GEOTECHNICAL INVESTIGATION
4590 PATRICK HENRY DRIVE
SANTA CLARA, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Langan Engineering and Environmental Services (Langan) for the proposed development at 4590 Patrick Henry Drive in Santa Clara, California. We previously provided a preliminary geotechnical evaluation for the same site in a letter dated 26 June 2020.

The site is bound by Patrick Henry Drive on the east, a one-story office building and parking lot on the north, a two story office building and associated parking lot on the south and a channelized creek (Calabazas Creek) on the west, as shown on Figure 1.

The site has an overall plan dimension of about 323 by 377 feet and is currently occupied by a one-story office building in the central portion of the site, surrounded by an asphalt-paved parking lot. The site is relatively flat with the exception if the western boundary, which slopes up and then down to the adjacent to the Calabazas Creek. The foundation type for the existing building are currently unknown.

An existing utility easement runs parallel to the southern and western property lines. We understand the easements are for PG&E and public utilities.

We understand the existing building will be razed and removed from the site. The proposed building will be an eight-story, at-grade L-shaped structure. Per Holmes Structure, the building will be constructed using three concrete podium levels with five wood framed levels above, with an average pressure of approximately 1,050 pounds per square foot. Associated site improvements will include new landscape and hardscape areas and open space on the southern portion of the site.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our Budget Increase Request dated 14 October 2021. The purpose of our study was to provide geotechnical recommendations for the final design and construction of the proposed building. Our services consisted of a field investigation to evaluate the subsurface conditions, laboratory testing on selected soil samples obtained during the field investigation, and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions
- site seismicity and seismic hazards, including potential for fault rupture, ground shaking, liquefaction, lateral spreading and seismically induced settlements;
- applicable foundation type(s) for the proposed structure;
- design criteria for the recommended foundation type(s), including axial and lateral capacities;
- estimates of foundation settlements, including total and differential settlements;
- subgrade preparation for floor slabs and flatwork;
- compaction of backfill;
- seismic design criteria in accordance with the 2022 California Building Code (as appropriate); and
- construction considerations.

3.0 FIELD INVESTIGATIONS AND LABORATORY TESTING

We began our current subsurface investigation by reviewing the results of investigations previously performed at the site, which included borings performed by Levine Fricke in 1989. To supplement available subsurface information and gain further site-specific data, we drilled three borings, designated LB-1 through LB-3, and advanced seven cone penetration tests (CPTs) designated CPT-1 through CPT-7 at the project site. We performed laboratory testing on representative samples. The approximate locations of the geotechnical borings and CPTs are presented on Figure 2.

3.1 Field Investigation

Prior to performing our field investigation, we obtained a drilling permit from the Santa Clara Valley Water District (SCVWD), notified Underground Service Alert (USA), and checked the boring and CPT locations for utilities using an independent private utility locator.

All borings (LB-1 through LB-3) were drilled to a depth of approximately 91.5’ below the ground surface by means of mud rotary drilling by Pitcher Services, LLC between the 13 and 15 June

2022. The borings were advanced with a truck mounted drill rig using rotary wash equipment. During drilling, our field engineer logged the soil encountered and obtained soil samples for visual classification and laboratory testing. The boring logs are presented in Appendix A on Figures A-1 through A-3. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-4.

Soil samples were obtained using three different types of samplers: two driven split-barrel samplers, and one pushed thin-walled. The sampler types are as follows:

1. Standard Penetration Test (SPT) sampler with a 2.0-inch outside and 1.5-inch inside diameter (without liners)

2. Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter brass tubes

3. Shelby Tube (ST) sampler with a 3.0-inch outside diameter and a 2.875-inch inside diameter

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to hard cohesive soil, and the SPT sampler was used to evaluate the relative density of granular soils. Lastly, the Shelby Tube sampler was used to obtain relatively undisturbed samples of soft to medium stiff cohesive soil.

The SPT and S&H samplers were driven with a 140-pound hammer falling 30 inches. The samplers were driven up to 18 inches, and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the SPT and S&H samplers were converted to approximate SPT N-values using factors of 1.2 and 0.7 to account for sampler type and hammer energy and are shown on the boring logs.

The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

The Shelby Tube sampler was hydraulically pushed into the soil; the pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

Upon completion of drilling, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD, except for LB-2, which was converted to a piezometer. The soil cuttings and drilling fluid from the borings were placed in 55-gallon drums, which were stored temporarily at the site, tested, and were transported off-site for proper disposal.

3.2 Cone Penetration Tests

On 13 June 2022, ConeTec, Inc. of San Leandro, California, advanced seven supplemental CPTs at the site, designated CPT-1 through CPT-7, to depths of about 70 to 100 feet bgs. The CPTs were performed by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe, with a projected area of 15 square centimeters, into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measured the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data is processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. Pore pressure dissipation tests and shear wave velocity measurements were also taken in several of the CPTs. The results of the CPTs can be found in Appendix B.

Soil cuttings were not generated during the CPTs. After completion, the CPT holes were backfilled with cement grout in accordance with SCVWD requirements.

3.3 Laboratory testing

All samples recovered from the field exploration program were re-examined in the office for soil classifications, and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity (Atterberg limits), fines content (percent passing the No. 200 sieve), compressibility (consolidation), and undrained shear strength. Results of the laboratory tests are included on the boring logs and in Appendix C.

3.4 Borings by Others

Several borings were drilled and converted to monitoring wells within the project site by Lavine-Fricke in 1989. The approximate locations of the borings are shown on Figure 2 and the logs of borings are included in Appendix D. Because these borings were not drilled for the purpose of a geotechnical evaluation, they are of somewhat limited use for this study, however, the soil descriptions are consistent with our findings during our investigation and subsurface/ground water conditions described within this report.

4.0 SITE AND SUBSURFACE CONDITIONS

Our understanding of the site conditions and site subsurface conditions described in this section of the report are based on the previously generated data and results of our current geotechnical investigation of the site.

4.1 Site Conditions

The site has an overall plan dimension of about 323 by 377 feet and is currently occupied by a one-story office building in the central portion of the site, surrounded by an asphalt-paved parking lot. The site is relatively flat, with grades ranging between about Elevation 16 and 18 feet[1], with the exception if the western boundary, which slopes up adjacent to the Calabazas Creek, forming a type of levee. The site lies within the 500 year flood plain designated as Zone X.

An existing utility easement runs parallel to the southern and western property lines. We understand the easements are for PG&E and public utilities.

4.2 Subsurface Conditions

Based on the results of our field investigation, we conclude that the project site is underlain by alluvial deposits that predominantly consist of clay and sandy clay interbedded with lenses or layers of clayey sand and silty sand. The near-surface clay is typically medium stiff to stiff and overconsolidated, with overconsolidation[2] rations (OCRs) ranging from about 2 to greater than 5. The results of Atterberg limits test indicate the surface clay had a moderate expansion potential as indicated by Plasticity Index (PI) which was found to range from 13 to 25.

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1 All elevations referenced in this report are based on NAVD88 datum and were determined from the plan titled "Existing Conditions", Sheet C1.0, by BKF, dated 6 June 2023.

2 An overconsolidated clay has experienced a pressure greater than its current load.

The weaker surficial clay extends approximately 10 to 15 feet below the ground surface and is generally underlain by stiff to very stiff clay to a depth of about 30 feet below the ground surface. Below this level, the native clay is stiff to hard, is over consolidated and generally exhibits lower plasticity.

Sand layers encountered were generally dense with thin medium dense seams, wet, and contained varying amounts of silt and clay. Sand lenses were found to range from 6 inches to 24 inches in thickness and found randomly throughout the project site.

Groundwater was encountered in the borings and estimated in the CPTs at about 6.5 to 9.5 feet bgs (approximately Elevation of 7 to 10 feet) at the time of investigation. The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts. A design groundwater elevation of 11 should be used.

5.0 REGIONAL SEISMICITY AND FAULTING

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The San Andreas, Hayward, and San Gregorio faults are the major faults closest to the site. These and other faults of the region are shown on Figure 3. For each of these faults, as well as other active faults within about 50 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude[3] [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165] are summarized in Table 1. The mean Moment magnitude presented on Table 1 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

[3] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	12	Southwest	6.50
Total Hayward	13	Northeast	7.00
Total Hayward-Rodgers Creek	13	Northeast	7.33
Total Calaveras	17	East	7.03
N. San Andreas - Peninsula	18	Southwest	7.23
N. San Andreas (1906 event)	18	Southwest	8.05
N. San Andreas - Santa Cruz	25	South	7.12
Zayante-Vergeles	34	South	7.00
San Gregorio Connected	38	West	7.50
Mount Diablo Thrust	40	Northeast	6.70
Greenville Connected	41	Northeast	7.00

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 41 kilometers from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The 2016 U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
Hunting Creek/ Berryessa/ Green Valley/ Concord/ Mt. Diablo/ Greenville	16
San Gregorio	6

6.0 SEISMIC HAZARDS

The site is in a seismically active area and will be subject to strong to violent ground shaking during a major earthquake on a nearby fault. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4] and lateral spreading[5] and cyclic densification. Each of these conditions has been evaluated based on available subsurface information and are discussed in the following sections.

4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

6.1 Liquefaction and Lateral Spreading

The site is located within a zone designated with the potential for liquefaction, as identified by the California Geologic Survey (CGS) in a map titled, *State of California Seismic Hazard Zones, Milpitas Quadrangle*, prepared by the California Geologic Survey, dated October 19, 2004, as shown on Figure 5.

When saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

We used the procedures presented in Boulanger and Idriss (2014) to evaluate the liquefaction potential at the site. The Boulanger and Idriss procedures are updates of the simplified procedures developed by Seed et al. (1971) and later by the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Zhang et al. (2002) for the CPTs.

These analytical methods calculate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses:

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, relative density, depth of groundwater, earthquake magnitude, and overall soil behavior;
- Cyclic Stress Ratio (CSR), which quantifies the stresses that may develop during cyclic shaking.

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, we assume the soil layer may generate excess pore pressure and liquefy during a large seismic event.

In our analyses of the CPT results, soil that has significant amount of plastic fines, with I_c greater than 2.6 was considered too cohesive to liquefy. Additionally, a cone tip resistance q_{c1N} greater than 160 tons per square foot (tsf) was considered too dense to liquefy. Because the predominant earthquake is a moment magnitude 8.1, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Idriss (Youd and Idriss 2001).

Layers of loose to medium dense saturated sand and silts varying in thickness from approximately ½- to about 1-feet were encountered below the design groundwater level. These layers range in depth from about 8 feet bgs down to approximately 50 feet bgs. On the basis of the results of our analyses, we conclude several of these soil layers could potentially liquefy during a major earthquake.

We further used the CPT data to evaluate contractive and dilative behavior of the medium dense sand based on the findings by Robertson (2016). Robertson suggests a boundary line between contractive and dilative behavior, when plotting normalized cone resistance versus normalized friction ratio CPT data, where soil with data points that plot below that line exhibit contractive behavior and can potentially liquefy and strain significantly, whereas soil with behavior that plots above the line tends to dilate during shearing and, therefore, have reduced potential for large strain behavior. We applied a contractive-dilative boundary of 70, per Robertson (2016), to the CPT data obtained at the site.

The results of our liquefaction analyses indicate there are thin layers of loose to medium dense sand below the groundwater table that are contractive and susceptible to liquefaction and associated settlements during a major earthquake. Based on our liquefaction analyses using the borings and CPTs, we conclude that up to about 1¼ inch of liquefaction-induced total settlement may occur at the site as a result of a major earthquake on a nearby fault. The liquefaction may occur in isolated areas and differential settlement may be abrupt; therefore, differential settlements equivalent to the total settlement of 3/4 inch should be anticipated over short distances.

The layers below the anticipated groundwater level that have the potential for liquefaction are typically dense enough to resist lateral spreading and are generally clayey. We therefore conclude the potential for lateral spreading at the project site is low.

6.2 Cyclic Densification

Cyclic densification refers to seismically-induced settlement of non-saturated granular material (sand and gravel above the groundwater table) caused by earthquake vibrations. The borings and

CPTs indicate that the materials above the water table are predominantly composed of stiff to very stiff clayey soils, and therefore seismic densification is unlikely.

6.3 Fault Rupture

Historically, ground surface displacements closely follow the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure at the site is very low.

7.0 DISCUSSION AND CONCLUSIONS

We conclude that, from a geotechnical engineering standpoint, the site can be developed as proposed, provided the recommendations presented in this report are incorporated into the project plans and specifications are implemented during construction.

The primary geotechnical concerns for the site are:

- the anticipated settlement of the building under design loads
- potential for differential settlement during/following a seismic event
- settlement of the structure relative to site utilities.

These and other geotechnical issues are discussed in this section.

7.1 Foundations and Settlement

Based on our discussions with the structural engineer, we estimate that if the building were to be supported on a mat foundation, the resulting average net long-term gravity bearing pressure would be about 1,050 pounds per square foot (psf). Using this average bearing pressure for consolidation settlement calculations, we anticipated a mat foundation can be used and the anticipated total static settlement would be on the order of 1½ inch; with a differential of ¾ inch over 50 feet.

In addition to the static settlement, there are potentially liquefiable layers below the building footprint at depth. Upon review, we conclude the potential for lateral spreading is low and no specific mitigation measures are needed. However, total seismic settlement from liquefaction during a major earthquake is anticipated to be up to 1¼ inch beneath the building, with differential settlement of about ¾ inch over 50 feet. We conclude it is feasible to support the structure from a geotechnical standpoint on a mat foundation that is designed for the static settlement and the settlement due to liquefaction.

If a mat cannot be designed for the anticipated settlements, the building can be supported on deep foundations or a mat foundation over ground improvement.

The medium stiff to stiff clay that will likely be exposed at the foundation level will be susceptible to disturbance under construction equipment loads and a working pad may be needed. The need for the working pad should be evaluated when the bottom of the excavation is reached; however, for budgeting purposes an allowance for the working pad should be included.

7.2 Expansive Soil Considerations

The existing near-surface soil has a moderate expansion potential. Moisture fluctuations in near-surface expansive soil could cause the soil to expand or contract, resulting in movement and potential damage to improvements that overlie them. Potential causes of moisture fluctuations include drying during construction, subsequent wetting from rain, capillary rise, landscape irrigation, storm water infiltration, and type of plant selection.

For improvements at-grade, the volume changes from expansive soils can cause cracking of foundations, slabs and exterior flatwork. Therefore, foundations, slabs and concrete flatwork should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by supporting foundations below the zone of severe moisture change, moisture conditioning the expansive soil, and providing select, non-expansive fill below interior and exterior slabs.

Detailed recommendations for mitigating the effects of the moderately expansive near-surface soils are described in Section 8.

7.3 Exterior Improvements and Underground Utilities

Ground settlements of 1½ inches may occur under the proposed structure as a result of static building loads and up to 1¼ inches of seismic settlement may occur during a major earthquake. This settlement could affect various aspects of the planned development, including utilities, building entrances, and sidewalks. Design of these elements should incorporate features to mitigate the effects of the settlements. To mitigate the anticipated differential settlement (liquefaction and consolidation ground settlement), flexible connections can be planned where utilities enter the buildings. Additionally, exterior slabs and ramps attached to buildings can be designed to accommodate differential settlement between the buildings and exterior ground at all entrances. Because of the potential for settlements during an earthquake, it may be necessary to repair/replace utilities or exterior flatwork after an earthquake.

7.4 Construction Considerations

Details about previous developments at this site are not known. Buried debris (e.g. foundations and pipes) will be encountered during site preparation. Therefore, the earthwork on this site should include locating and removing near-surface and surface obstructions prior to construction of the project.

8.0 RECOMMENDATIONS

Recommendations for site preparation and grading, excavations and shoring, foundation support, exterior improvements, and seismic design are presented in the following sections of this report.

8.1 Earthwork

The site should be prepared and fill and backfill compacted in accordance with the recommendations presented in the following sections.

8.1.1 Site Preparation

Existing pavements, building foundations, utilities and other obstructions should be removed from areas to receive improvements. We anticipate the excavation for this project can be made using conventional earth-moving equipment except where old foundations and other obstructions are encountered. These may require hoe rams or jackhammers to remove. Any portions of existing buried foundations or other obstructions that extend below new improvements and could interfere with the proposed excavation and foundations should be removed to a depth of at least one foot below the bottom of the proposed foundation depth.

Demolished asphalt and concrete at the site may be crushed to provide recycled construction materials, including sand, free-draining crushed rock, and potentially Class 2 aggregate base (AB) provided it is properly crushed and screened and acceptable from an environmental standpoint. Where this recycled material will be used as a crushed rock in applications where free-draining materials are required, it should have no greater than six percent of material passing the 3/8-inch sieve. Where recycled Class 2 AB will be used beneath pavements, it should meet requirements of the Caltrans Standard Specifications. Recycled Class 2 AB that does not meet the Caltrans specifications should not be used beneath City streets, but could be acceptable for use as select fill within building pads and beneath concrete flatwork, provided it meets the requirements for select fill as presented later in this section.

Existing underground utilities should be removed to the service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the planned construction, they may be abandoned in place, provided the lines are filled with lean concrete or cement grout to the limits of the project. Voids resulting from the demolition activities should be properly backfilled with lean concrete or engineered fill as described below.

Prior to placing fill the subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned to at least three percent above the optimum moisture content, and compacted to at least 88 to 92 percent relative compaction[6]. An exception to this general procedure is within any proposed vehicle pavement areas, where the upper six inches of the pavement subgrade should be compacted to at least 95 percent relative compaction regardless of expansion potential.

8.1.2 Fill Placement and At-Grade Improvements

Fill should consist of onsite native soil or imported soil that is non-corrosive, free of organic matter or other deleterious material, and contains no rocks or lumps larger than four inches in greatest dimension.

A minimum of 8-inches of select fill should be placed beneath exterior concrete flatwork. The existing near-surface soil does not meet the requirements for select fill. If native, moderately expansive soil will be used as select fill, it should be lime-treated to meet the criteria for select fill discussed below. The existing soil may be used as general site fill below the select fill, provided the soil is moisture-conditioned to at least three percent above the optimum moisture content, and compacted to at least 88 to 92 percent relative compaction.

Select fill should consist of imported or on-site soil that is free of organic matter and hazardous material, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. In addition, select fill used within the at-grade building footprints and flatwork areas should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath slabs.

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6 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

Any select fill should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture-conditioned to near the optimum moisture content, and compacted to at least 90 percent relative compaction, except for fill that is placed within the proposed pavement areas. In these situations, the upper six inches of the soil subgrade and aggregate baserock materials should be compacted to at least 95 percent relative compaction. Fill thicker than five feet or clean sand or gravel (soil with less than 10 percent fines by weight) used as fill should be compacted to at least 95 percent relative compaction. A flowable cement grout, lean concrete, or lightweight cell-crete may be used to backfill areas not accessible to compaction equipment.

Langan should approve all sources of fill at least three days before use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material. A bulk sample of approved fill should be provided to the geotechnical engineer at least three working days before use at the site in order to obtain a compaction curve.

We recommend new sidewalks and concrete flatwork be underlain by at least 8-inches of select fill, of which the upper portion should consist of four inches of Class 2 aggregate base material (or the minimum thickness per City of Santa Clara Standards) that is compacted to at least 95 percent relative compaction.

8.1.3 Lime Treatment

An alternative to importing select fill is to perform lime treatment of the near surface soil. Lime treatment can be used to reduce the expansion potential of the near-surface soils, can be used to stabilize areas where weak clayey soils are encountered, or can be used to winterize clayey surface soils. Lime treatment of fine-grained soils generally includes site preparation, application of lime, mixing, compaction, and curing of the lime treated soil. Field quality control measures should include checking the depth of lime treatment, degree of pulverization, lime spread rate measurement, lime content measurement, and moisture content and density measurements, and mixing efficiency. Quality control may also include laboratory tests for unconfined compressive strength tests or Atterberg Limit tests on representative samples.

Lime stabilization of the at-grade pads and the subgrade of exterior flatwork may be a cost-effective means of improving on-site soils for use as non-expansive fill within the site.

The lime treatment process should be designed by a contractor specializing in its use and who is experienced in the application of lime in similar soil conditions. Based on our experience with lime treatment, we judge that the specialty contractor should be able to treat the moderately expansive on-site material to produce a non-expansive fill for building or flatwork subgrade.

If the lime treatment alternative is selected, we recommend that the specialty contractor prepare a treatment specification for our review prior to construction and perform Atterberg Limit tests for various types and content of lime to develop an adequate mix design to meet the criteria for select fill. The contractor should also check for sulfate content in the native soil and determine if it is sufficiently high to cause heaving.

If construction continues during the winter, the near surface soil may become wet and difficult to compact. If required, the soil can be mixed with lime to aid in compaction. The effectiveness of the lime for compaction and degree to which lime will react with soil depends on such variables as type of soil, minerals present, quantity of lime, the length of time the lime-soil mixture is cured and mixture methods.

8.1.4 Utility Trench Backfill

All trenches should conform to the current OSHA requirements for slopes, shoring, and other safety concerns. The thickness and type of bedding material required for utility conduits will depend on the soil conditions at the utility trench bottom. As a minimum, bedding should extend at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe and should be at least four inches thick. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of at least six inches with sand or fine gravel, which should be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be compacted according to the recommendations presented in Section 8.1.2. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted.

Where utility trenches backfilled with sand or gravel enter the building pads, an impermeable plug consisting of low-expansion potential clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building, improvements, or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

8.2 Mat Foundation

The mat for the proposed structure should bear on undisturbed or recompacted native clay, provided the estimated settlements are acceptable. To design the mat using the modulus of subgrade reaction method, we recommend a static moduli of subgrade reaction of 10 kips per cubic foot (kcf) be used. The modulus values are representative of the anticipated consolidated settlement of the clay under the building loads provided by Holmes Structure. Localized pressures may be higher; maximum allowable bearing pressures should be limited to 3,500 psf under static conditions; these may be increased by 1/3 for total load conditions including wind or seismic loads.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. We recommend an equivalent fluid weight of 260 pcf be used to compute passive resistance. Frictional resistance should be computed using a base friction coefficient of 0.15; this friction value assumes a waterproofing membrane is placed below the mat. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

Uplift loads may be resisted by the weight of the building. If building weight is inadequate to provide the necessary uplift resistance, tiedown anchors may be used. If tiedown anchors are required, we should present design recommendations in an addendum.

To mitigate the potential for shrink/swell behavior from the moderately expansive near-surface clay, the mat foundation should be embedded at least 30 inches beneath the adjacent exterior grade which can be accommodated using a turned-down edge if necessary. Additionally, the mat should have top and bottom reinforcement used in both directions.

The exposed subgrades and excavations for foundations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottom of the excavation should be kept moist until concrete is placed. We should check the subgrade after cleaning, and prior to placement of crushed rock or reinforcing bar to check that loose to disturbed material has been removed and the subgrade is firm and non-yielding. If loose, soft, disturbed, or otherwise undesirable material is observed at the subgrade, it should be overexcavated to firm, competent material and be replaced with either engineered fill or concrete.

8.3 Floors and Floor Slab

The top of the mat may be used as a floor, or a topping slab may be placed above the mat to provide a smooth wearing surface. At the planned depth, it will be above groundwater. We recommend installing a capillary moisture break and a water vapor retarder if water vapor moving through the mat is unacceptable or if there are finished floor coverings susceptible to moisture.

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the gravel/crushed rock should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

If weak or soft material is encountered in the mat or lean concrete excavation bottom, it should be overexcavated and replaced with lean concrete. We should observe mat subgrade and overexcavated subgrade prior to placement of reinforcing steel or lean concrete. The excavation for the mat should be free of standing water, debris, and disturbed materials prior to placing concrete.

8.4 Below Grade and Retain Walls

Any below grade walls, including elevator pits, should be designed to resist lateral pressures imposed by the adjacent soil and any surcharge loads. Because the site is in a seismically active area, basement walls should be checked for the seismic condition. Under seismic loading conditions, there will be a seismic increment that should be added to active earth pressures (Sitar, et al., 2012). We used the procedures outlined by Sitar, et al. (2012) to compute the seismic pressure increment. We estimated the seismic pressure increment using the geometric mean of the PGA. The more critical condition of either at-rest pressure or active pressure plus a seismic increment (total pressure) should be checked. At-rest and total pressures for level backfill, are presented in Table 4.

TABLE 4
Earth Pressures for Basement Wall Design

Drainage Condition	Static Conditions		Seismic Conditions	
			Total Pressure (Active Pressure plus Seismic Pressure Increment)	
	Active Pressure (pcf)	At-Rest Pressure (pcf)	DE (PGA = 0.389g) (pcf)	MCE_R (PGA = 0.583g) (pcf)
Drained	40	60	65	75
Undrained	80	95	95	100

Walls that are within 10 feet of the streets should be designed for an additional lateral pressure of 100 psf in the upper 10 feet. If additional surcharge loads occur within the zone of influence (defined by an imaginary plane projected up from the bottom of the wall at a 30-degree angle from horizontal), a surcharge pressure should be included in the wall design.

