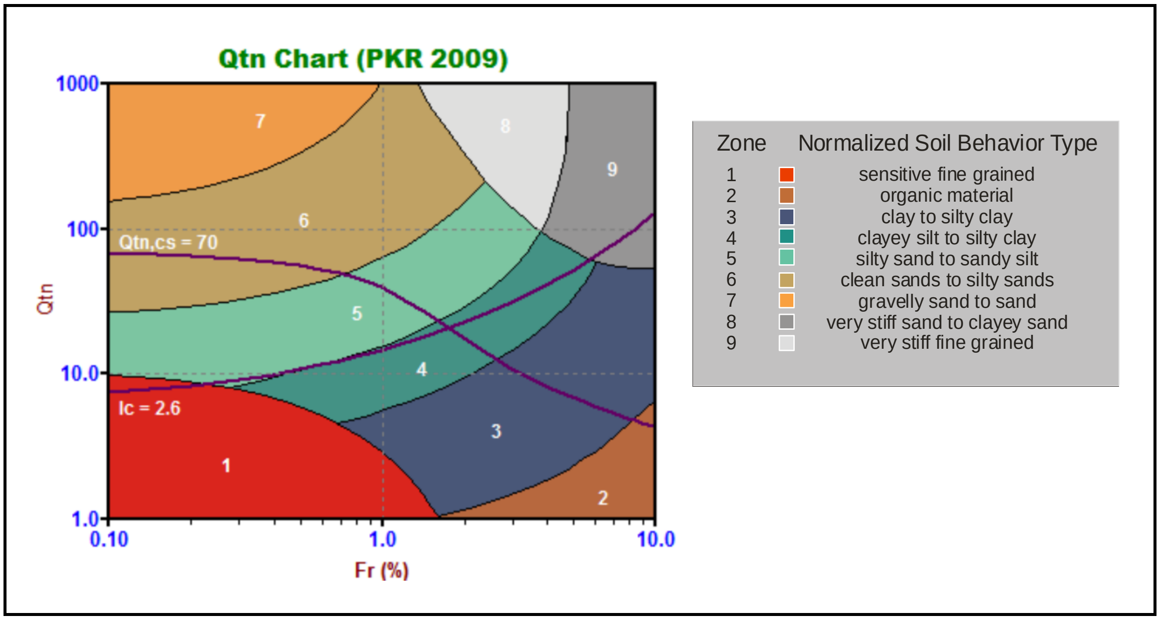


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

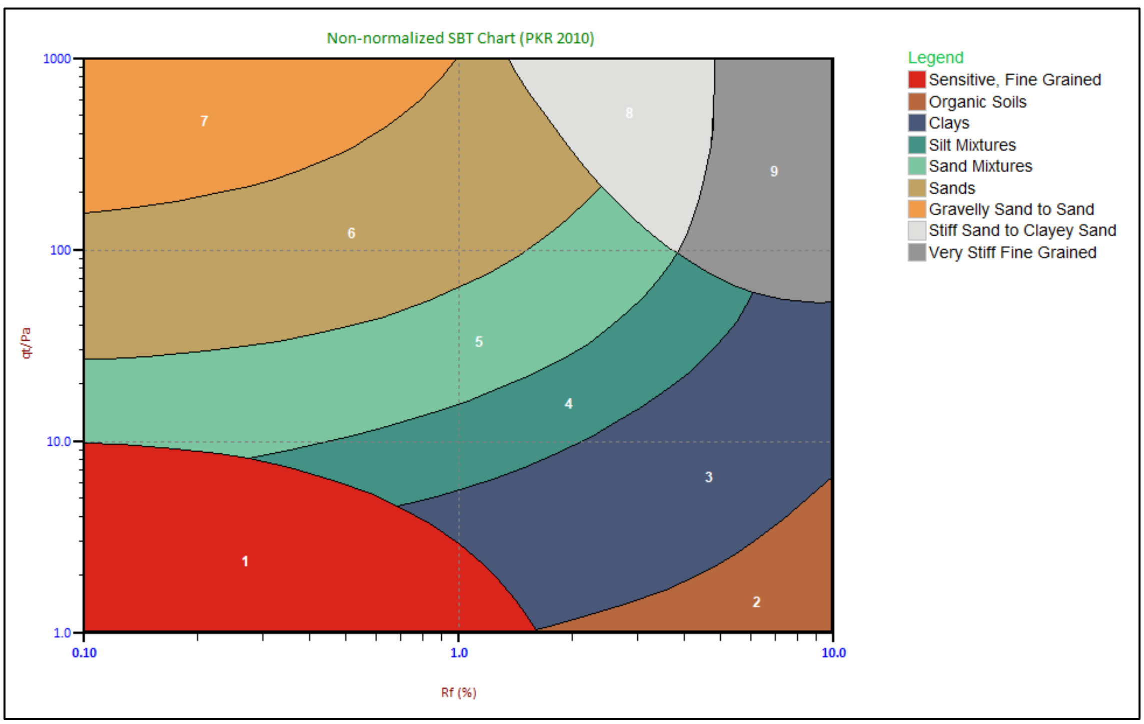


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

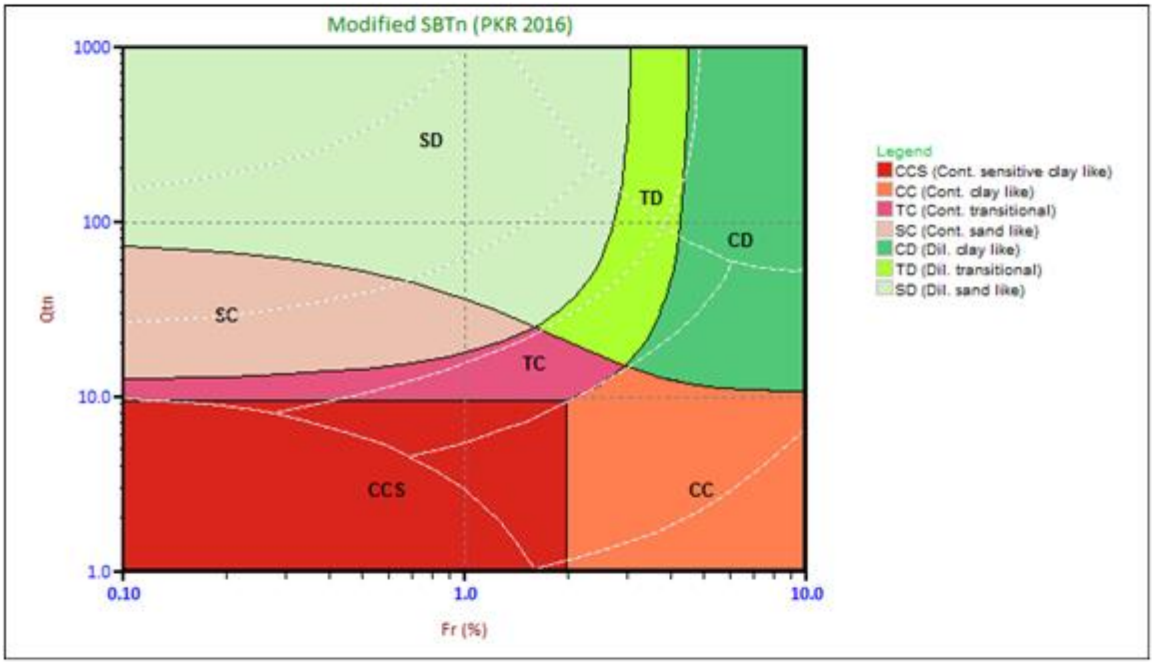


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

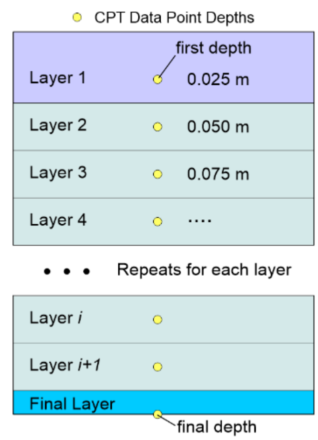
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

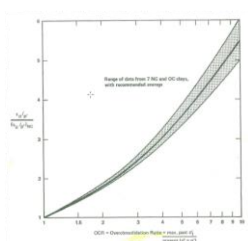
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

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

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

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

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

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

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

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