To protect against moisture migration, any below-grade walls should be waterproofed and water stops should be placed across all construction joints if water vapor is not desired. The waterproofing should be placed directly against the backside of the walls. The waterproofing should be designed by a consultant with local experience.

Walls should be properly backdrained if they are designed for the drained condition. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

8.5 Excavations

In general, site excavations are likely limited to utility trenches and excavations for foundations and elevator pits. Where space permits, the sides of excavations can be sloped. Temporary excavation slopes should be no steeper than 1½:1 (horizontal to vertical) in the fill and native soil above the water table. Where space does not permit a sloped excavation or where excavations extend below five feet and/or below water, shoring will be required.

If water seepage is encountered during excavation, dewatering measures, such as placing pumps in sumps in the bottom of the excavation, should be employed.

8.6 Concrete Pavement and Exterior Slabs

Differential ground movement due to expansive soil and settlement will tend to distort and crack the pavements and exterior improvements such as courtyards and sidewalks. Periodic repairs and replacement of exterior improvements should be expected during the life of the project. Mastic joints or other positive separations should be provided to permit any differential movements between exterior slabs and the buildings.

To reduce the potential for cracking related to expansive soil, we recommend exterior concrete flatwork be underlain by at least 8-inches of select fill, of which the upper four inches should consist of aggregate base compacted to at least 95 percent relative compaction. The select fill should extend 3 feet beyond the edge of the concrete flatwork. The subgrade should be compacted to at least 90 percent relative compaction, and should provide a smooth, non-yielding surface for support of the concrete slabs.

Where rigid pavement is required for loading and service areas, we recommend a minimum of six inches of concrete for medium traffic and a minimum of eight inches of concrete for heavy traffic. For vehicular concrete pavements contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement or the existing structure, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 to 10. The slab edges should be confined by curbs or pavement, and slabs should have dowels connecting adjacent slabs. In addition, at areas subject to vehicles with heavy axle loads, we recommend the slabs be reinforced with a minimum of No. 4 bars at 16-inch-spacing in both directions.

The upper six inches of subgrade should be compacted to at least 95 percent relative compaction and should provide a smooth, non-yielding surface. The concrete should be underlain by at least 6 inches of Class 2 Aggregate base. Aggregate base material should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications.

8.7 Surface Drainage

Drainage control design should include provisions for positive surface gradients of at least 1½ percent within 5 feet of the building so that surface runoff is not permitted to pond, particularly above slopes or adjacent to building foundations, roadways, pavements, or slabs. Surface runoff should be directed away from slopes and foundations and collected in lined ditches or drainage swales. The water collected should be directed to a storm drain or paved roadway.

Discharge from the roof gutter and downspout systems should be included in the collection system and not allowed to infiltrate the subsurface near the structures or in the vicinity of slopes.

8.8 Landscaping

The use of water-intensive landscaping within 5 feet of the buildings should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around the building should be limited to drip or bubbler-type systems. Trees with large roots or have high water demand should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which have been known to cause significant differential movement over short horizontal distances in expansive soil, resulting in cracking and tilting of slabs and architectural damage.

To reduce the potential for irrigation water entering the pavement section, vertical curbs adjacent to landscaped areas should extend through any aggregate base and at least six inches into the underlying soil. In heavily watered areas, such as lawns, it may also be necessary to install a subdrain behind the curb to intercept excess irrigation water.

8.9 Seismic Design Criteria

Because the soil underlying the site consists predominantly stiff clays, we recommend using Site Class D for seismic design of the proposed development. A site-specific ground motion analysis is required for structures on Site Class D sites with S_1 greater than or equal to 0.2, in accordance with the 2022 CBC and by reference ASCE 7-16, unless the criteria listed in the exception in Section 11.4.8 of ASCE 7-16 as modified by Supplement 3 of ASCE 7-16 are met. If the project structural engineer confirms that these criteria will be met for the project, the seismic design can be performed in accordance with the provisions of 2022 CBC/ASCE 7-16 and Supplement 3, and we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion parameter spectral response acceleration (5% critical damping) for 0.2 seconds (S_s) and 1 second (S_1) of 1.50g and 0.60g, respectively

- Site Class D

- Site Coefficients Fa and Fv of 1.0 and 1.7 , respectively

- MCE_R and DE spectral response acceleration parameters at short periods, SM_S and SD_S, of 1.50g and 1.00g, respectively

- MCE_R and DE spectral response acceleration parameters at one-second period, SM_1 and SD_1, of 1.53g and 1.02g , respectively (these values have been increased by 50% in accordance with ASCE 7-16 Supplement 3).

If the criteria in accordance with Section 11.4.8 of ASCE 7-16 cannot be met, a site-specific ground motion analysis will be required and can be issued as an addendum.

9.0 SERVICES DURING DESIGN, CONSTRUCTION DOCUMENTS, AND CONSTRUCTION QUALITY ASSURANCE

During final design, we should be retained to consult with the design team as geotechnical questions arise. Technical specifications and design drawings should incorporate Langan's recommendations. When authorized, Langan will assist the design team in preparing specification sections related to geotechnical issues such as foundation installation and testing, ground improvement design and installation (if required), earthwork, and backfill. Langan should also, when authorized, review the project plans, as well as Contractor submittals relating to materials and construction procedures for geotechnical work, to check that the designs incorporate the intent of our recommendations.

Langan has investigated and interpreted the site subsurface conditions and developed the foundation design recommendations contained herein, and is therefore best suited to perform quality assurance observation and testing of geotechnical-related work during construction. The work requiring quality assurance confirmation and/or special inspections per the Building Code includes, but is not limited to, installation and testing of foundations, ground improvement, earthwork, backfill, and excavation support. In fulfillment of these duties, we should observe the installation and testing of ground improvement and excavation of the final foundation subgrade/installation of deep foundation elements. We will also review monitoring data pertaining to shoring system (if used) and settlement of adjacent structures provided by the surveyor. We should also observe any fill placement and perform field density tests to check that adequate fill compaction has been achieved.

Recognizing that construction observation is the final stage of geotechnical evaluation, quality assurance observation during construction by Langan is necessary to confirm the design assumptions and design elements, to maintain our continuity of responsibility on this project, and allow us to make changes to our recommendations, as necessary. The foundation system and general geotechnical construction methods recommended herein are predicated upon Langan reviewing the final design and providing construction observations.

10.0 OWNER AND CONTRACTOR RESPONSIBILITIES

The contractor should be responsible for construction quality control, which includes satisfactorily constructing the ground improvement, foundation system, and any associated temporary works to achieve the design intent while not adversely impacting or causing loss of support to neighboring properties, structures, utilities, roadways, etc. Construction activities that can alter the existing ground conditions such as excavation, fill placement, foundation construction, etc. can also induce stresses, vibrations, and movements in nearby structures and utilities, and disturb occupants. Contractors should ensure that their activities will not adversely affect the structures and utilities. Contractors should also take the necessary measures to protect the existing structures, utilities, etc. during construction.

11.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings and CPTs. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations.

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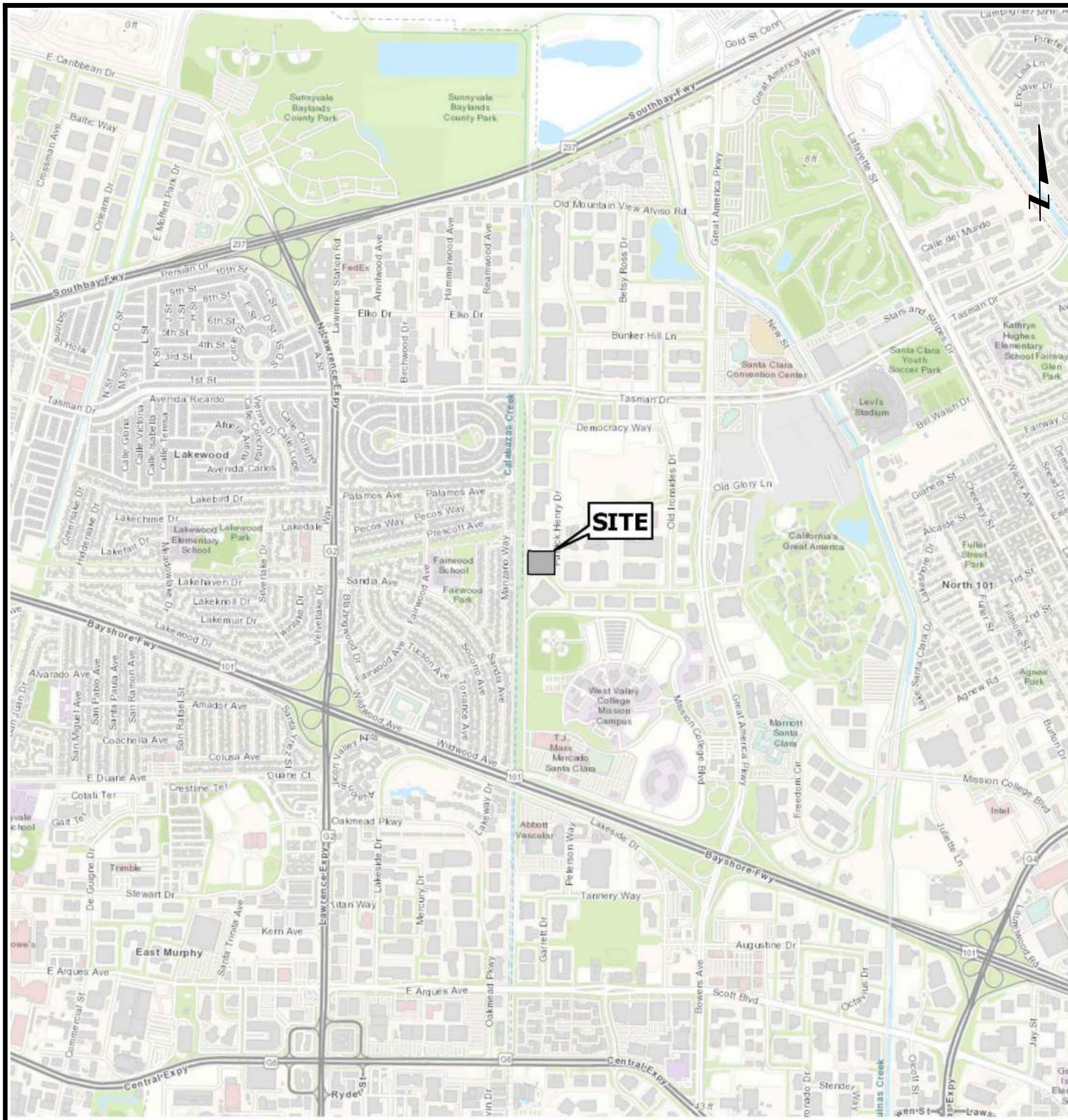
FIGURES



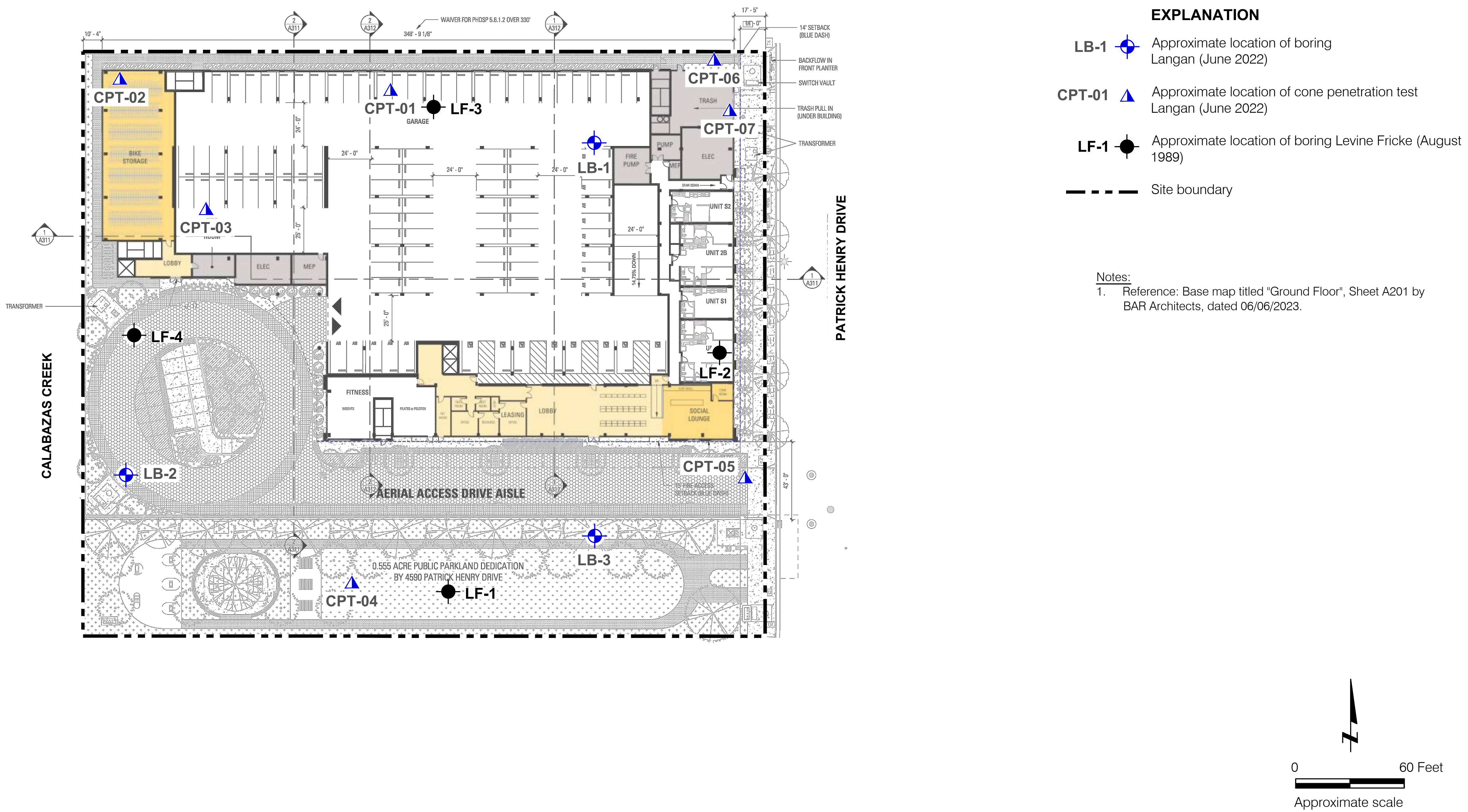
Note:
Topographic base map is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed.

0 — 1,000 Feet

Approximate scale

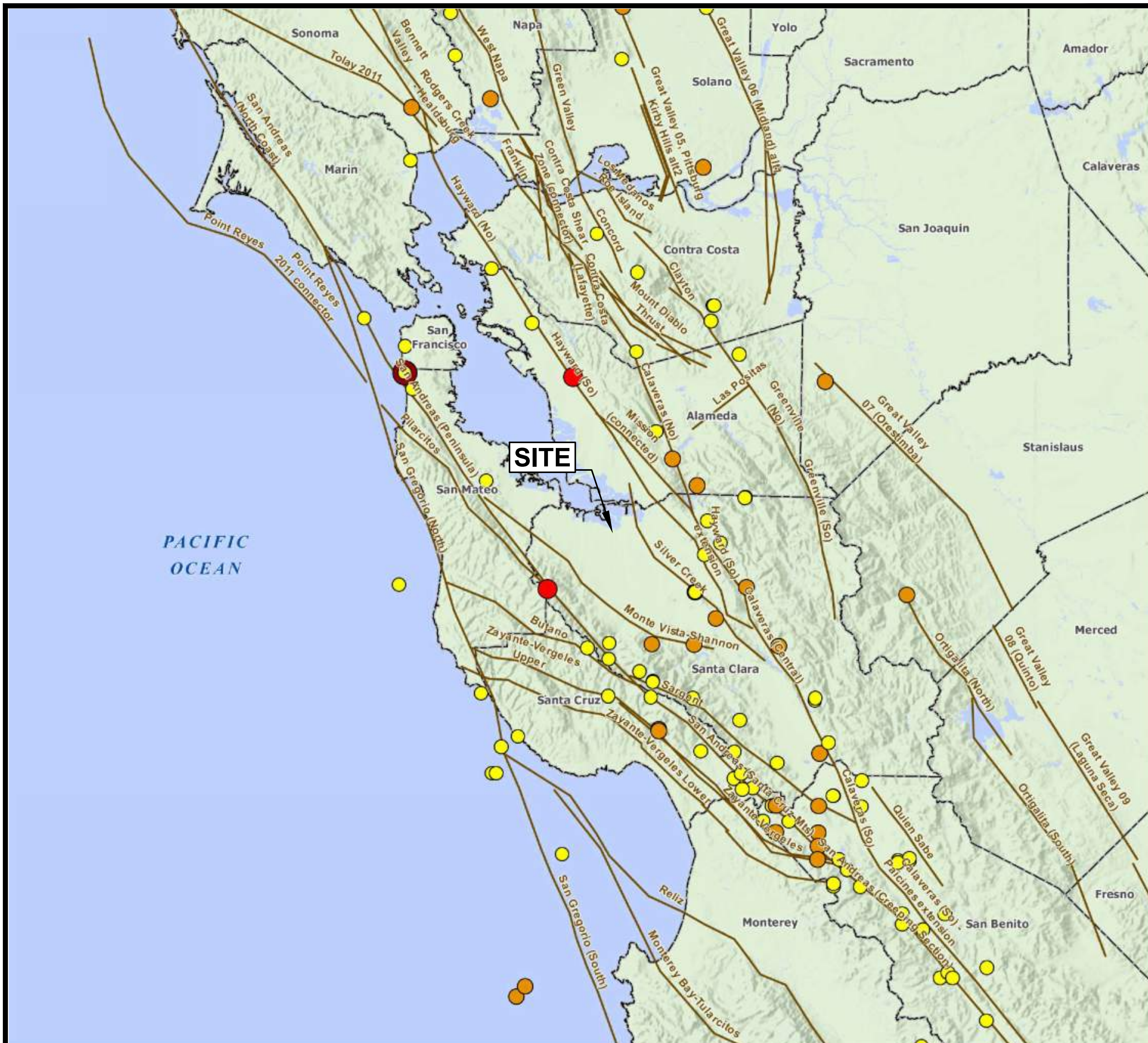
LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1814 Franklin Street, Suite 505 Oakland, CA 94612 T: 510.874.7000 F: 510.874.7001 www.langan.com	**4590 PATRICK HENRY DRIVE** SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	**SITE LOCATION MAP**	750664902 Date 07/31/2023 Drawn By AG Checked By PB	1

Filename: \\langan.com\data\OAK\data9\750664901\Project Data\CAD\01\2D-DesignFiles\Geotechnical\750664901-B-GI0101_SiteLocMap.dwg Date: 7/31/2023 Time: 10:39 User: agekas Style Table: Langan.stb Layout: Figure 1 Site Loc



LANGAN Langan Engineering and Environmental Services, Inc. 1814 Franklin Street, Suite 505 Oakland, CA 94612 T: 510.874.7000 F: 510.874.7001 www.langan.com	Project 4590 PATRICK HENRY DRIVE SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	Drawing Title SITE PLAN	Project No. 750664902 Date 07/31/2023 Drawn By JDF/AG Checked By ECE	Figure 2

Filename: \\langan.com\data\OAK\data9\750664901\Project Data\CAD\01\2D-DesignFiles\Geotechnical\750664901-B-SP0101.dwg Date: 8/1/2023 Time: 13:01 User: agekas Style Table: Langan.stb Layout: Fig 2 Site Plan



LEGEND

County Boundary

Fault

Earthquake Epicenter Magnitude

- Magnitude 5 to 5.9
- Magnitude 6 to 6.9
- Magnitude 7 to 7.4
- Magnitude 7.5 to 8

0 20 km

Approximate scale

Notes:

1. Quaternary fault data displayed are provided by the CGS Map Sheet 48: Fault based seismic sources used in the Uniform California Earthquake Rupture Forecast, Version (UCERF3).
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 2015.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

LANGAN Langan Engineering and Environmental Services, Inc. 1814 Franklin Street, Suite 505 Oakland, CA 94612 T: 510.874.7000 F: 510.874.7001 www.langan.com	Project 4590 PATRICK HENRY DRIVE SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	Figure Title MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA	Project No. 750664902 Date 07/31/2023 Drawn By AG Checked By PB	Figure 3

Filename: \\langan.com\data\OAK\data9\750664901\Project Data\CAD\01\2D-DesignFiles\Geotechnical\750664901-B-GI0101_SiteLocMap.dwg Date: 7/31/2023 Time: 10:38 User: agekas Style Table: Langan.stb Layout: Figure 3 FaultMap

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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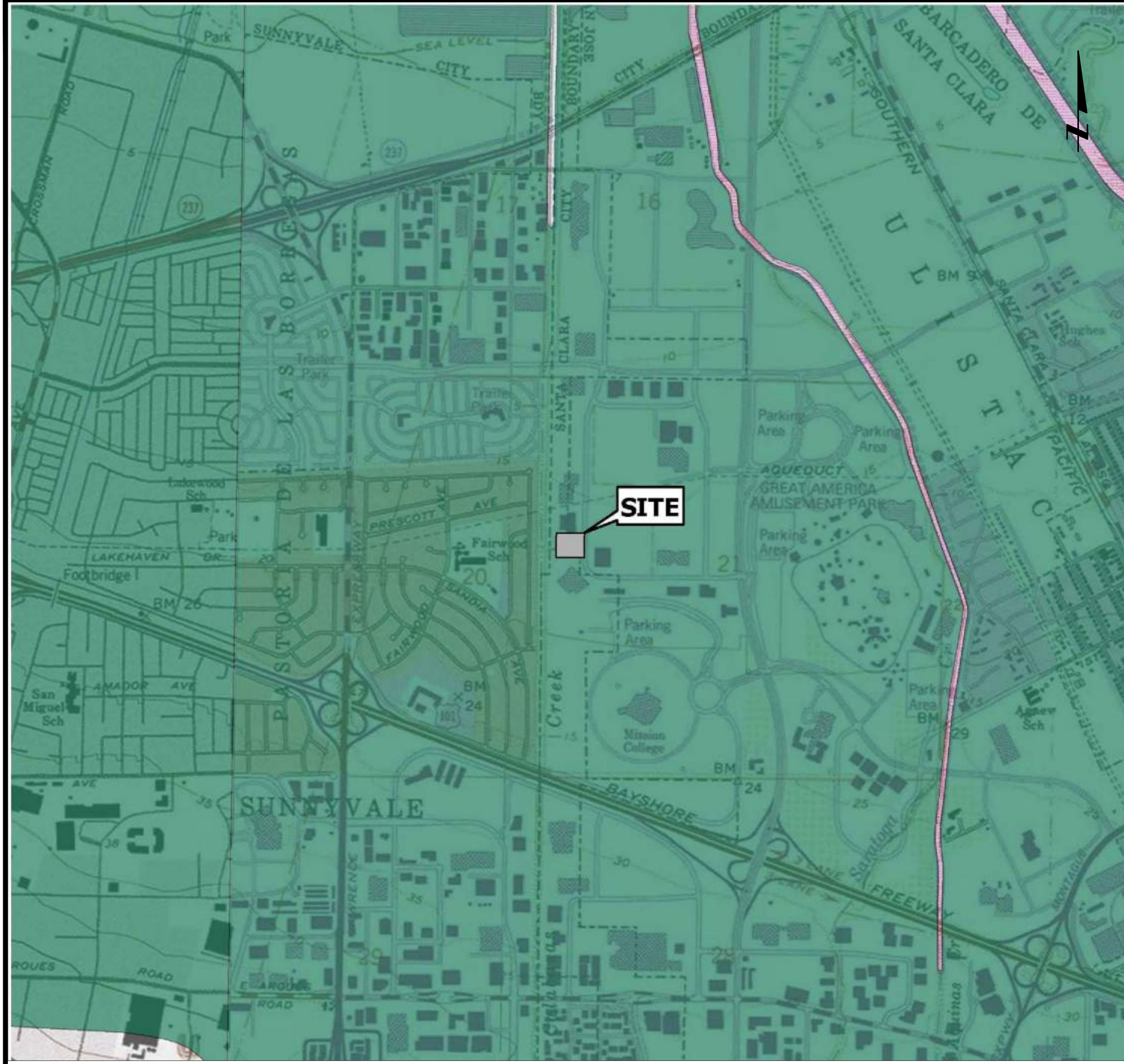
Project	Figure Title	Project No.	Figure
4590 PATRICK HENRY DRIVE SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	MODIFIED MERCALLI INTENSITY SCALE	750664902 Date 07/31/2023 Drawn By AG Checked By PB	4



EXPLANATION

Liquefaction; Areas where historic occurrence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

Notes:
1. Topographic basemap is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed and the USGS.
2. Data provided by the CGS through the GIS Seismic Hazard Zone Map presenting areas where liquefaction and land slides may occur during a strong earthquake.

0 2000 Feet
Approximate scale

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project: 4590 PATRICK HENRY DRIVE, SANTA CLARA, SANTA CLARA COUNTY, CALIFORNIA	Figure Title: REGIONAL SEISMIC HAZARD ZONES MAP	Project No. 750664902; Date 07/31/2023; Drawn By AG; Checked By PB	Figure 5 ###

Filename: \\langan.com\data\OAK\data9\750664901\Project Data\CAD\01\2D-DesignFiles\Geotechnical\750664901-B-GI0101_SiteLocMap.dwg Date: 7/31/2023 Time: 10:21 User: agekas Style Table: Langan.stb Layout: Figure 5 Haz Map

APPENDIX A

LOGS OF BORINGS

PROJECT:	4590 PATRICK HENRY DRIVE Santa Clara, California	Log of Boring LB-1 PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Date started: 06/13/22 | Date finished: 06/13/22

Drilling method: Mud Rotary

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: P. Marien
Drilled By: Pitcher Services, LLC

Ground Surface Elevation: 16.5 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4 inches asphalt concrete (AC)						
					6 inches aggregate base (AB)						
1–5	GRAB			CL	CLAY (CL) dark brown to gray, moist, trace fine-grained sand; light brown, trace fine- to medium-grained sand, trace silt						
5–6.5	S&H	0 4 5	6	CL	CLAY with SAND (CL) brown to light brown, medium stiff, moist, fine- to medium-grained sand						
6.5–7				SC	CLAYEY SAND (SC) light brown, loose, moist, fine-grained, trace silt						
7–8	SPT	0 3 5	10								
8–10				CL	CLAY with SAND (CL) light brown, stiff, moist, fine- to coarse-grained sand, trace silt; groundwater observed at 8.5 feet bgs (9:12am, 6/13/2022)						
10–11	SPT	0 1 4	6	SP-SC	SAND with CLAY (SP-SC) brown to light brown, loose, wet, fine-grained						
11–18				CL	CLAY with SAND (CL) light brown to brown, medium stiff, wet, fine-grained sand, trace silt						
15–16	S&H	7 10 10	14		light brown, stiff, fine- to coarse-grained sand	PP		1,500		23.9	103
18–29				CL	CLAY (CL) light brown, stiff to very stiff, wet, trace fine-grained sand						
21–23	ST		psi 60 80 120 180 180			PP		1,000		18.3	104
25–26	S&H	0 7 11	13		olive-gray, stiff, trace fine- to medium-grained sand	PP		500		27.3	97
29–30				SM	SILTY SAND (SM)						

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Project No.: 750664902 | Figure: A-1a

TEST GEOTECH LOG 750664902 BORING LOGS.GPJ TR.GDT 08/01/23

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		6 11 6	20	SM	SILTY SAND (SM) (continued) gray-brown, medium dense, wet, fine to medium grained, trace clay, little cohesion				14.6	20.6	
32–34	SPT		2 4 4	10	CL	CLAY with SAND (CL) olive-gray with gray-brown mottling, stiff, wet, fine- to medium-grained sand, trace silt						
35–37	S&H		11 19 23	29	CL	CLAY (CL) olive-gray, very stiff, wet, trace fine-grained sand	PP		4,250			
38–41	S&H		4 4 8	8	SC	CLAYEY SAND (SC) gray, loose, wet, fine- to medium-grained sand						
						LL = 23, PI = 8, see Appendix C				33.2	19.1	
42–43	SPT		5 8 17	30		gray, dense, wet, fine- to coarse-grained						
44–49	ST			psi 50 100 150 190 200	CL	CLAY (CL) gray, very stiff, wet, trace fine-grained sand, trace silt Consolidation Test, see Appendix C					25.0	97
47.5							PP		1,750			
50–51	S&H		11 21 31	36	CL	CLAY with SAND (CL) olive-gray with yellow-brown mottling, hard, wet, fine- to medium-grained sand, trace fine subangular gravel	PP		3,750			
54						trace sand seam at 54 feet						
55–56	S&H		6 9 15	17		olive-gray, very stiff	PP		2,500			
57–60												

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Project No.: 750664902

Figure: A-1b

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H	6 12 19	22	CL	CLAY with SAND (CL) (continued)	PP		2,750			
61–62				CL	SANDY CLAY (CL) olive-gray, very stiff, wet, fine-grained sand	PP		3,750	74.3	22.7	
63					LL = 41, PI = 22, See Appendix C						
64					trace sand seam at 63.5 feet						
65–66	S&H	7 13 17	21		olive-gray with yellow-brown mottling						
66–67						PP		3,750			
71	S&H	10 16 21	26		olive-gray, very stiff, fine-grained sand						
71–72						PP		1,750			
76	SPT	7 14 30	53		hard trace silt						
77–78				CL	CLAY (CL) olive-gray, very stiff, wet, trace silt, fine- to medium-grained sand						
81	S&H	9 18 23	29								
81–82						PP		1,500		23.8	103
86	S&H	6 14 21	25								
86–87						PP		1,500			
89				CL	SANDY CLAY (CL) olive-gray, very stiff, wet, fine-grained sand						
90											

PROJECT: **4590 PATRICK HENRY DRIVE** Santa Clara, California

Log of Boring LB-1

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		8 19 23	29	CL	SANDY CLAY (CL) (continued)	PP		4,000			
92												
93												
94												
95												
96												
97												
98												
99												
100												
101												
102												
103												
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110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 91.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 8.5 feet below ground surface.
PP = pocket penetrometer.
bgs = below ground surface.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD88 datum and were determined from the plan titled "Existing Conditions," Sheet C1.0, by BKF, dated 6 June 2023.

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Project No.: 750664902 | Figure: A-1d

PROJECT:	**4590 PATRICK HENRY DRIVE** Santa Clara, California	**Log of Boring LB-2** PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 06/15/22 — Date finished: 06/15/22
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches — Hammer type: Automatic
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: P. Marien
Drilled By: Pitcher Services, LLC

LABORATORY TEST DATA

Ground Surface Elevation: 16 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches asphalt concrete (AC)						
						8 inches aggregate base (AB)						
1–3	GRAB				CL	CLAY (CH) gray, moist, trace fine-grained sand, trace organics						
4					CL	CLAY (CL) olive-gray, moist, trace fine-grained sand						
5–9	S&H		7 12 18	21	CL	CLAY with SAND (CL) olive-gray with yellow-brown mottling, very stiff, moist, fine- to medium-grained sand, trace fine angular gravel, trace silt LL = 36, PI = 17, see Appendix C groundwater observed at 7.5 feet bgs (8:04am, 6/14/2022)					18.2	112
10–14	S&H		6 11 11	15	CL	CLAY (CL) olive-gray with yellow-brown mottling, stiff to very stiff, wet, trace fine- to medium-grained sand, decrease in sand content	PP		3,000			
15–18	S&H		2 5 7	8	CL	SANDY CLAY (CL) light-brown to yellow-brown with olive-gray mottling, medium stiff to stiff, wet, fine- to medium-grained sand	PP		1,250	51.8	19.3	
19–21	SPT		8 5 10	18	SC	CLAYEY SAND with GRAVEL (SC) light brown, medium dense, wet, fine- to coarse-grained, fine subangular gravel LL = 27, PI = 13, See Appendix C				15.4	11.1	
22–24	SPT		8 12 15	32	SP-SC	SAND with CLAY (SP-SC) brown to light brown, dense, wet, fine- to medium-grained, trace fine to coarse subangular gravel						
25–27	SPT		13 19 18	44	GW	SANDY GRAVEL (GW) brown, very dense, wet, fine to coarse subangular to subrounded, fine- to coarse-grained sand, trace clay						
28–30					SP-SC	SAND with CLAY (SP-SC) brown, wet, fine-grained						

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Project No.: 750664902 — Figure: A-2a

PROJECT:	4590 PATRICK HENRY DRIVE Santa Clara, California	Log of Boring LB-2

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	ST		psi 80 80 120 120 150	SP-SC	SAND with CLAY (SP-SC) (continued)						
32											
33	SPT	12 11 11	26	GP	SANDY GRAVEL (GP) gray-brown to brown, meidum dense, wet, fine to coarse subangular to angular, fine- to coarse-grained sand, trace clay						
34											
35	SPT	17 24 23	56	SP	SAND (SP) dark-brown to brown, very dense, wet, fine- to coarse grained, trace fine to coarse subangular gravel, decrease fine content						
36											
37					CLAY (CL) light brown, stiff, wet, trace fine-grained sand						
38											
39											
40											
41	ST		psi 60 100 200 200 250		Consolidation Test, see Appendix C					24.3	100
42						PP		1,500			
43											
44											
45											
46	S&H	6 11 15	18		olive-gray with yellow-brown mottling, very stiff, trace fine- to coarse-grained sand	PP		2,250			
47											
48				CL							
49											
50											
51	S&H	5 9 11	14		stiff, trace silt	PP		1,500		19.5	109
52											
53											
54											
55					Consolidation Test, see Appendix C						
56	ST		psi 40 60 80 150 200								
57					very stiff to hard	PP		2,000		21.9	102
58											
59											
60											

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Project No.: 750664902

Figure: A-2b

PROJECT:	4590 PATRICK HENRY DRIVE Santa Clara, California	Log of Boring LB-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		6 11 13	17	CL	CLAY (CL) (continued) very stiff	PP		2,000			
62												
63												
64												
65							PP		1,250			
66	S&H		7 11 15	18	CL	SANDY CLAY (CL) olive-gray with yellow-brown mottling, very stiff, fine-grained sand, increased silt content	PP		1,750			
67												
68												
69												
70												
71	S&H		7 12 22	24		LL = 31, PI = 14, see Appendix C	PP		2,500	83.0	20.1	
72												
73												
74					CL	CLAY (CL) olive-gray with yellow-brown mottling, very stiff, wet, trace fine-grained sand						
75												
76	S&H		6 9 15	17			PP		>4,500			
77												
78												
79												
80												
81	S&H		8 14 19	23			PP		2,000			
82												
83												
84												
85												
86	S&H		9 13 20	23			PP		1,500			
87												
88												
89					CL	SANDY CLAY (CL) olive-gray, very stiff, wet, fine- to medium-grained sand, trace silt						
90												

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Project No.: 750664902

Figure: A-2c

PROJECT:	**4590 PATRICK HENRY DRIVE** Santa Clara, California	**Log of Boring LB-2** PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		8 10 15	18	CL	SANDY CLAY (CL) (continued)	PP		3,500			
92												
93												
94												
95												
96												
97												
98												
99												
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 91.5 feet below ground surface.
Boring backfilled with cement grout with a vibrating wire piezometer grouted in place at a depth of 34 feet.
Groundwater encountered at 7.5 feet below ground surface.
PP = pocket penetrometer.
bgs = below ground surface.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD88 datum and were determined from the plan titled "Existing Conditions," Sheet C1.0, by BKF, dated 6 June 2023.

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Project No.: 750664902 | Figure: A-2d

PROJECT:	4590 PATRICK HENRY DRIVE Santa Clara, California	Log of Boring LB-3

Boring location: See Site Plan, Figure 2

Date started: 06/15/22 | Date finished: 06/15/22

Drilling method: Mud Rotary

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: P. Marien

Drilled By: Pitcher Services, LLC

Ground Surface Elevation: 16.5 feet[2]

Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
				2.5 inches asphalt concrete (AC)						
				6 inches aggregate base (AB)						
GRAB			CL	CLAY (CL) dark brown to brown, moist, trace fine- to medium-grained sand, trace silt						
			CL	CLAY with SAND (CL) light brown to yellow-brown, moist, fine- to medium-grained sand, trace silt						
S&H	3 6 9	11	CL	CLAY (CL) olive-gray with yellow-brown and brown mottling, stiff, moist, trace fines to medium-grained sand LL = 40, PI = 19, see Appendix C (7:48am, 6/15/2022)						
GRAB			CL	SANDY CLAY (CL) brown to light brown, wet, fine-grained sand, trace silt						
S&H	3 3 7	7	SP	SAND (SP) brown, loose, wet, fine- to coarse-grained, trace fine subangular gravel, trace clay						
SPT	6 8 10	22		dark brown to brown, medium dense, decrease in fines content						
			CL	SANDY CLAY (CL) light brown to yellow-brown, stiff, wet, fine- to medium-grained sand, trace fine subangular gravel, trace silt						
ST		psi 100 200 350 180 100	SP-SC	CLAYEY SAND with GRAVEL (SP-SC) light brown to brown, wet, fine- to coarse-grained, fine to coarse subrounded to subangular gravel, moderate cohesion						
S&H	3 5 10	11	CL	CLAY (CL) light brown to brown, stiff, wet trace fine- to medium-grained sand, trace fine subrounded gravel	PP		1,000 to 1,750		22.8	106
S&H	7 15 21	25	CL	CLAY with SAND (CL) yellow-brown to light brown with brown mottling, very stiff, wet, fine-grained sand						
S&H	4 7 10	12	CL	CLAY (CL) olive-gray with yellow-brown mottling, stiff, wet, trace fine-grained sand	PP PP		1,250 750			
			CL	SANDY CLAY (CL) olive-gray with yellow-brown mottling, stiff, wet, fine-grained sand						
			SP-SC	SAND with CLAY (SP-SC)						

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Project No.: 750664902 | Figure: A-3a

Log of Boring LB-3

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		7 4 6	12		SAND with CLAY (SP-SC) (continued) brown, medium dense, wet, fine- to coarse-grained, trace fine subangular, nonplastic				12.7	16.7	
32–34					CL	CLAY with SAND (CL) olive-gray, wet, stiff, fine-grained sand						
35–39	S&H		9 14 20	24	CL	CLAY (CL) olive-gray, very stiff, wet, trace fine- to coarse-grained sand, trace silt	PP		3,500			
40–41	S&H		4 12 19	22		increase sand content						
41–44					CL	CLAY with SAND (CL) olive-gray with gray-brown and yellow-brown mottling, very stiff, wet, fine- to coarse-grained sand					18.3	115
45–47	S&H		13 23 26	34		olive-gray with yellow-brown mottling, hard, wet, fine-grained sand, trace fine subrounded gavel						
48–49	GRAB					olive-gray, trace fine-grained sand, trace silt						
50–54	S&H		13 22 27	34		CLAY with SAND (CL) olive-gray with gray-brown mottling, hard, wet	PP		3,000			
55–60	S&H		5 8 11	13	CL	olive-gray, stiff, decrease sand content, increase silt content	PP		1,750			

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	ST			psi 40 80 180 210 260	CL	CLAY with SAND (CL) (continued) Consolidation Test, see Appendix C					22.7	102
62–64					CL	CLAY (CL) olive-gray, wet, trace, fine-grained sand	PP		1,000			
65–66	SPT		3 5 7	14	SC	CLAYEY SAND (SC) olive-gray, medium dense, wet, fine-grained sand, trace silt LL= 25, PI = 9, see Appendix C				44.8	22.0	
67–68					CL	CLAY (CL) olive-gray, stiff, wet, trace fine- to coarse-grained sand						
69–71	S&H		7 10 12	15	CL	CLAY with SAND (CL) olive-gray, stiff, wet, trace fine- to coarse-grained sand trace organics	PP		1,250			
75–76	S&H		11 19 24	30		very stiff to hard	PP		2,000			
79–82	SPT		19 27 33	72	SP-SM	SAND with SILT (SP-SM) gray-brown to brown, very dense, wet, fine- to medium-grained						
83–89	S&H		10 15 16	22	CL	CLAY (CL) olive-gray with gray-brown mottling, very stiff, wet, trace fine-grained sand	PP		3,750			
90					CL							

PROJECT:	**4590 PATRICK HENRY DRIVE** Santa Clara, California	**Log of Boring LB-3** PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		9 13 27	28	CL	CLAY with SAND (CL) olive-gray, very stiff, wet, fine-grained sand	PP		>4,500			
92												
93												
94												
95												
96												
97												
98												
99												
100												
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114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 91.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 7 feet below ground surface.
PP = pocket penetrometer.
bgs = below ground surface.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD88 datum and were determined from the plan titled "Existing Conditions," Sheet C1.0, by BKF, dated 6 June 2023.

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Project No.: 750664902 | Figure: A-3d

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

PP = Pocket Penetrometer

TV = Torvane

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push or Drive sampler

Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter - see report text

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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Project	Figure Title	Project No. 750664902	Figure
4590 PATRICK HENRY DRIVE SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	SOIL CLASSIFICATION CHART	Date 07/31/2023 Drawn By AG Checked By PB	A-4

APPENDIX B

CONE PENETRATION TEST RESULTS

PRESENTATION OF SITE INVESTIGATION RESULTS

4590 Patrick Henry

Prepared for:

Langan Engineering

ConeTec Job No: 22-56-24292

Project Start Date: 2022-Jun-13
Project End Date: 2022-Jun-13
Report Date: 2022-Jun-15

Prepared by:

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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 Seismic 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	Langan Engineering
Project	4590 Patrick Henry
ConeTec Project Number	22-56-24292
Rig Description	30-ton Truck CPT Rig (C-15)

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 4590 Patrick Henry
- The Report was prepared by ConeTec for Langan Engineering

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 Langan Engineering
- The "Report" refers to this report titled 4590 Patrick Henry
- ConeTec was retained to collect and provide the raw data ("Data") which is included in the Report.

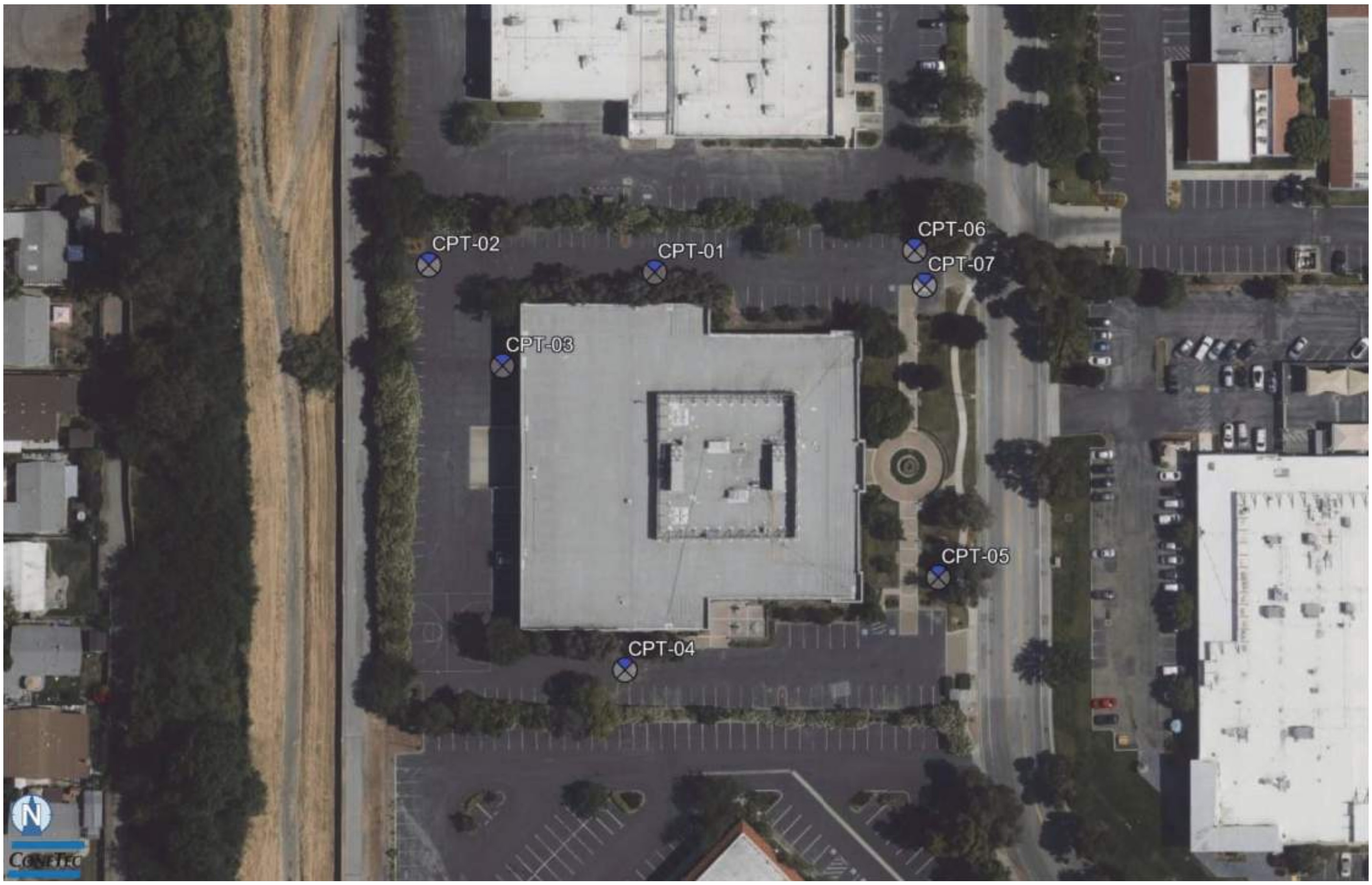
ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other 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, Advanced Plots, and Normalized Plots**
- **SBT Zone Scatter Plots**
- **Pore Pressure Dissipation (PPD) Test Summary**
- **PPD Test Plots**
- **Seismic CPTu Results, Plots, and Traces**
- **Methodology Statements**
- **Data File Formats**
- **Description of Methods for Calculated CPT Geotechnical Parameters**

SITE MAP



ConeTec Job Number: 22-56-24292

Client: Langan Engineering

Project: 4590 Patrick Henry

Report Date: 2022-Jun-15

Sounding Location

All sounding locations are approximate

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No:	22-56-24292
Client:	Langan Engineering
Project:	4590 Patrick Henry
Start Date:	13-Jun-2022
End Date:	13-Jun-2022

CONE PENETRATION TEST SUMMARY

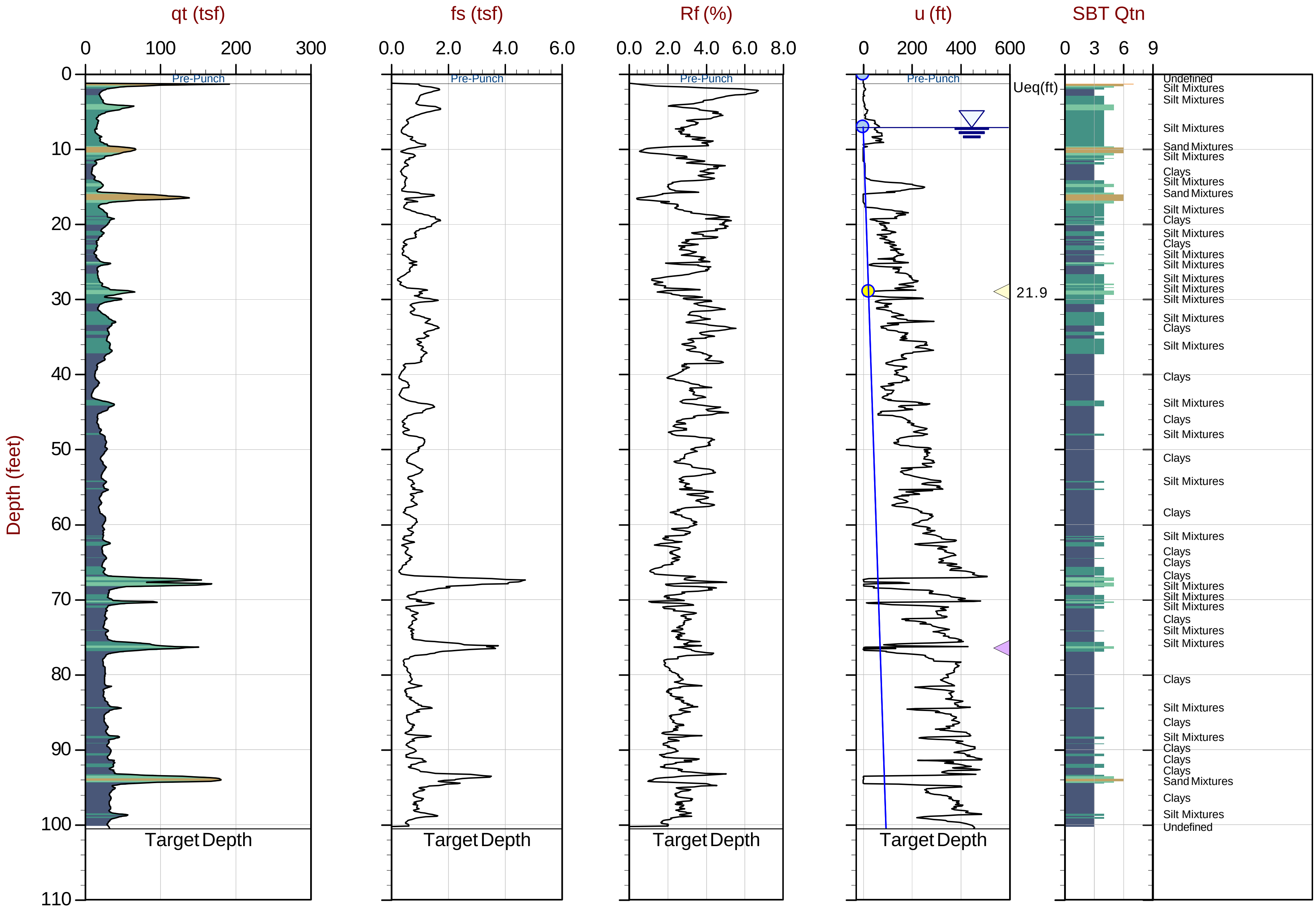
Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
CPT-01	22-56-24292_SP01	13-Jun-2022	EC811:T1500F15U35	15	7.1	100.56	4139427	589766	17	
CPT-02	22-56-24292_CP02	13-Jun-2022	EC811:T1500F15U35	15	6.5	70.54	4139428	589721	16	
CPT-03	22-56-24292_CP03	13-Jun-2022	EC811:T1500F15U35	15	7.9	70.54	4139408	589736	17	
CPT-04	22-56-24292_CP04	13-Jun-2022	EC811:T1500F15U35	15	7.0	70.54	4139348	589761	16	
CPT-05	22-56-24292_CP05	13-Jun-2022	EC811:T1500F15U35	15	9.5	70.54	4139367	589823	18	
CPT-06	22-56-24292_CP06	13-Jun-2022	EC811:T1500F15U35	15	7.6	70.54	4139432	589818	16	
CPT-07	22-56-24292_CP07	13-Jun-2022	EC811:T1500F15U35	15	8.0	75.62	4139425	589820	16	

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.

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35



Max Depth: 30.650 m / 100.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139427m E: 589766

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

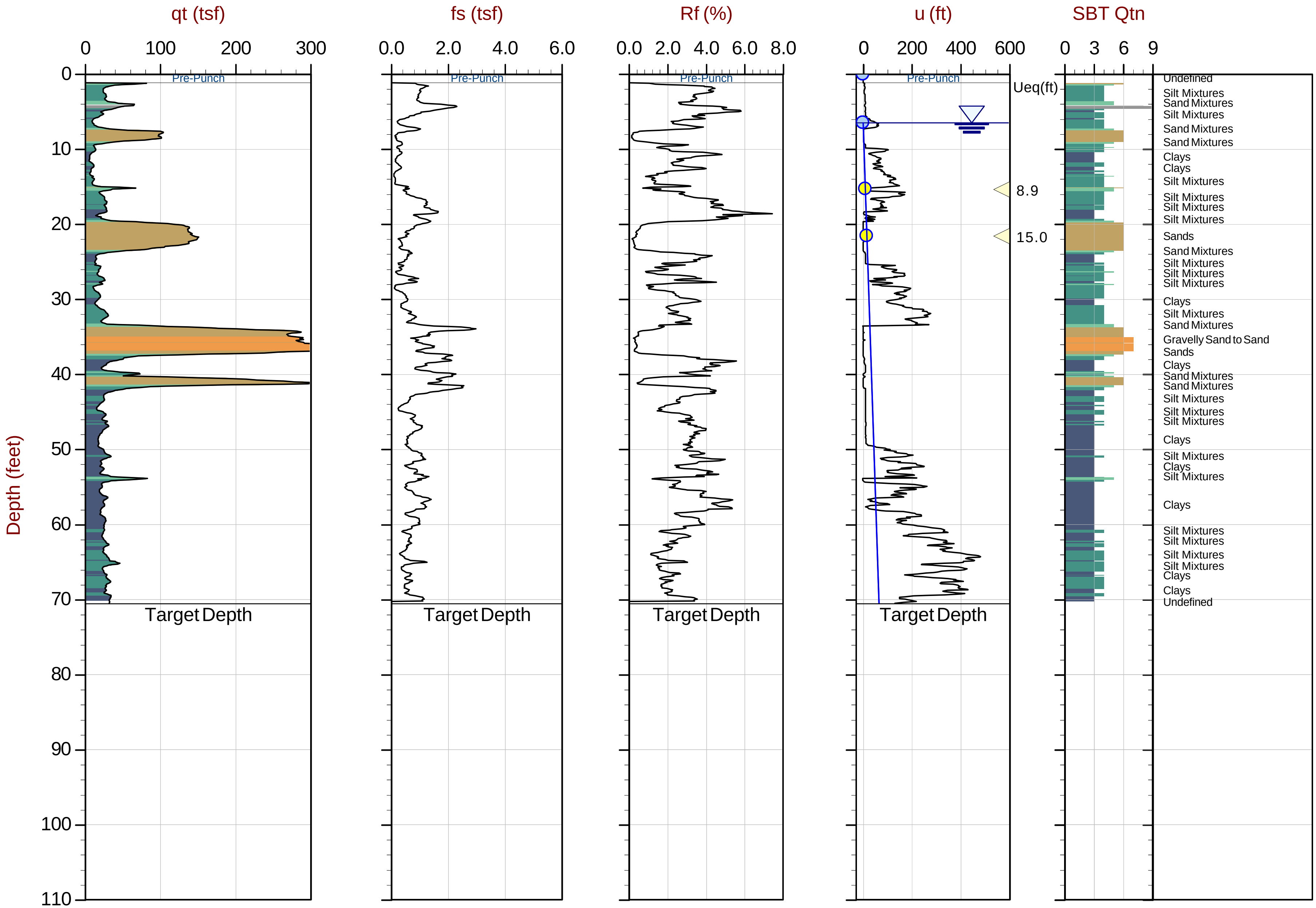
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139428m E: 589721

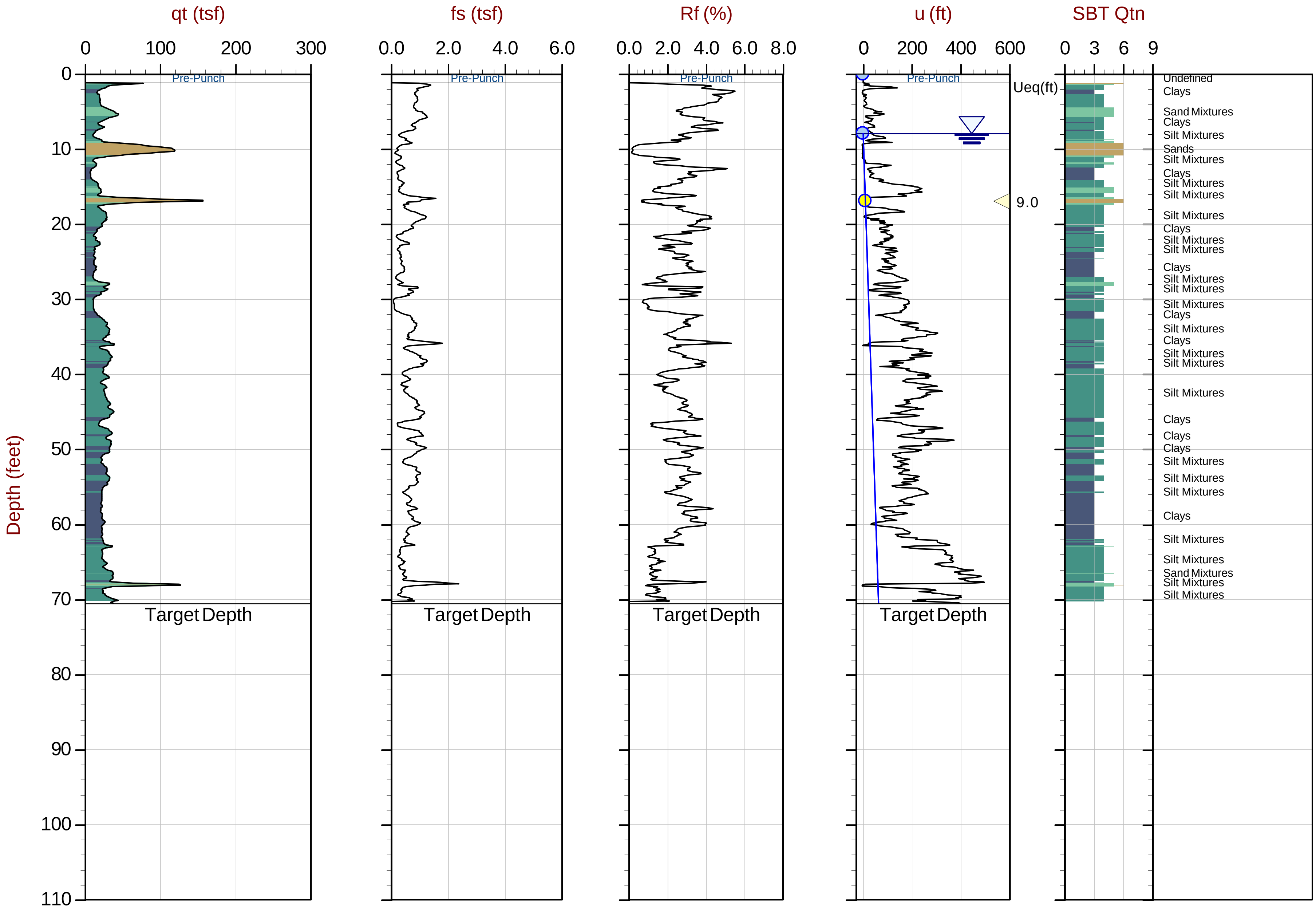
Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:02
Site: 4590 Patrick Henry

Sounding: CPT-03
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139408m E: 589736

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

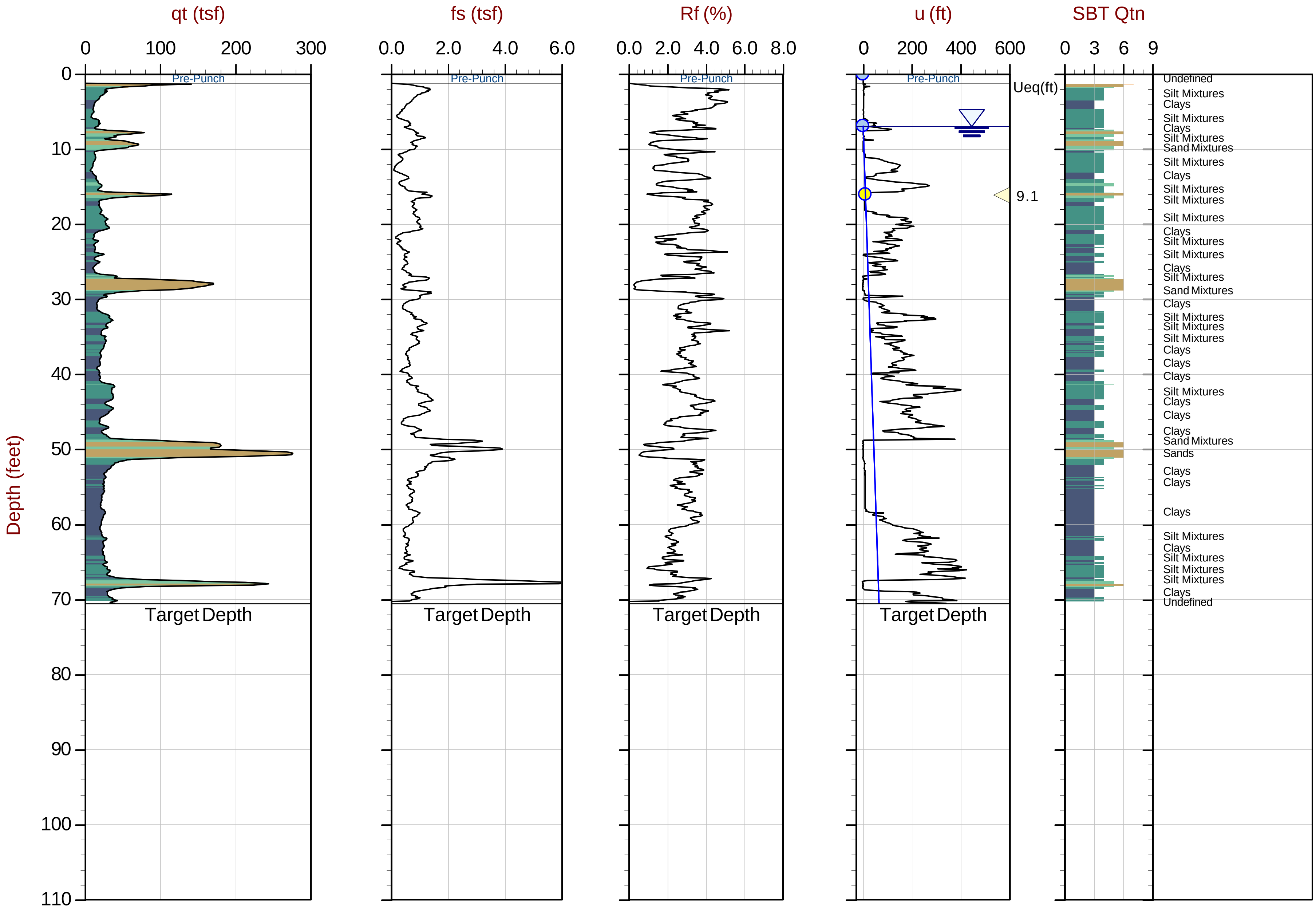
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:58
Site: 4590 Patrick Henry

Sounding: CPT-04
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139348m E: 589761

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

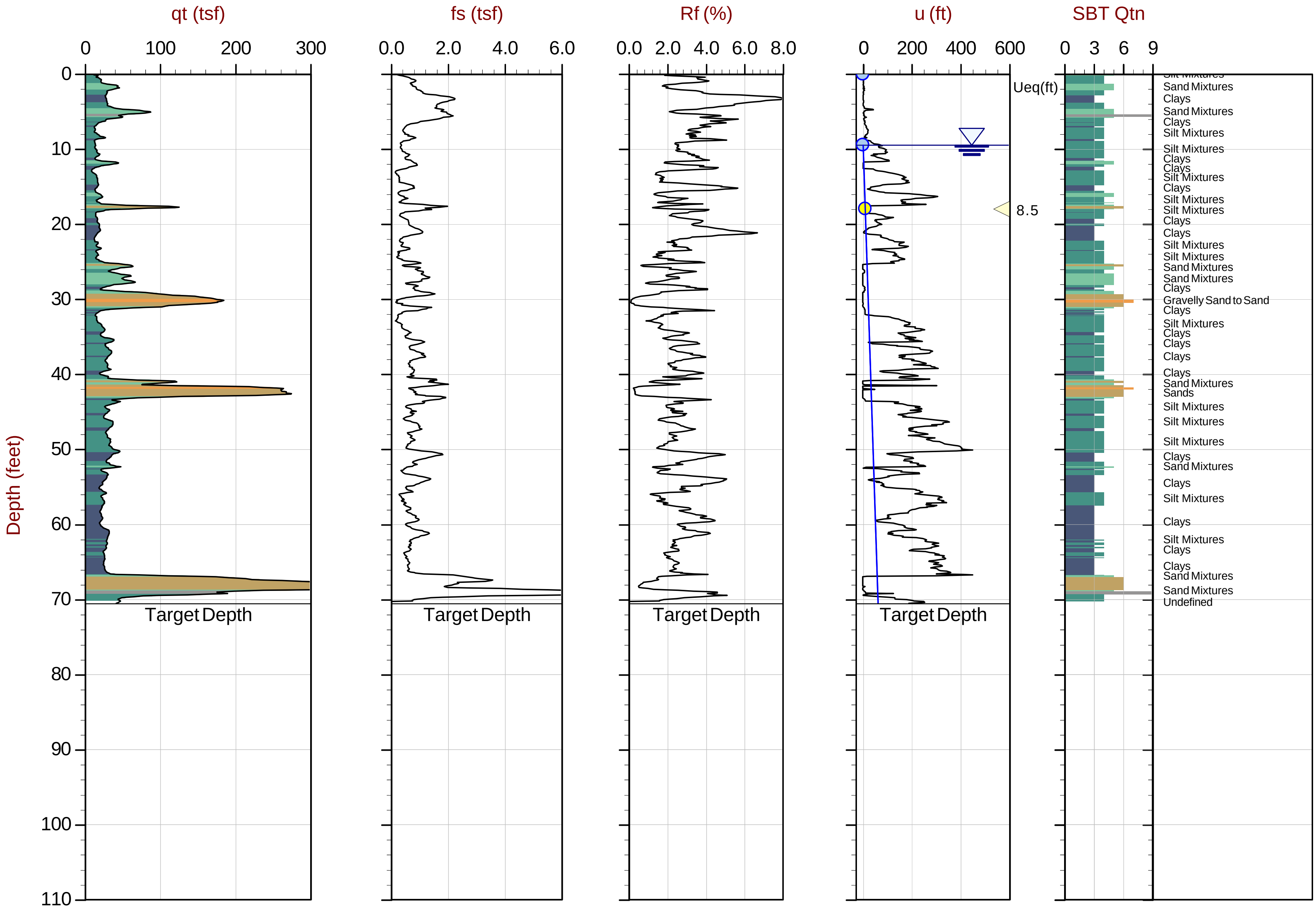
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 12:56
Site: 4590 Patrick Henry

Sounding: CPT-05
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139367m E: 589823

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

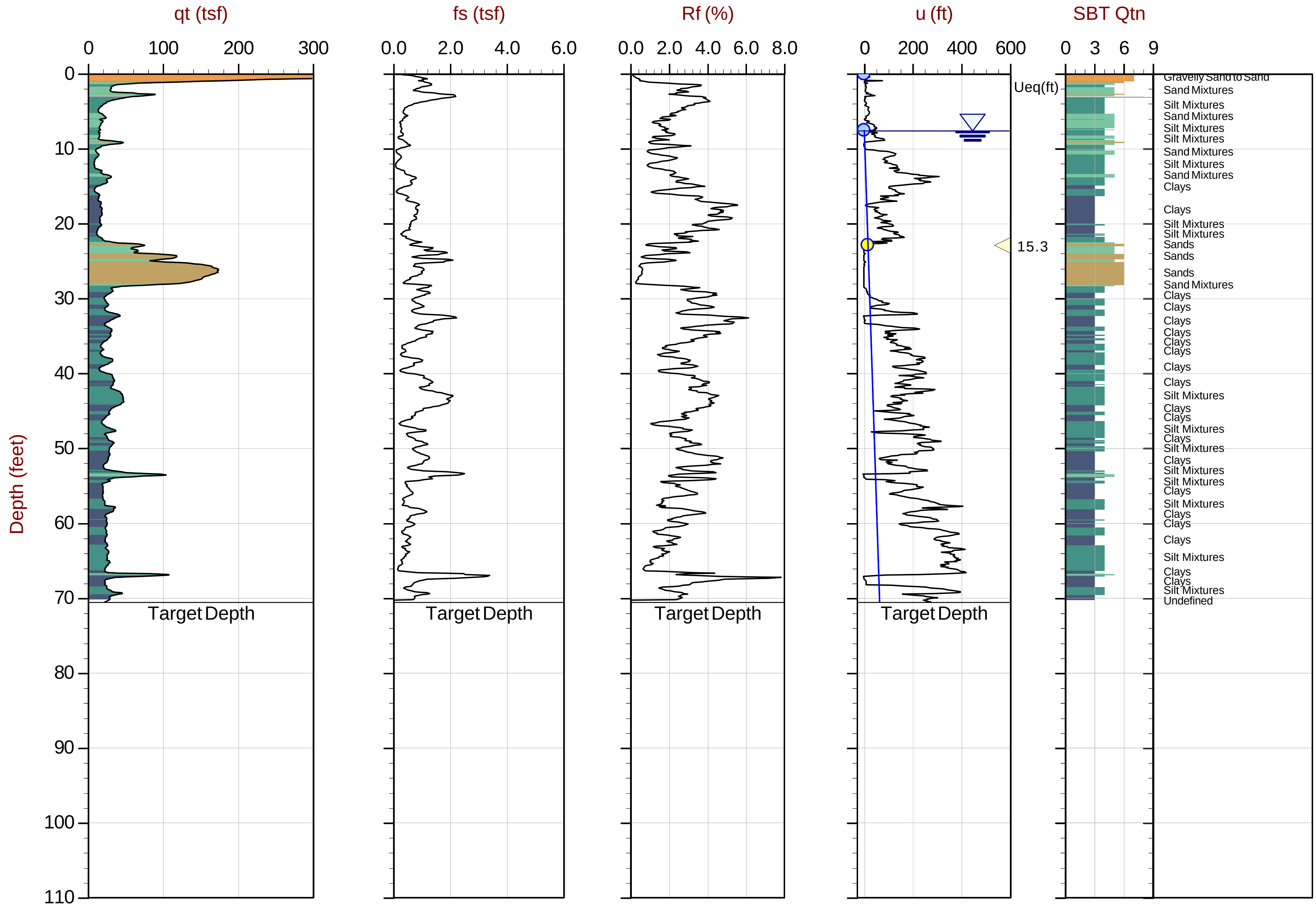
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 13:55
Site: 4590 Patrick Henry

Sounding: CPT-06
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139432m E: 589818

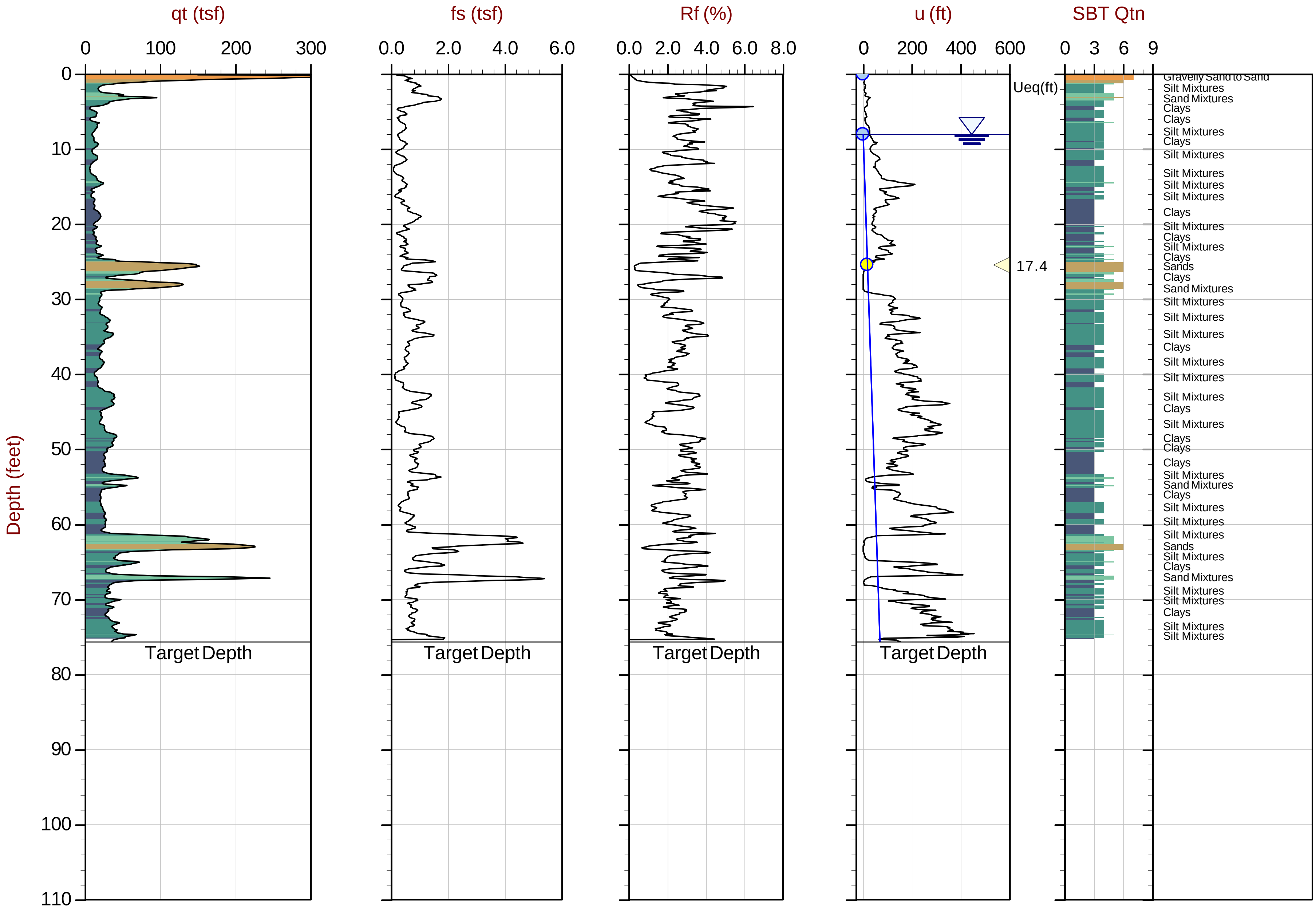
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 15:14
Site: 4590 Patrick Henry

Sounding: CPT-07
Cone: 811:T1500F15U35



Max Depth: 23.050 m / 75.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139425m E: 589820

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.

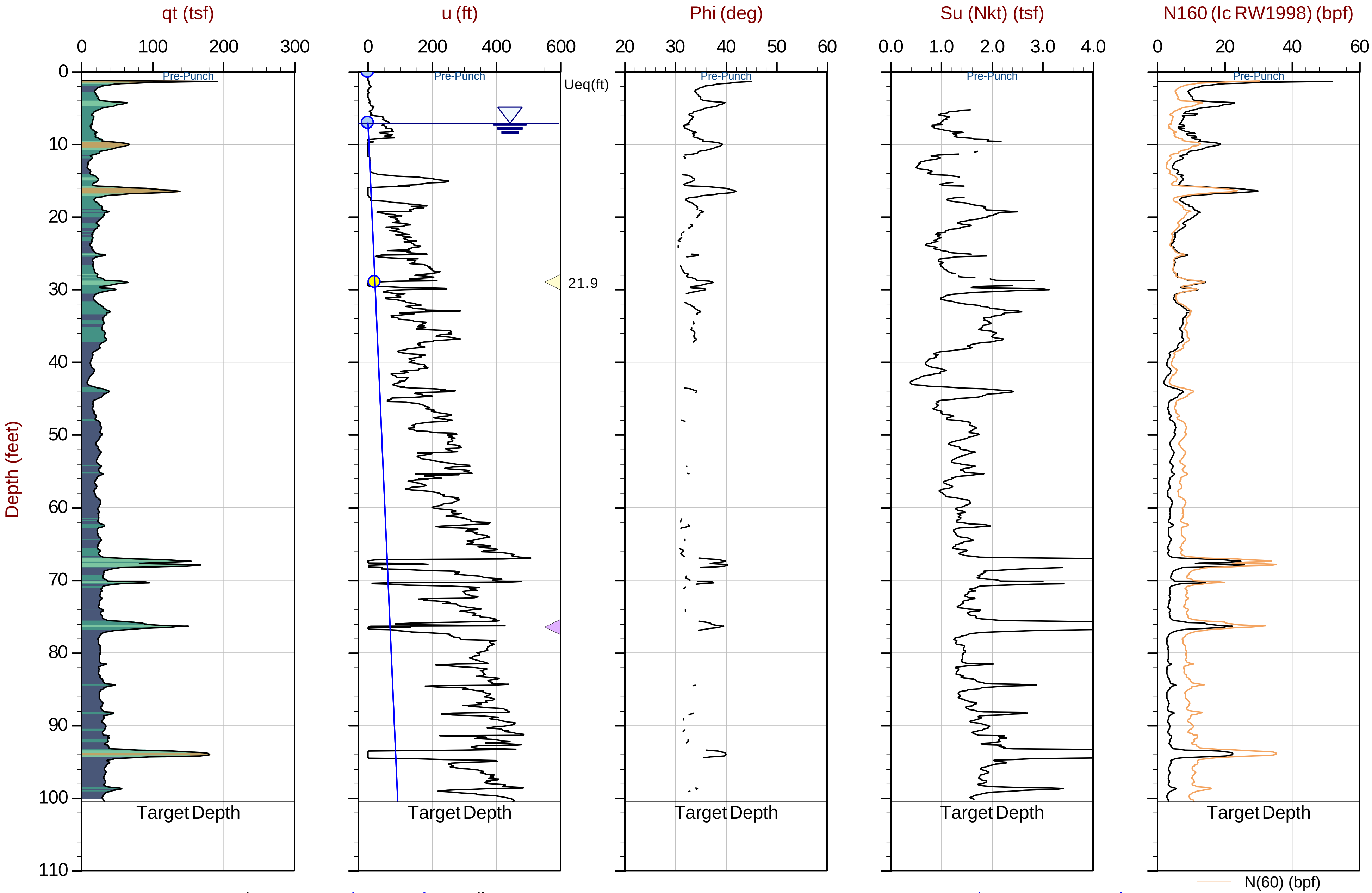
Advanced Cone Penetration Test Plots

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35



Max Depth: 30.650 m / 100.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139427m E: 589766

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

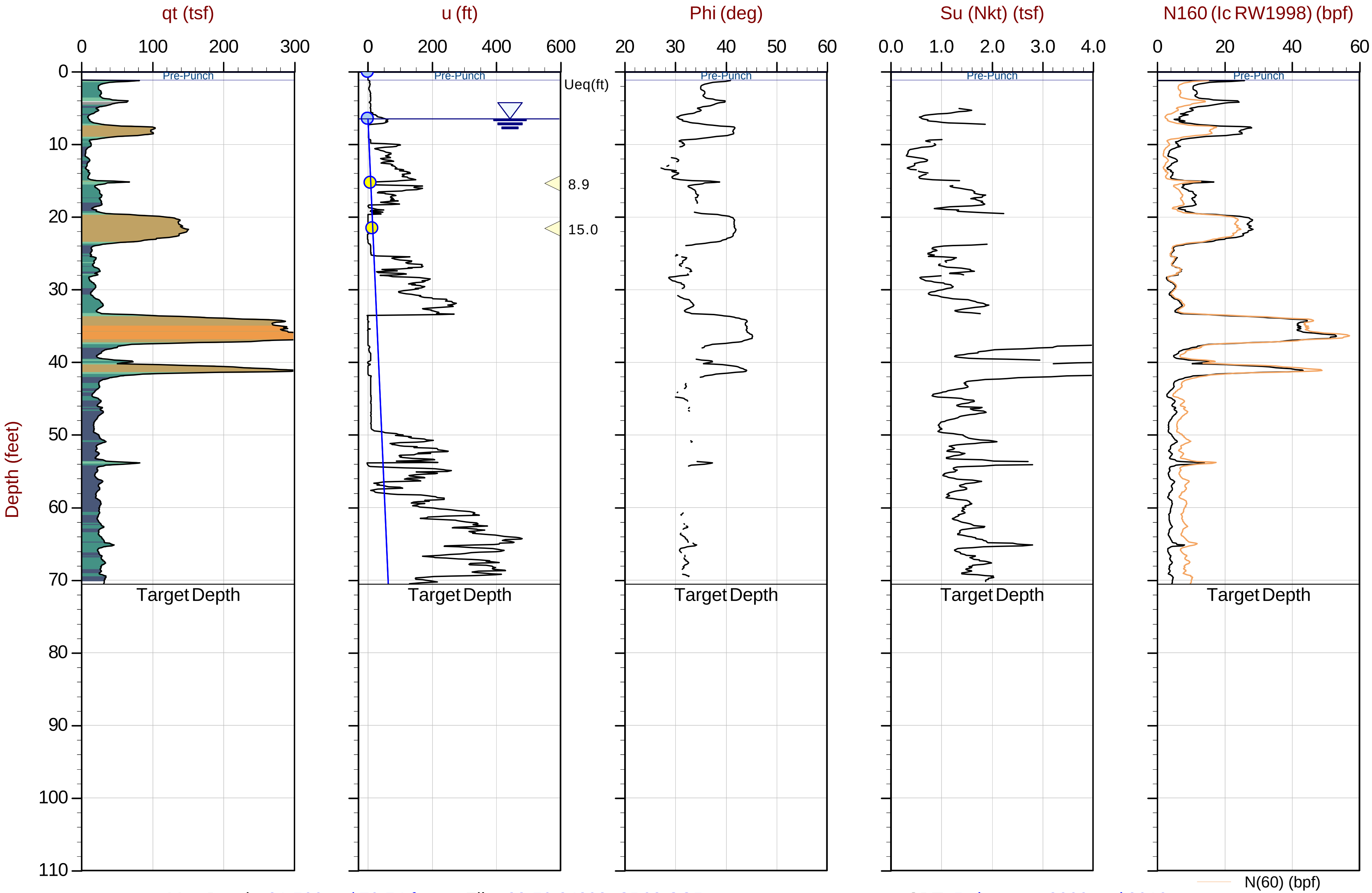
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139428m E: 589721

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

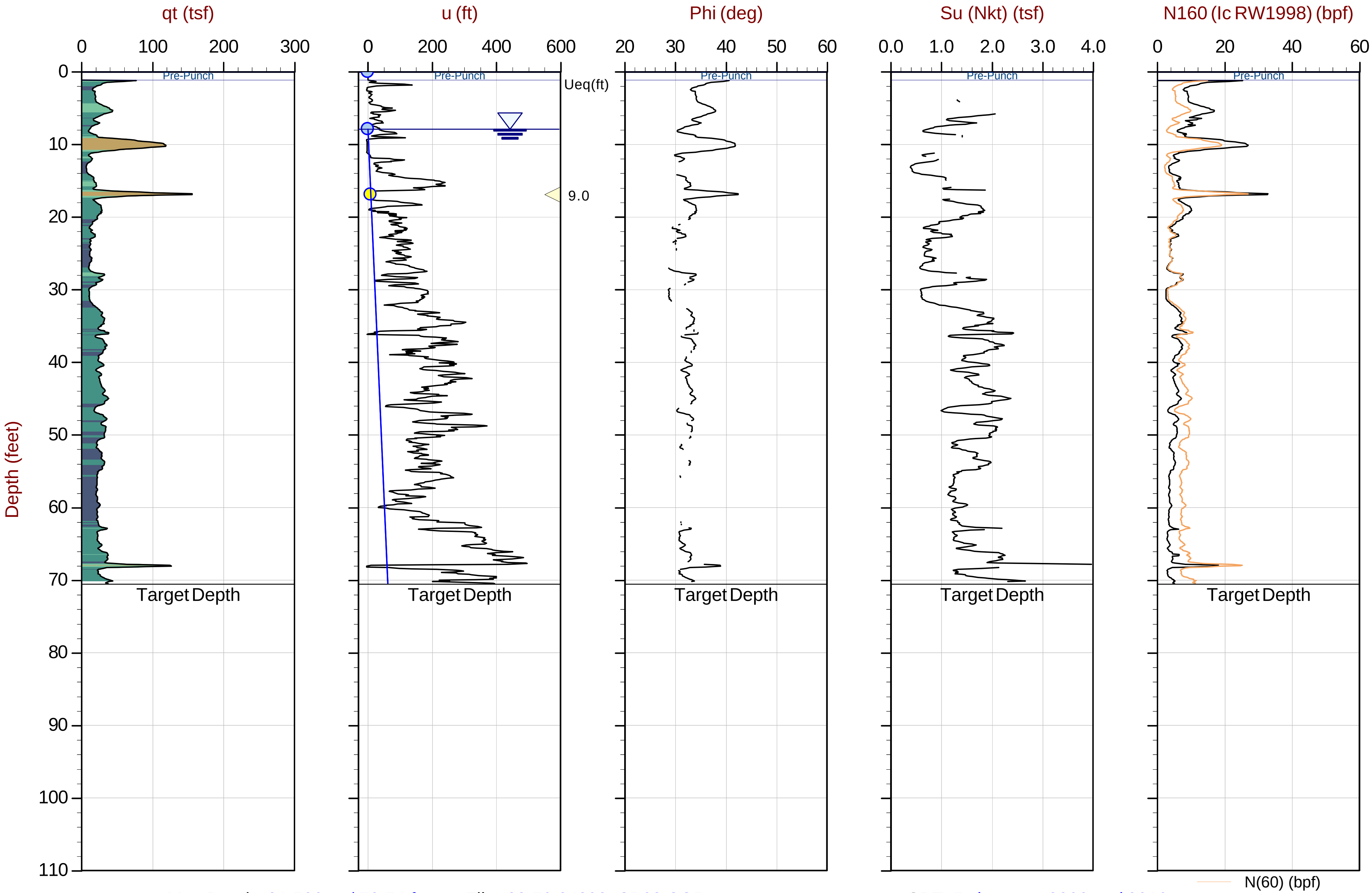
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:02
Site: 4590 Patrick Henry

Sounding: CPT-03
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139408m E: 589736

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

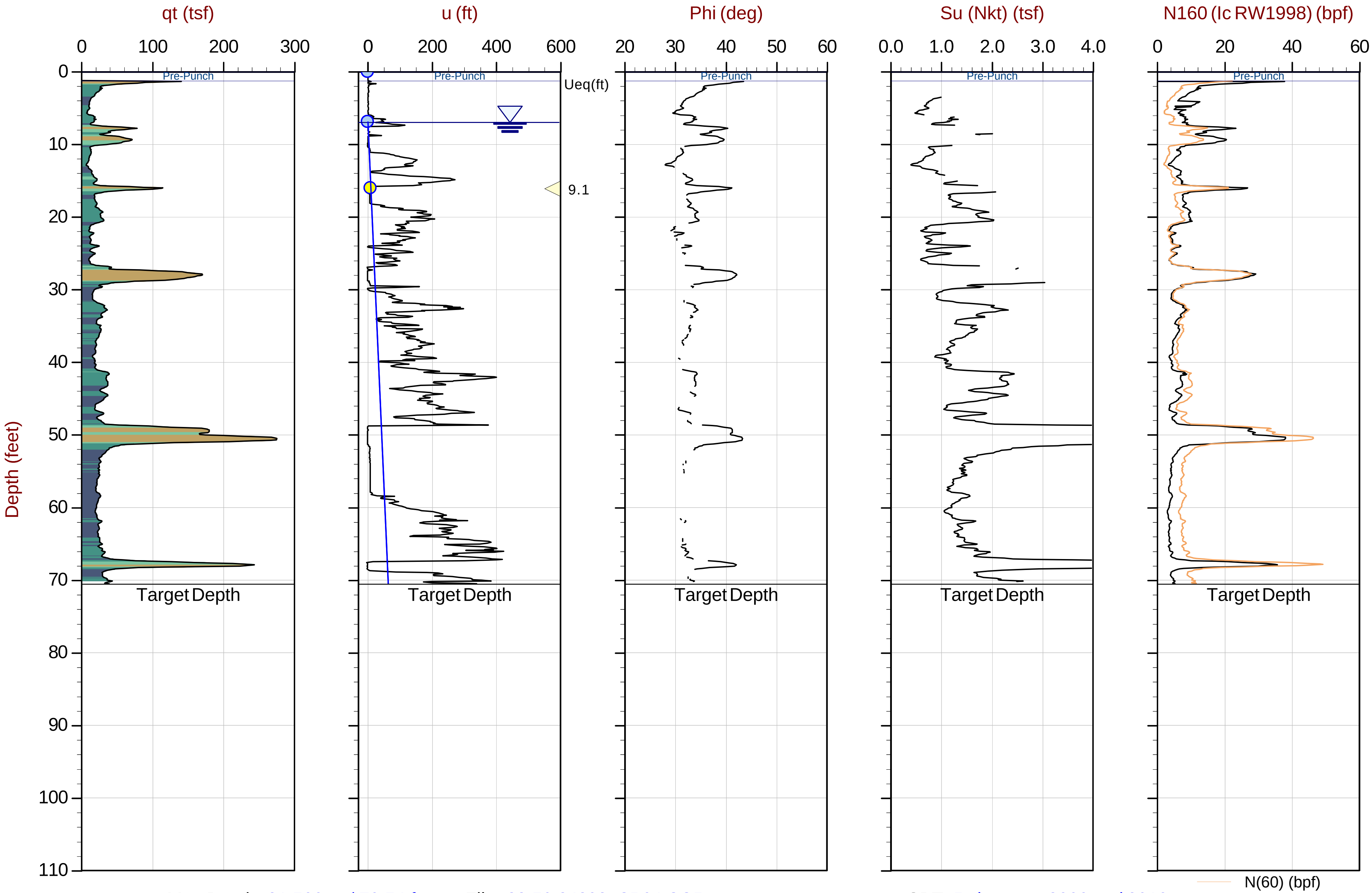
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:58
Site: 4590 Patrick Henry

Sounding: CPT-04
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139348m E: 589761

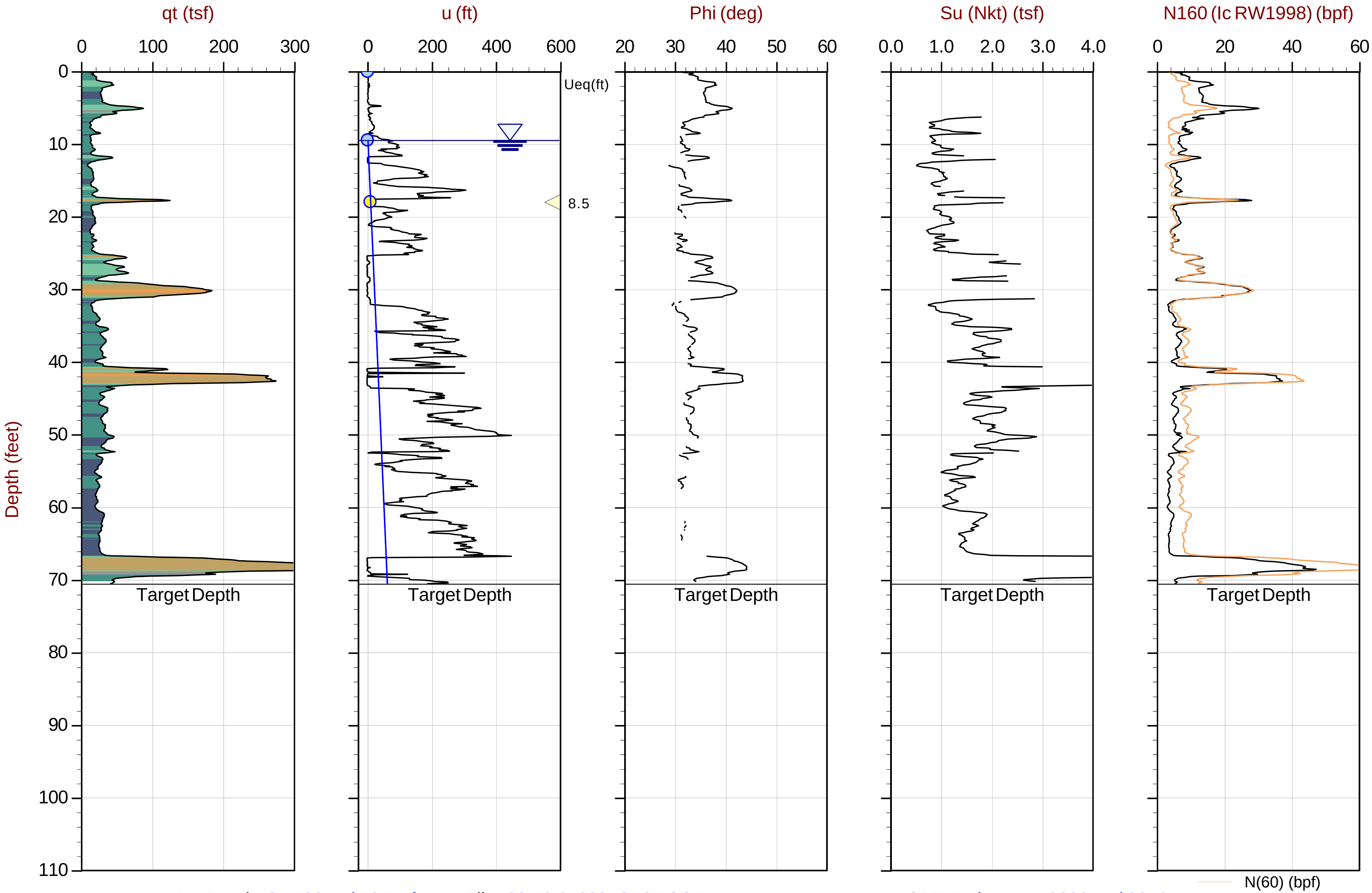
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 12:56
Site: 4590 Patrick Henry

Sounding: CPT-05
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139367m E: 589823

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

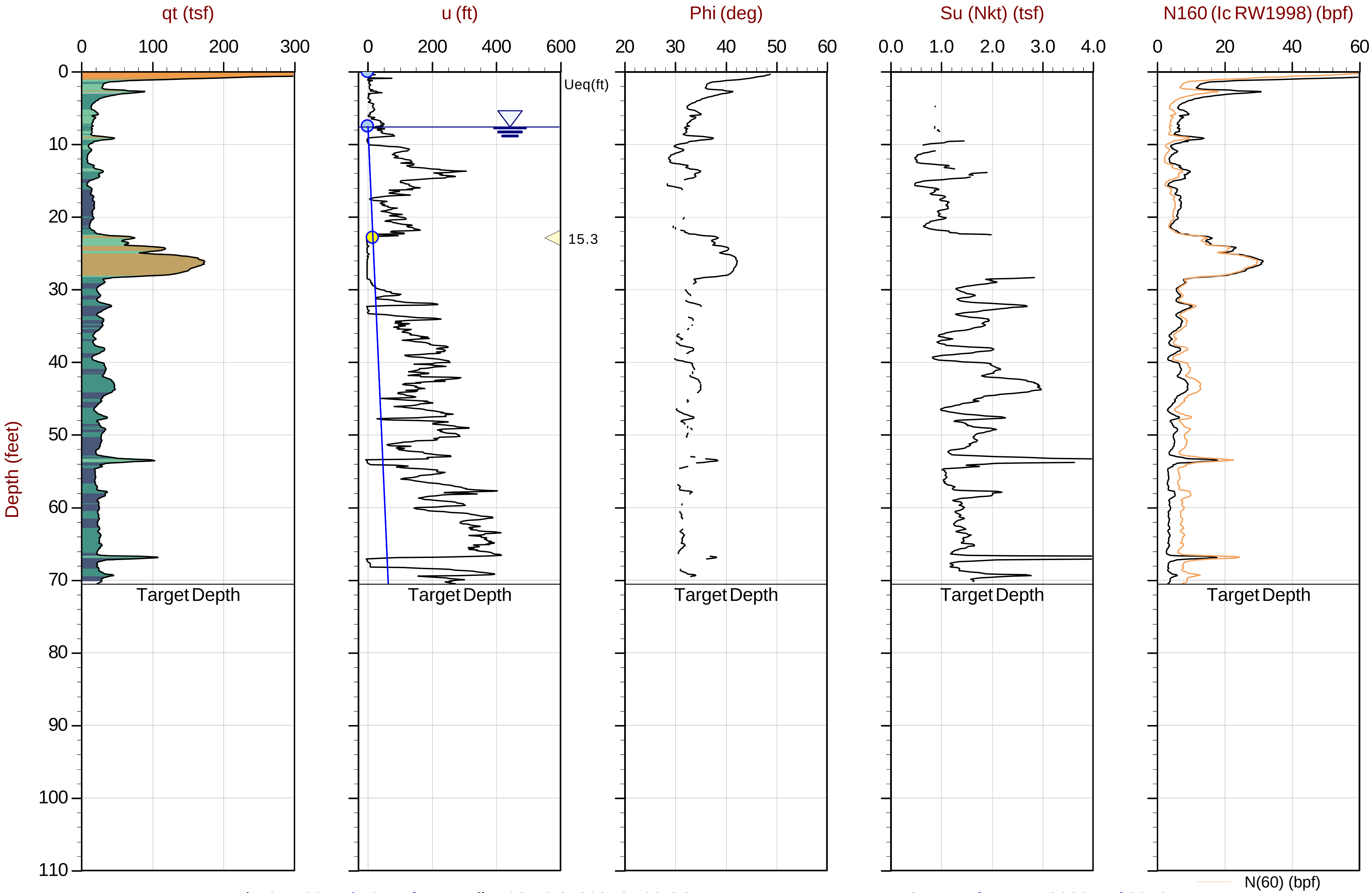
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 13:55
Site: 4590 Patrick Henry

Sounding: CPT-06
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139432m E: 589818

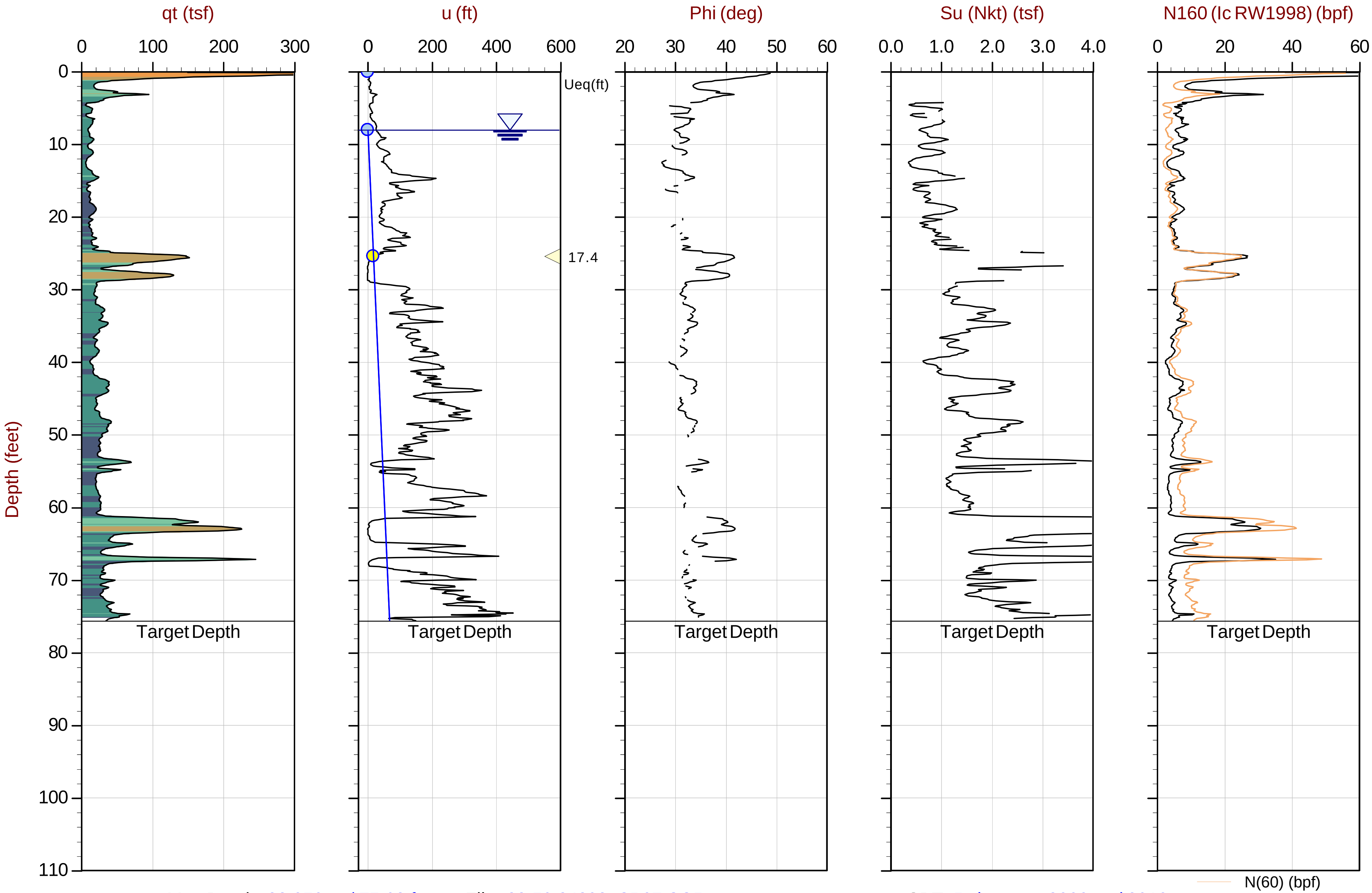
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 15:14
Site: 4590 Patrick Henry

Sounding: CPT-07
Cone: 811:T1500F15U35



Max Depth: 23.050 m / 75.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP07.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139425m E: 589820

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.

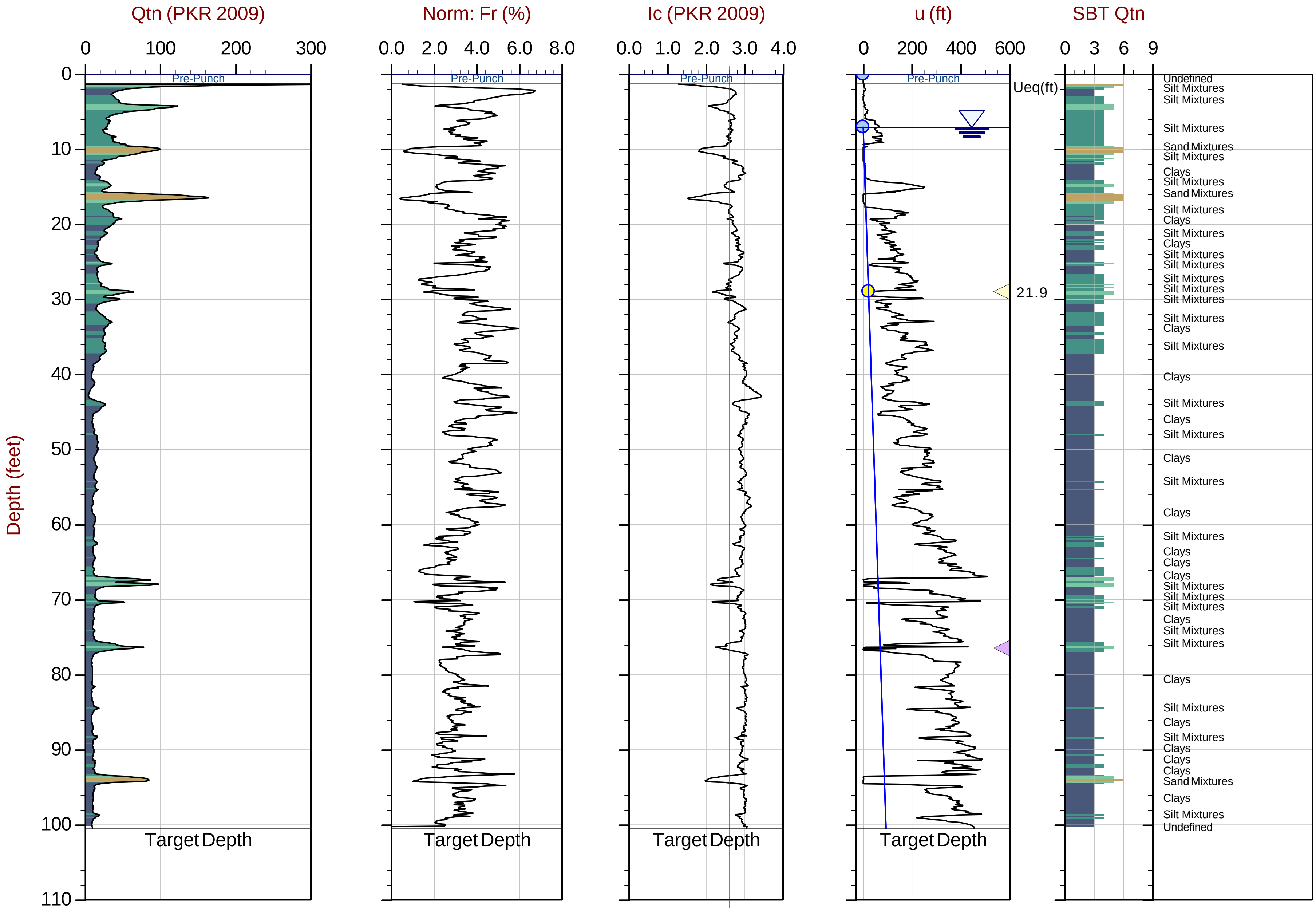
Normalized Cone Penetration Test Plots

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35



Max Depth: 30.650 m / 100.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139427m E: 589766

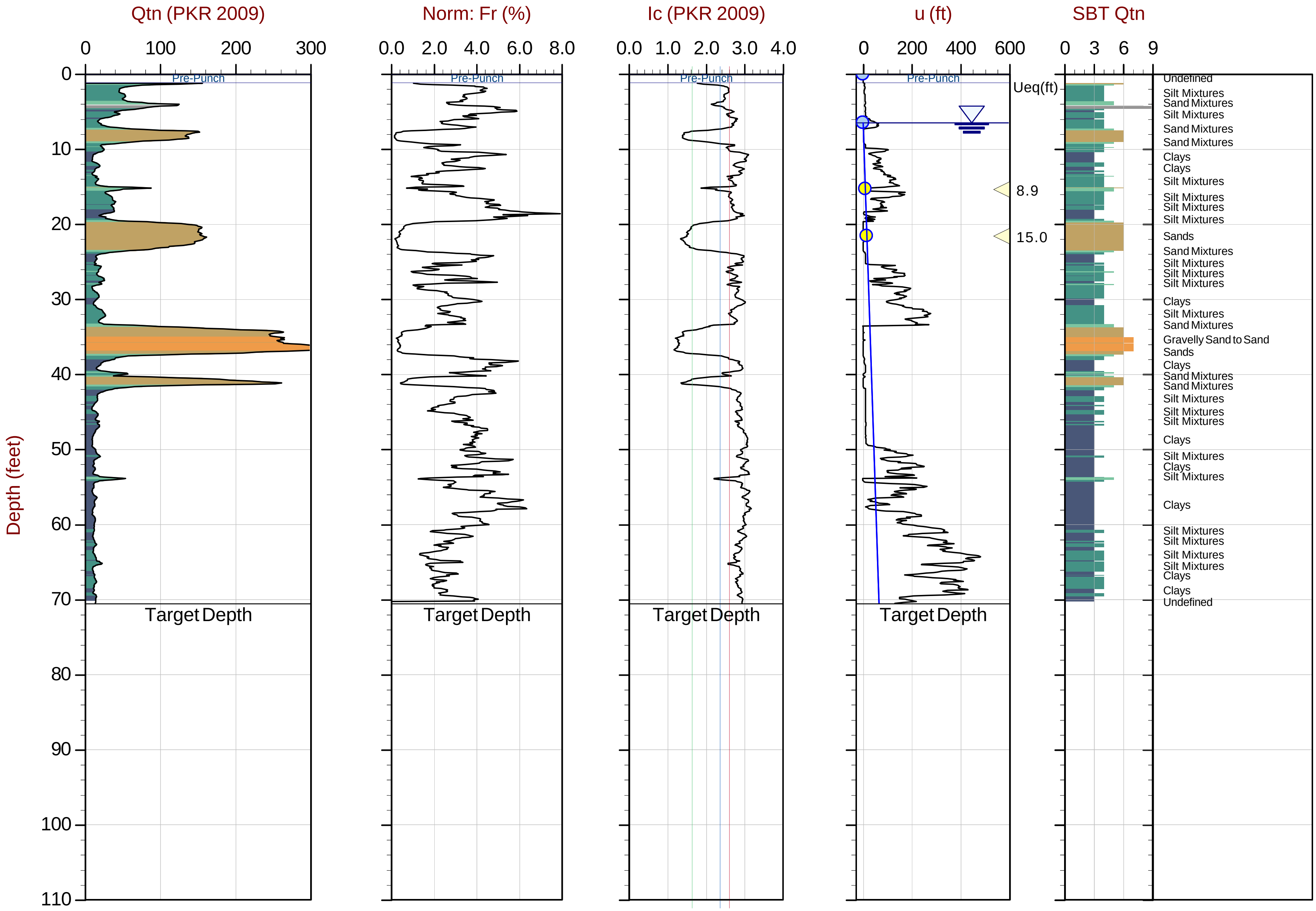
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139428m E: 589721

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

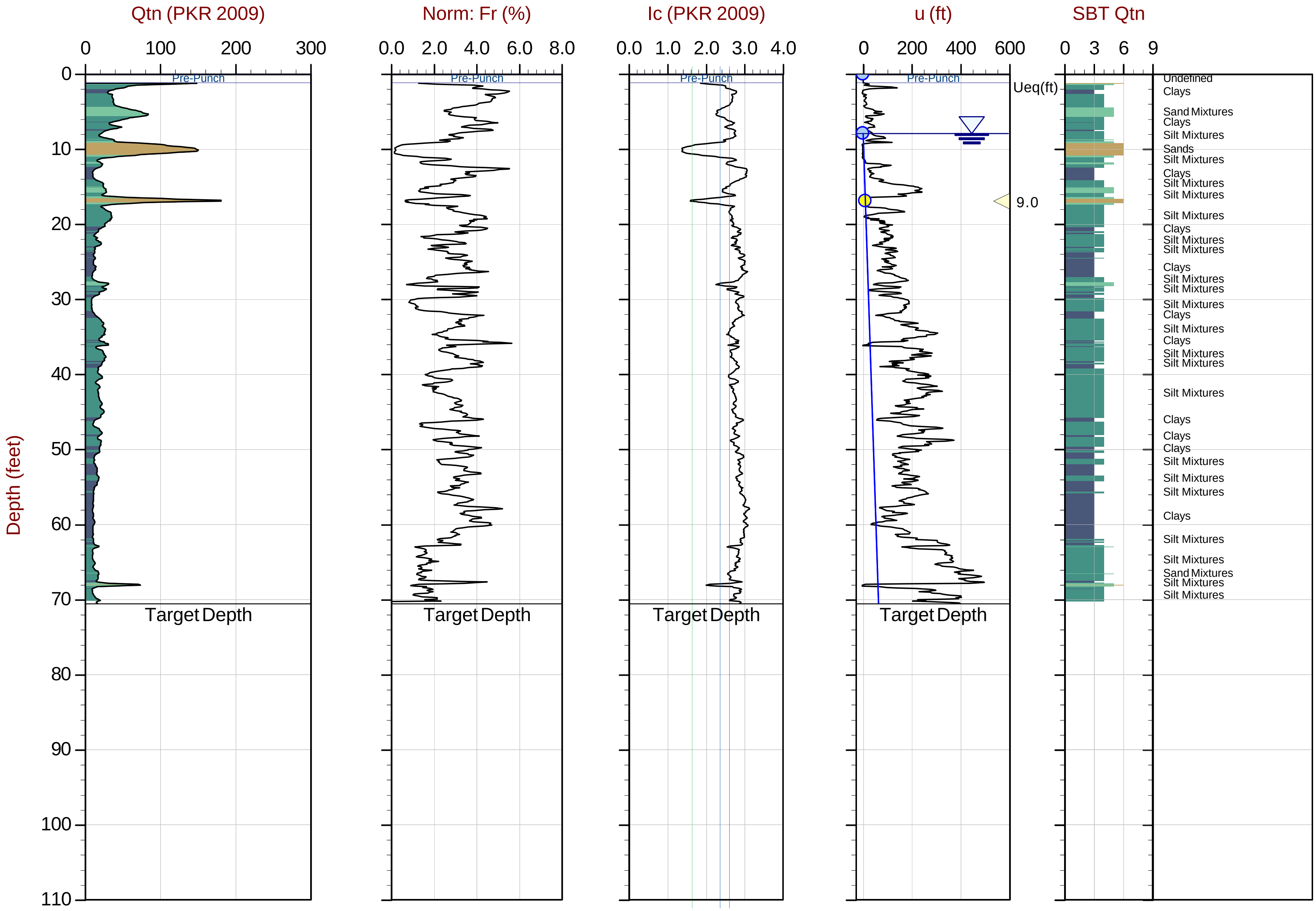
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:02
Site: 4590 Patrick Henry

Sounding: CPT-03
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

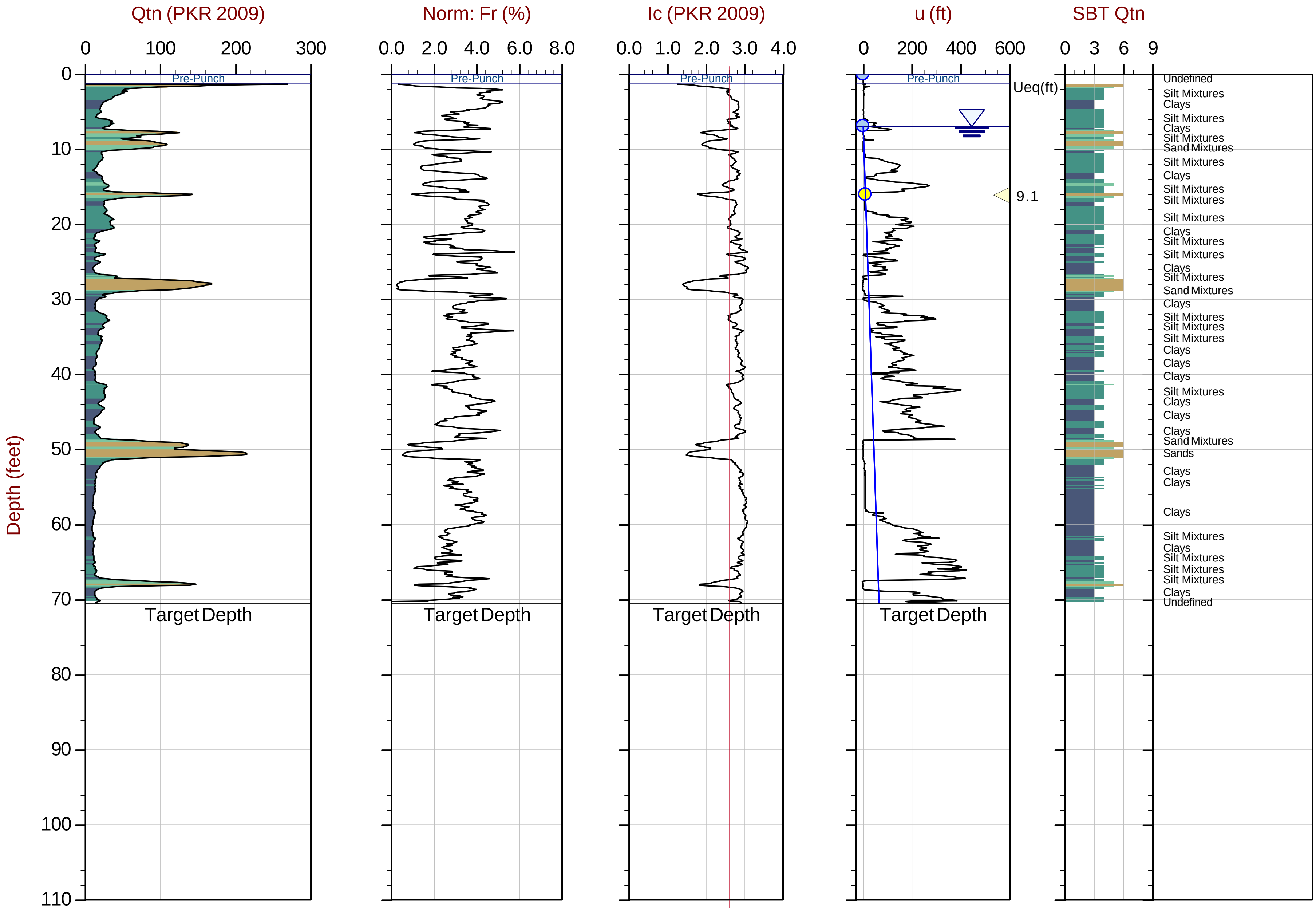
File: 22-56-24292_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139408m E: 589736

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.

Langan Engineering	Job No: 22-56-24292 Date: 2022-06-13 11:58 Site: 4590 Patrick Henry	Sounding: CPT-04 Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139348m E: 589761

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

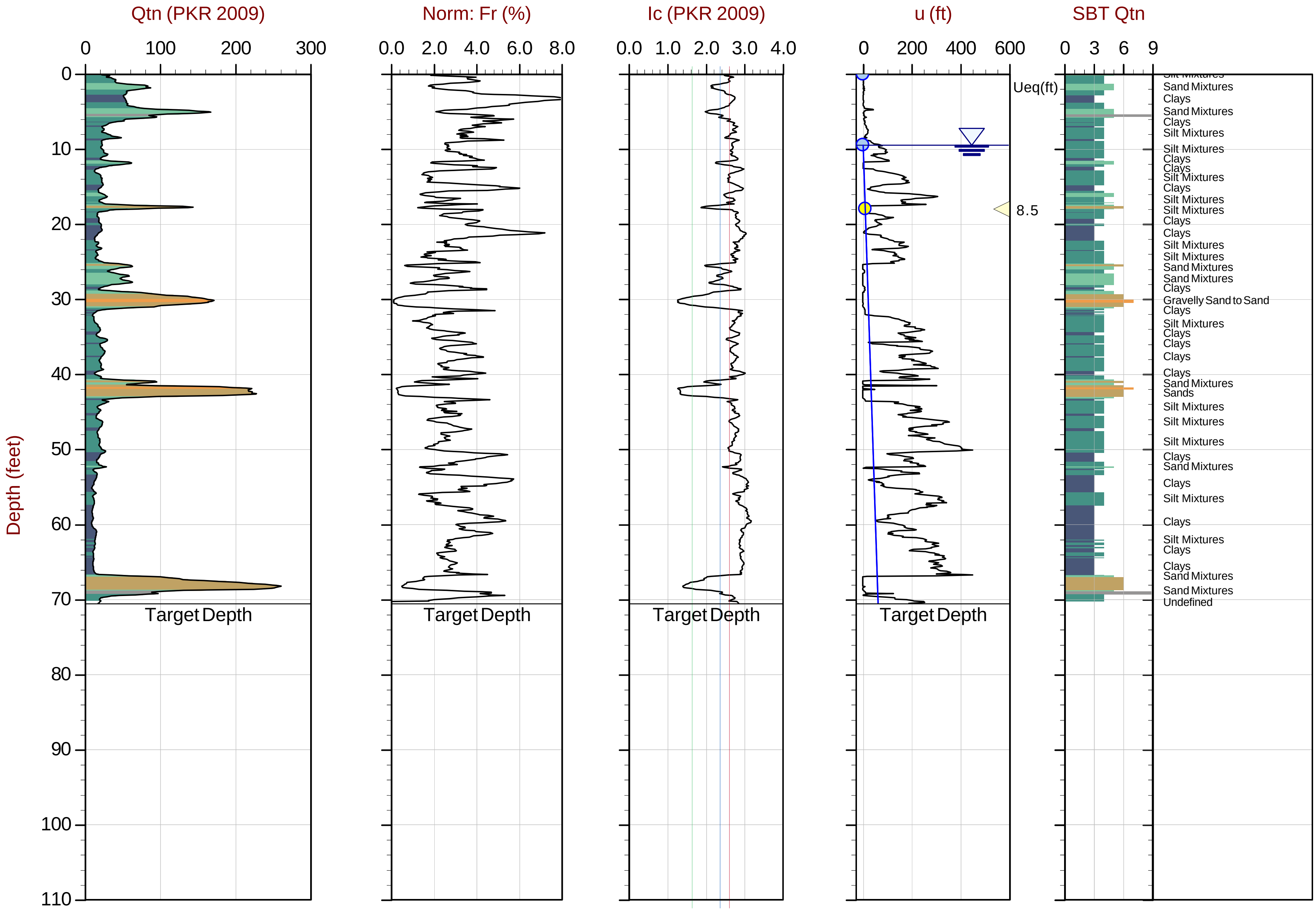
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 12:56
Site: 4590 Patrick Henry

Sounding: CPT-05
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139367m E: 589823

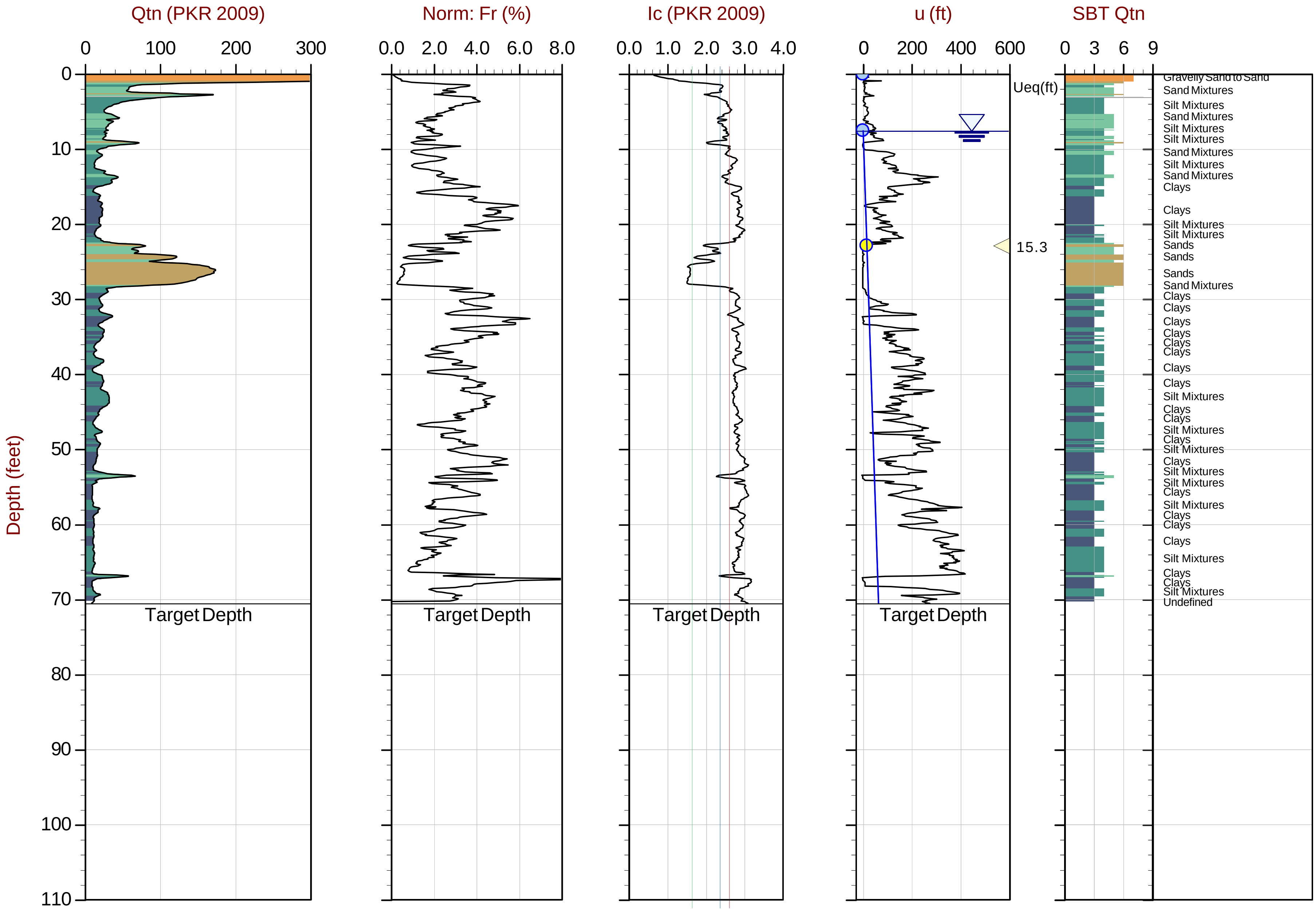
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 13:55
Site: 4590 Patrick Henry

Sounding: CPT-06
Cone: 811:T1500F15U35



Max Depth: 21.500 m / 70.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139432m E: 589818

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

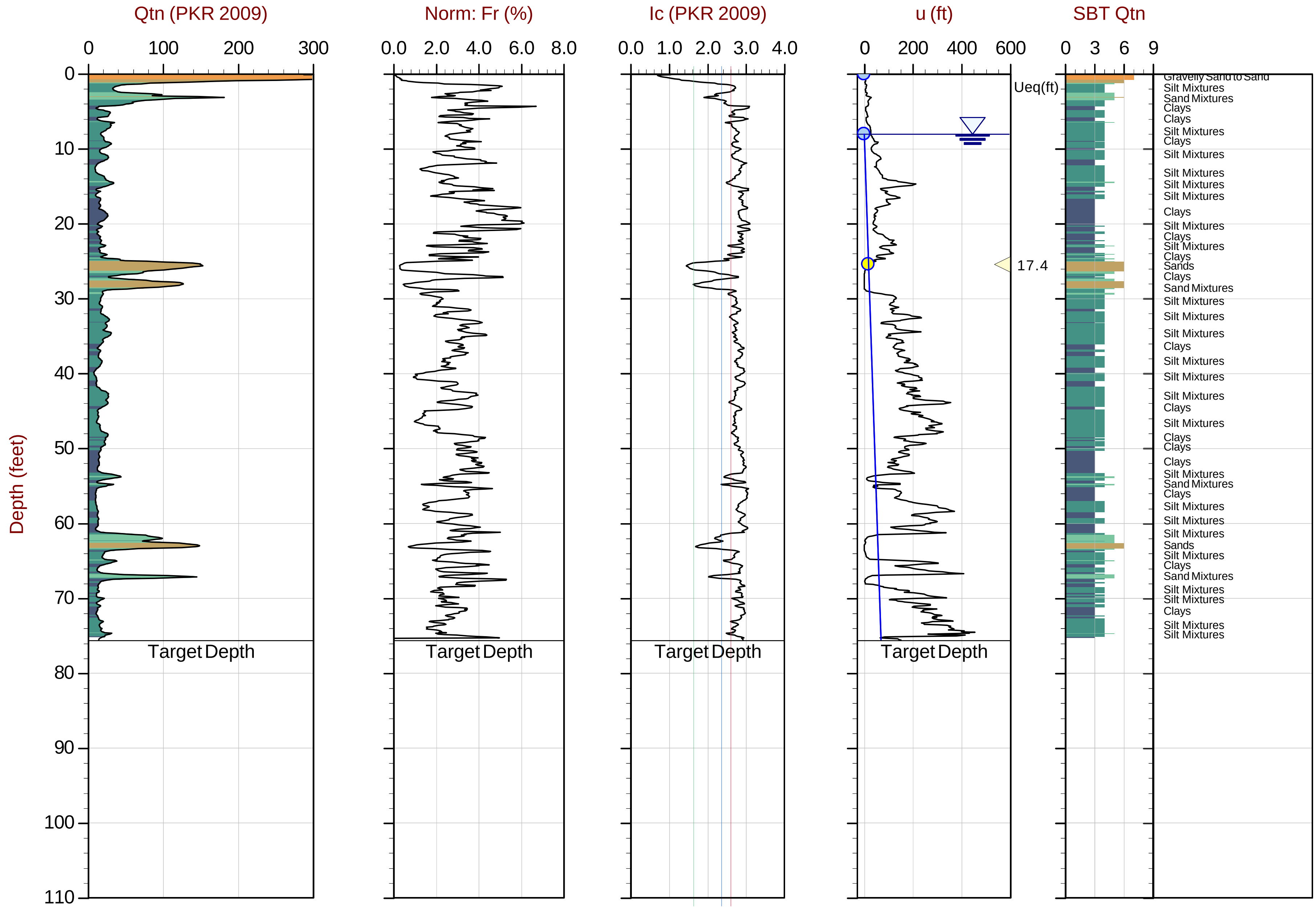
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 15:14
Site: 4590 Patrick Henry

Sounding: CPT-07
Cone: 811:T1500F15U35



Max Depth: 23.050 m / 75.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139425m E: 589820

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

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

Soil Behavior Type (SBT) Scatter Plots

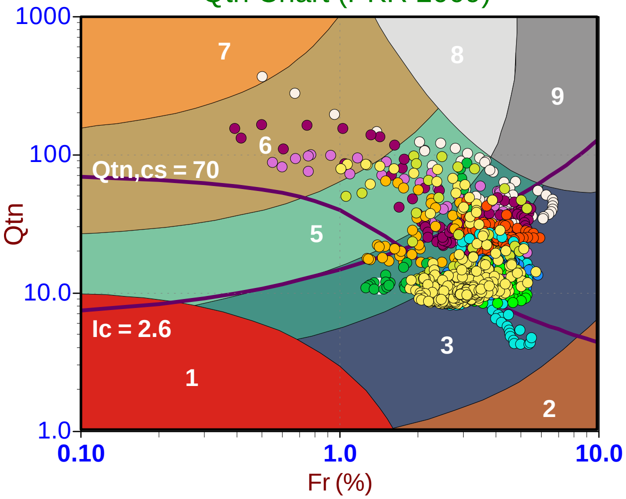
CONETEC

Langan Engineering

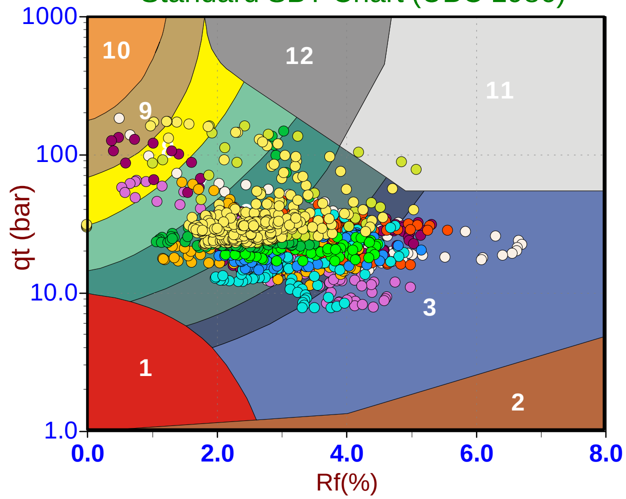
Job No: 22-56-24292
Date: 2022-06-13 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35

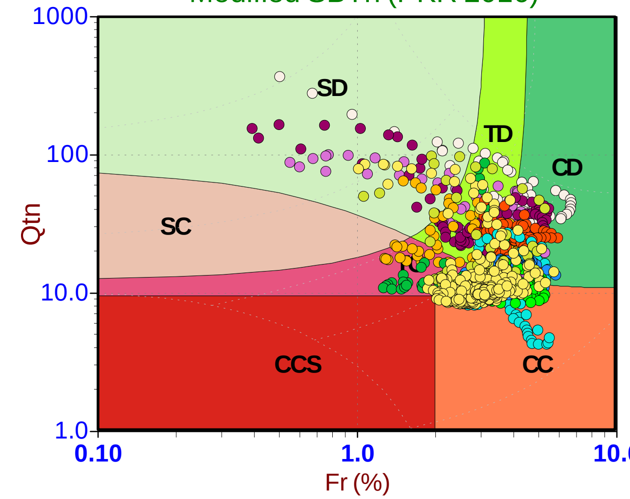
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

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

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

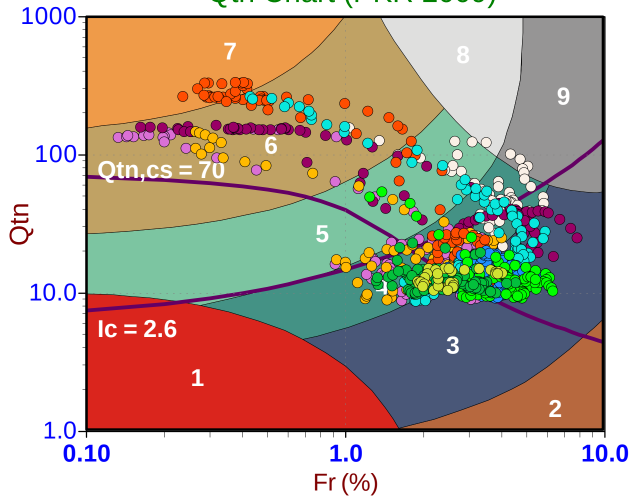
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Langan Engineering

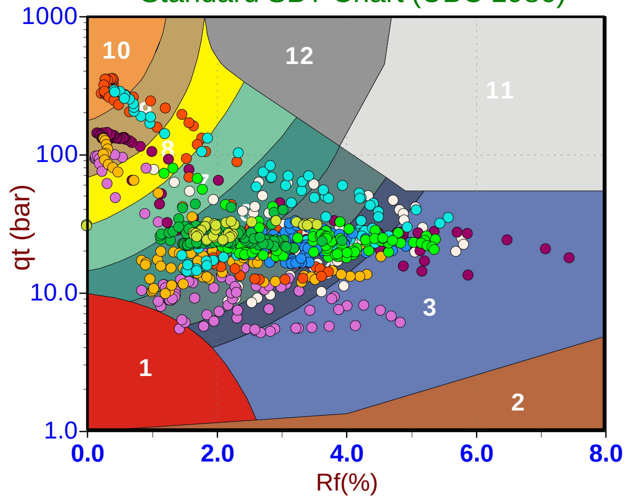
Job No: 22-56-24292
Date: 2022-06-13 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35

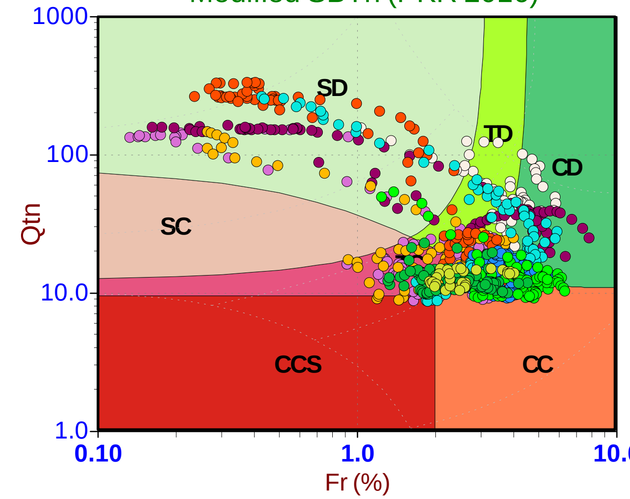
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

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

Legend

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

Legend

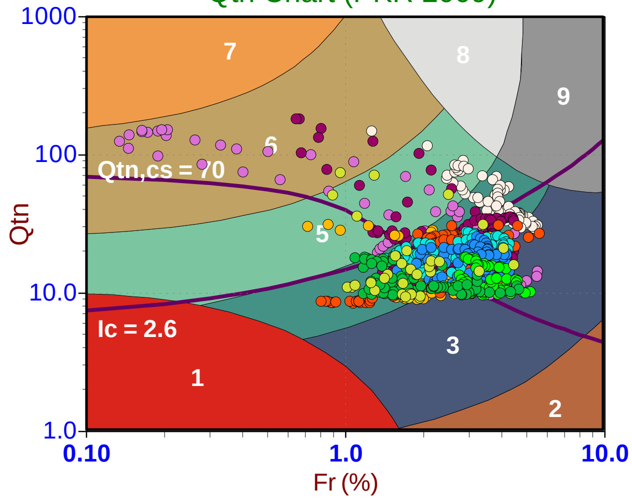
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

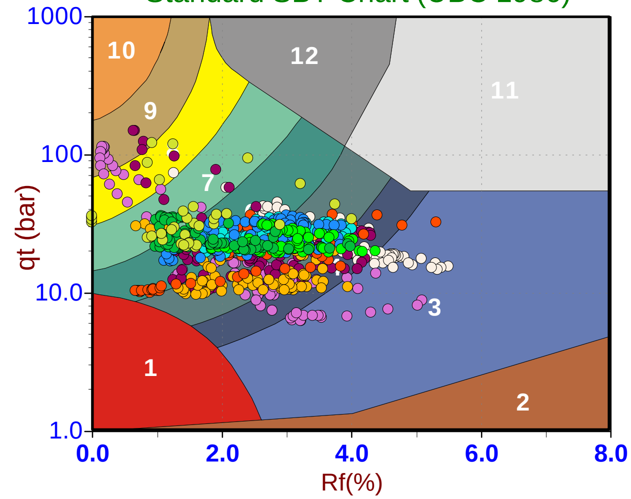
CONETEC

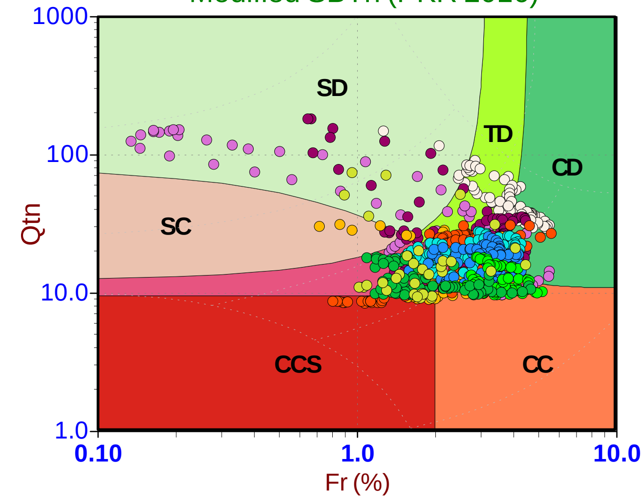
Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 11:02
Site: 4590 Patrick Henry

Sounding: CPT-03
Cone: 811:T1500F15U35







Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

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

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

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

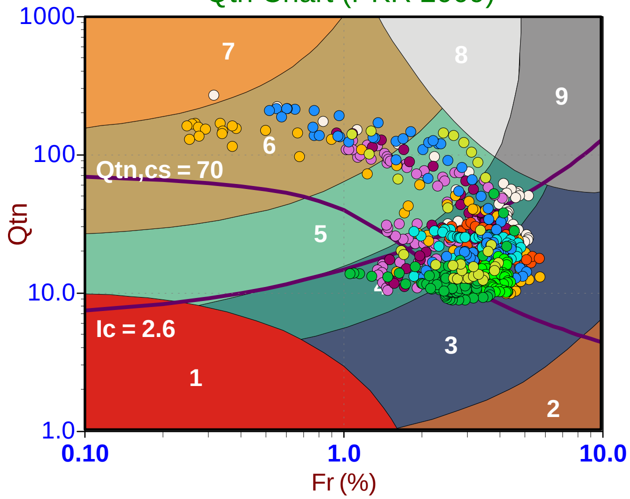
CONETEC

Langan Engineering

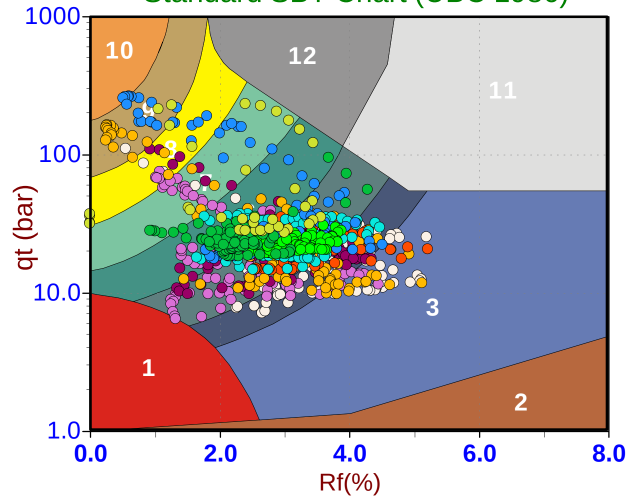
Job No: 22-56-24292
Date: 2022-06-13 11:58
Site: 4590 Patrick Henry

Sounding: CPT-04
Cone: 811:T1500F15U35

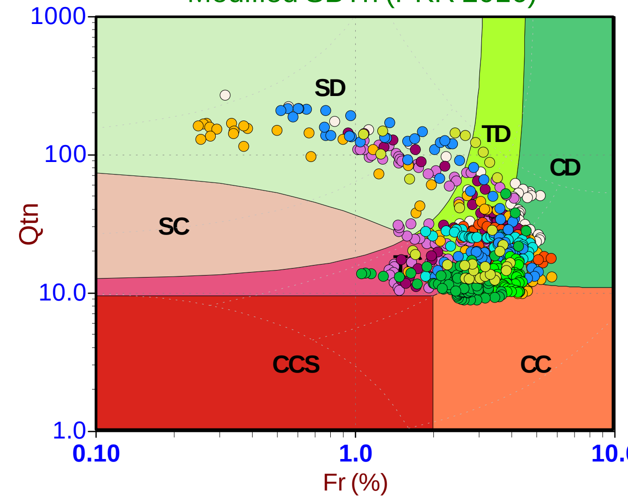
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

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

Legend

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

Legend

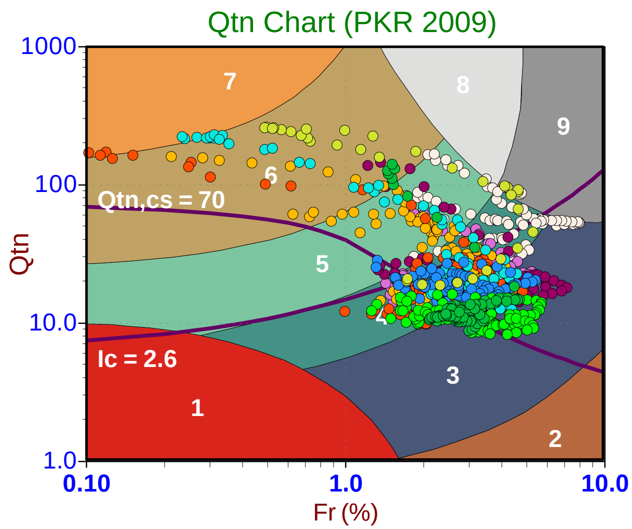
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

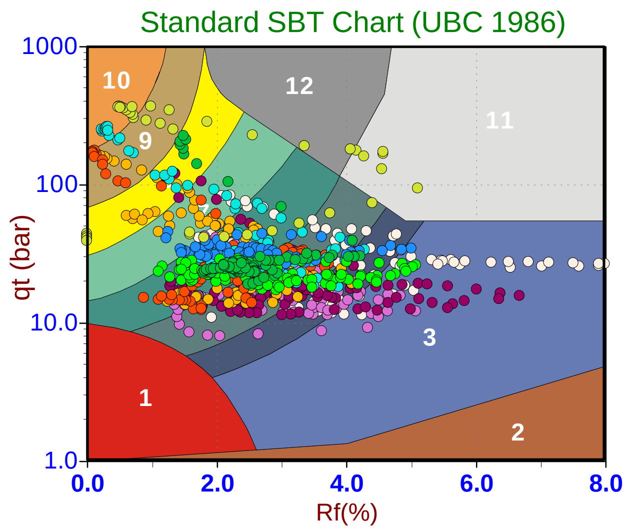
CONETEC

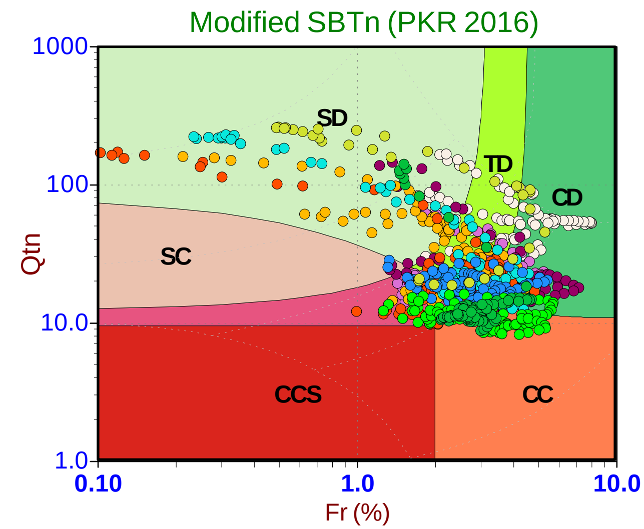
Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 12:56
Site: 4590 Patrick Henry

Sounding: CPT-05
Cone: 811:T1500F15U35







Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

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

Legend

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

Legend

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

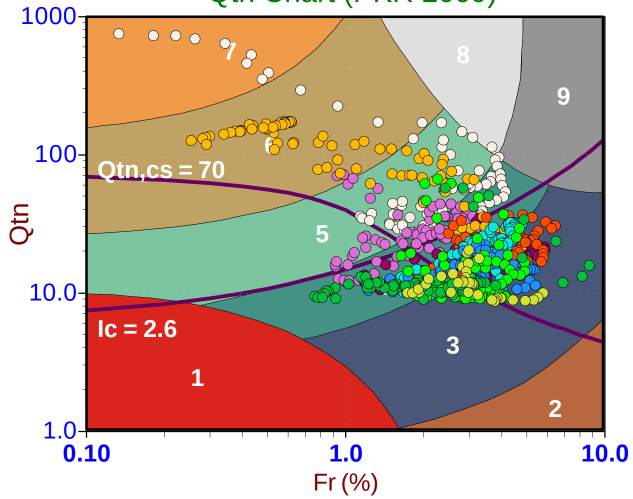
ConeTec

Langan Engineering

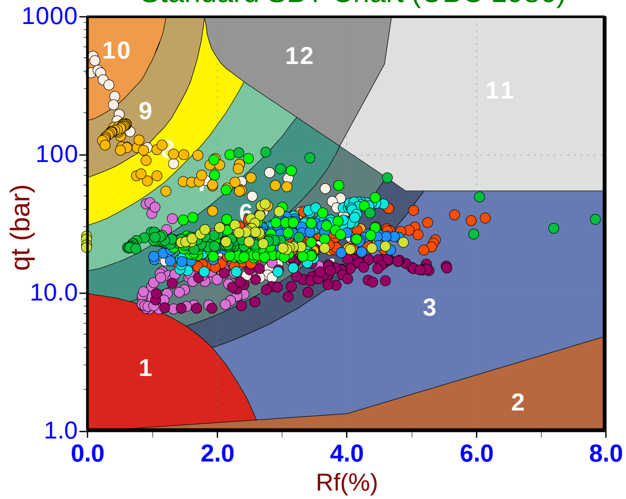
Job No: 22-56-24292
Date: 2022-06-13 13:55
Site: 4590 Patrick Henry

Sounding: CPT-06
Cone: 811:T1500F15U35

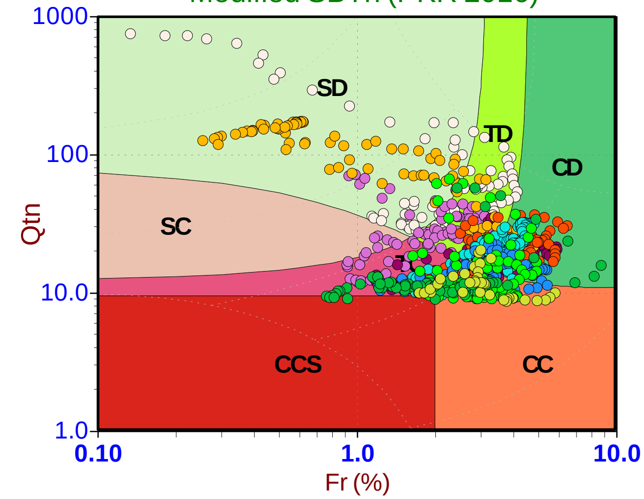
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
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- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

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- Sand Mixtures
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Legend

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- Gravelly Sand
- Stiff Fine Grained
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Legend

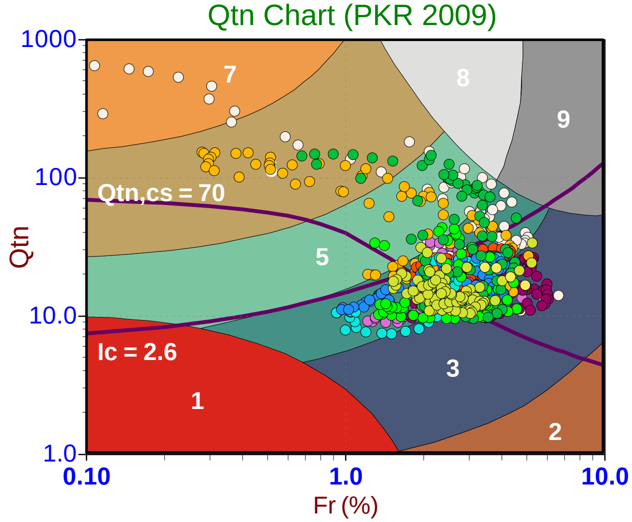
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

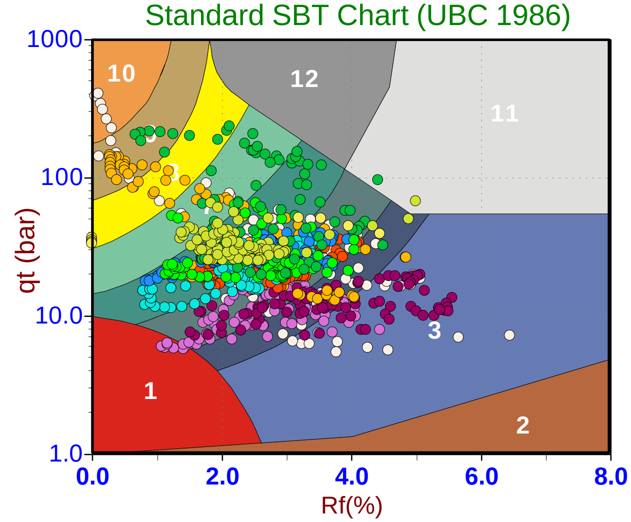
ConeTec

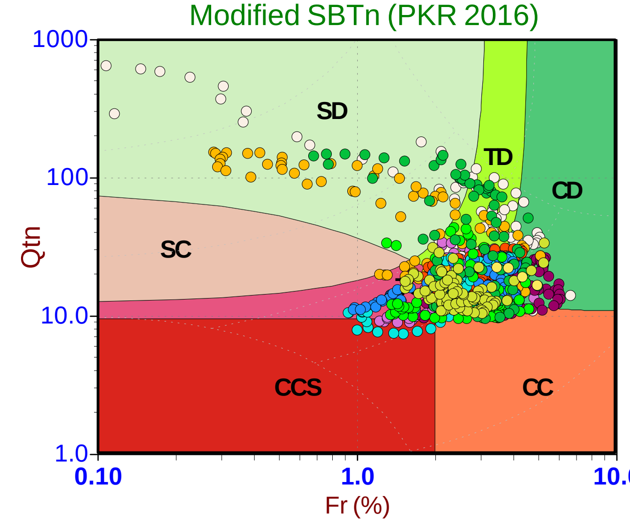
Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 15:14
Site: 4590 Patrick Henry

Sounding: CPT-07
Cone: 811:T1500F15U35







Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

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

Legend

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

Legend

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

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

	Job No:	22-56-24292
	Client:	Langan Engineering
	Project:	4590 Patrick Henry
	Start Date:	13-Jun-2022
	End Date:	13-Jun-2022

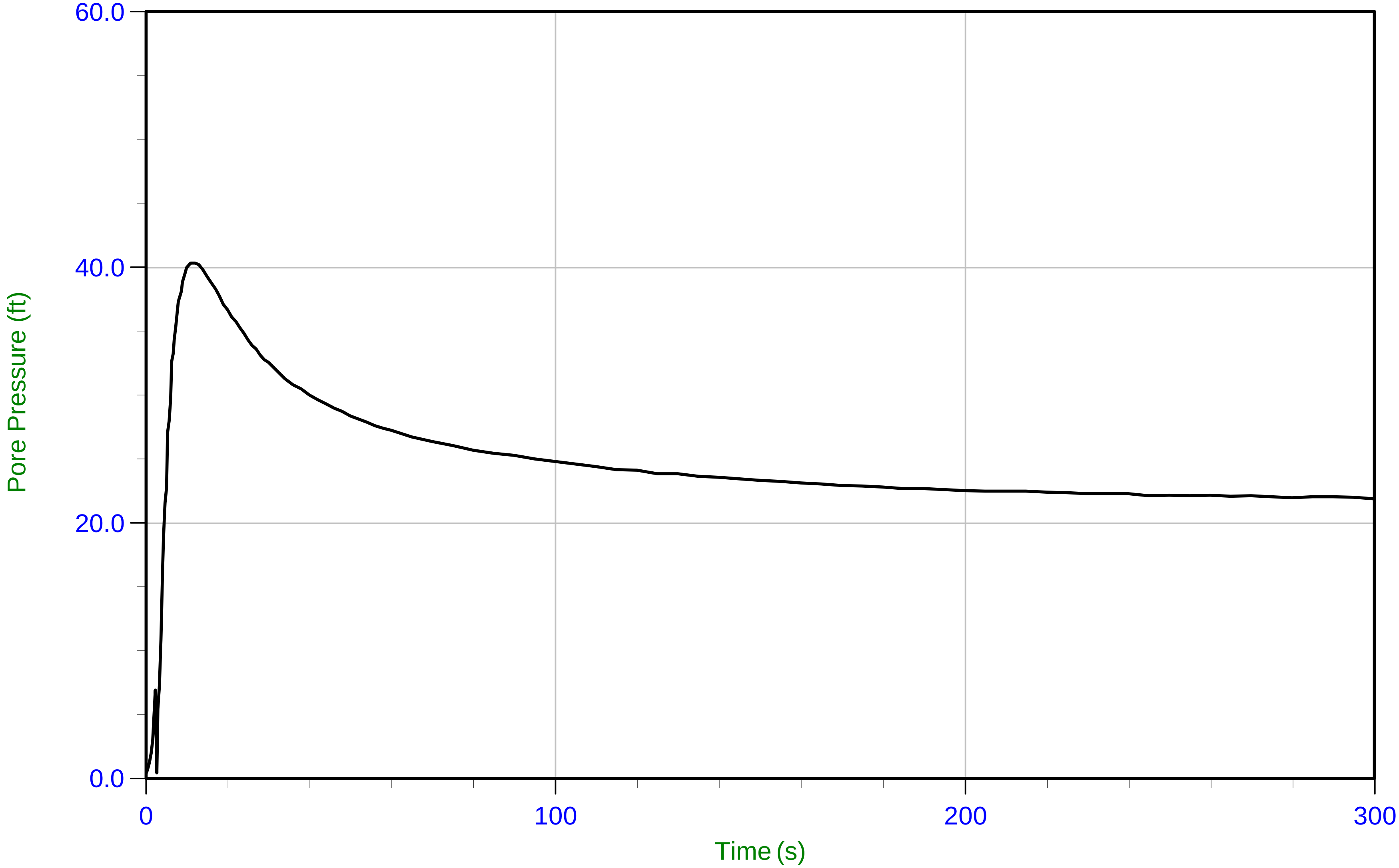
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)
CPT-01	22-56-24292_SP01	15	300	28.95	21.9	7.1
CPT-01	22-56-24292_SP01	15	295	76.44	Not Achieved	
CPT-02	22-56-24292_CP02	15	300	15.34	8.9	6.5
CPT-02	22-56-24292_CP02	15	300	21.57	15.0	6.6
CPT-03	22-56-24292_CP03	15	300	16.90	9.0	7.9
CPT-04	22-56-24292_CP04	15	300	16.08	9.1	7.0
CPT-05	22-56-24292_CP05	15	300	17.96	8.5	9.5
CPT-06	22-56-24292_CP06	15	300	22.88	15.3	7.6
CPT-07	22-56-24292_CP07	15	300	25.43	17.4	8.0

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

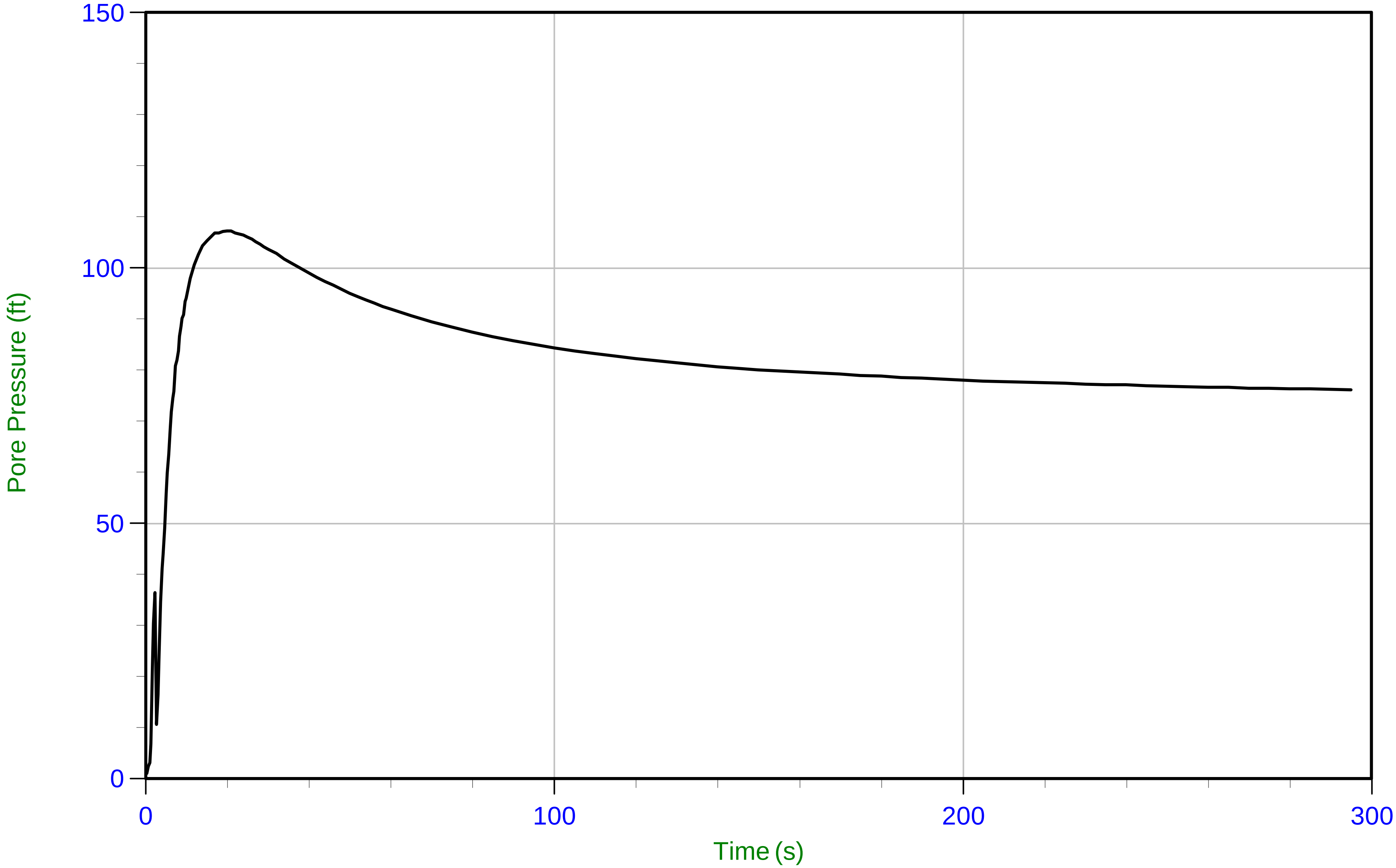
Filename: 22-56-24292_SP01.PPD2
Depth: 8.825 m / 28.953 ft
Duration: 300.0 s

u Min: 0.4 ft
u Max: 40.3 ft
u Final: 21.9 ft

WT: 2.153 m / 7.065 ft
Ueq: 21.9 ft

CONETEC

Langan Engineering	Job No: 22-56-24292 Date: 06/13/2022 07:49 Site: 4590 Patrick Henry	Sounding: CPT-01 Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_SP01.PPD2
Depth: 23.300 m / 76.443 ft
Duration: 295.0 s

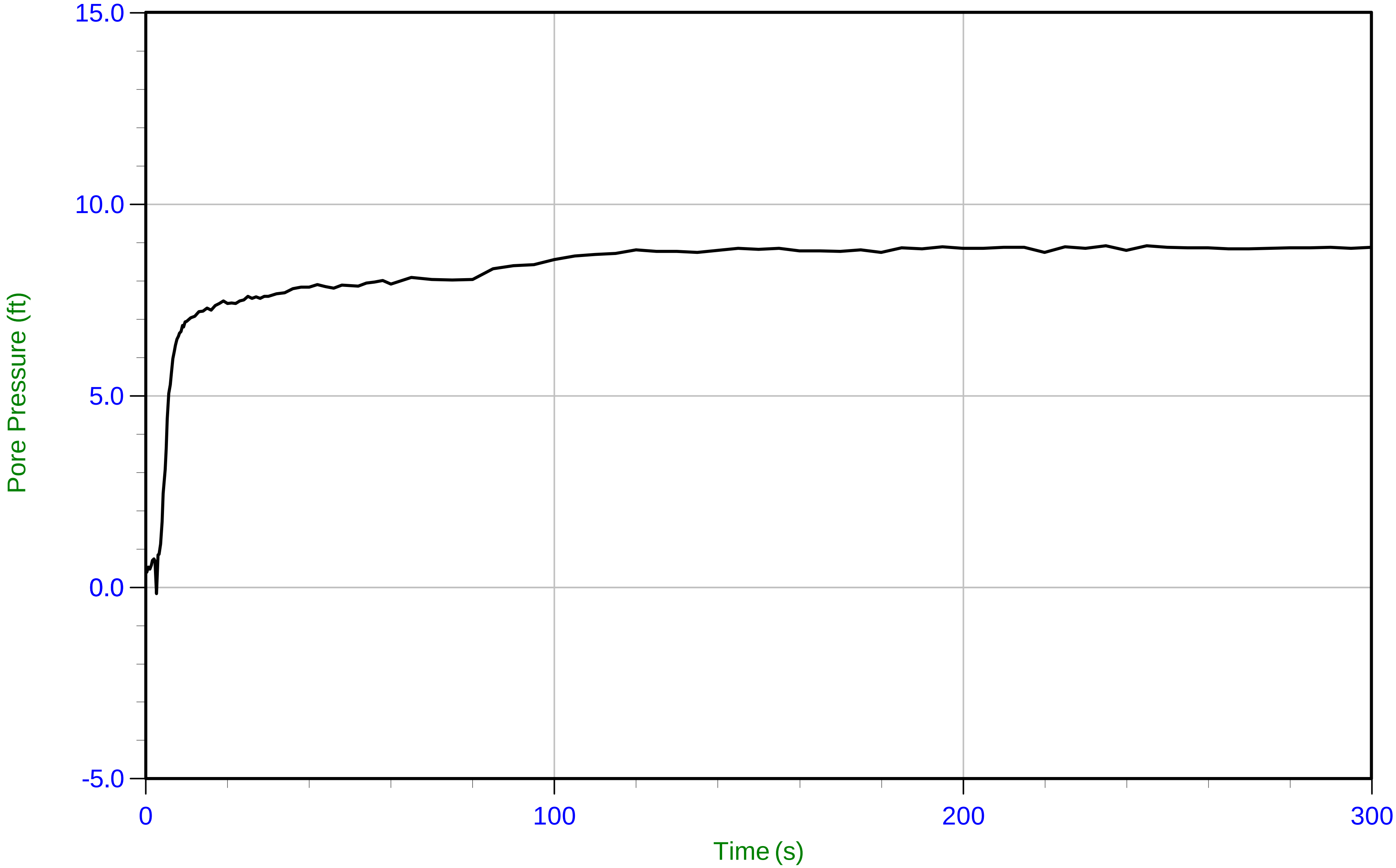
u Min: 1.0 ft
u Max: 107.3 ft
u Final: 76.1 ft

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP02.PPD2
Depth: 4.675 m / 15.338 ft
Duration: 300.0 s

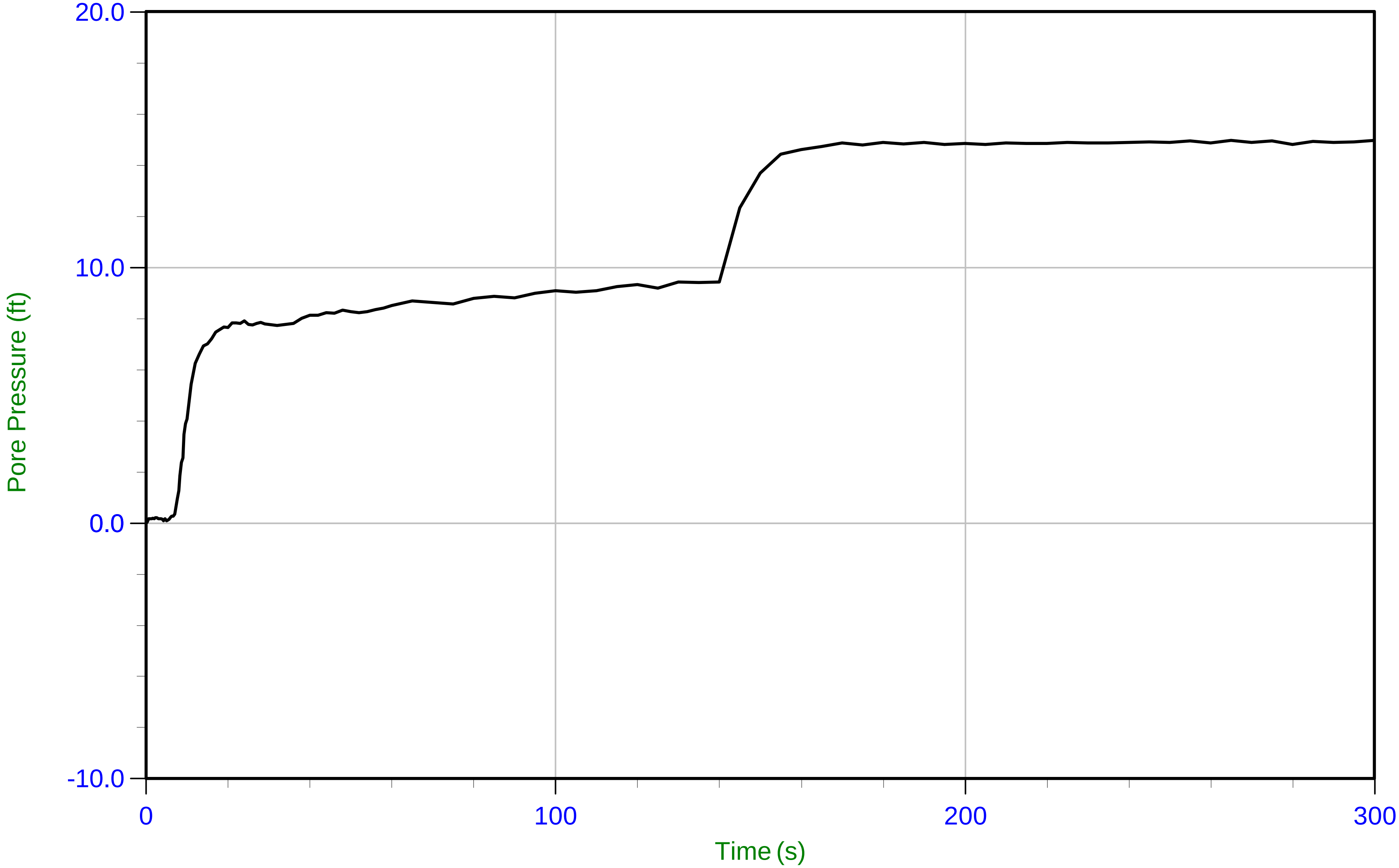
u Min: -0.2 ft
u Max: 8.9 ft
u Final: 8.9 ft

WT: 1.975 m / 6.480 ft
Ueq: 8.9 ft

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 09:58
Site: 4590 Patrick Henry

Sounding: CPT-02
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP02.PPD2
Depth: 6.575 m / 21.571 ft
Duration: 300.0 s

u Min: 0.0 ft
u Max: 15.0 ft
u Final: 15.0 ft

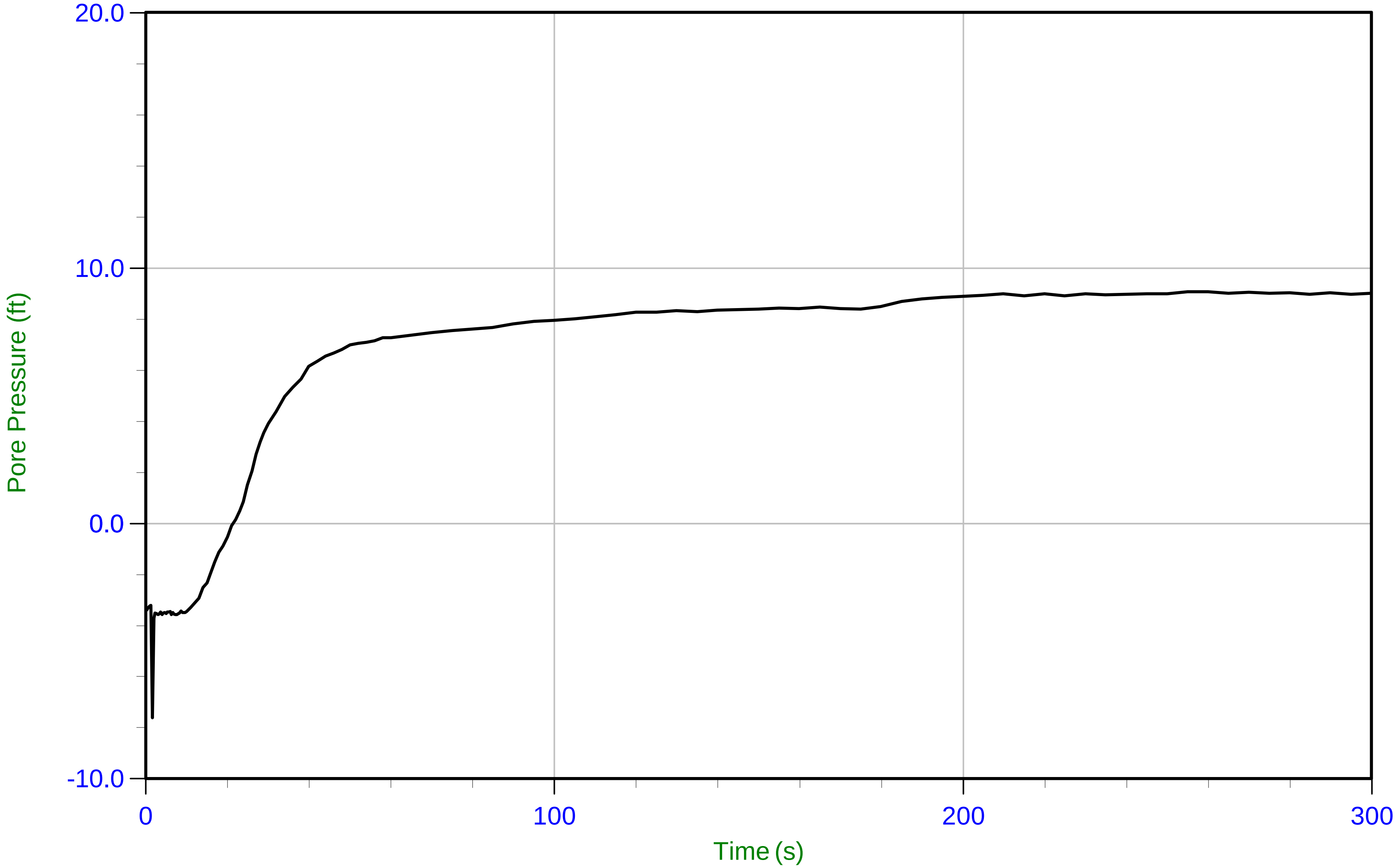
WT: 2.014 m / 6.608 ft
Ueq: 15.0 ft

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 11:02
Site: 4590 Patrick Henry

Sounding: CPT-03
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP03.PPD2
Depth: 5.150 m / 16.896 ft
Duration: 300.0 s

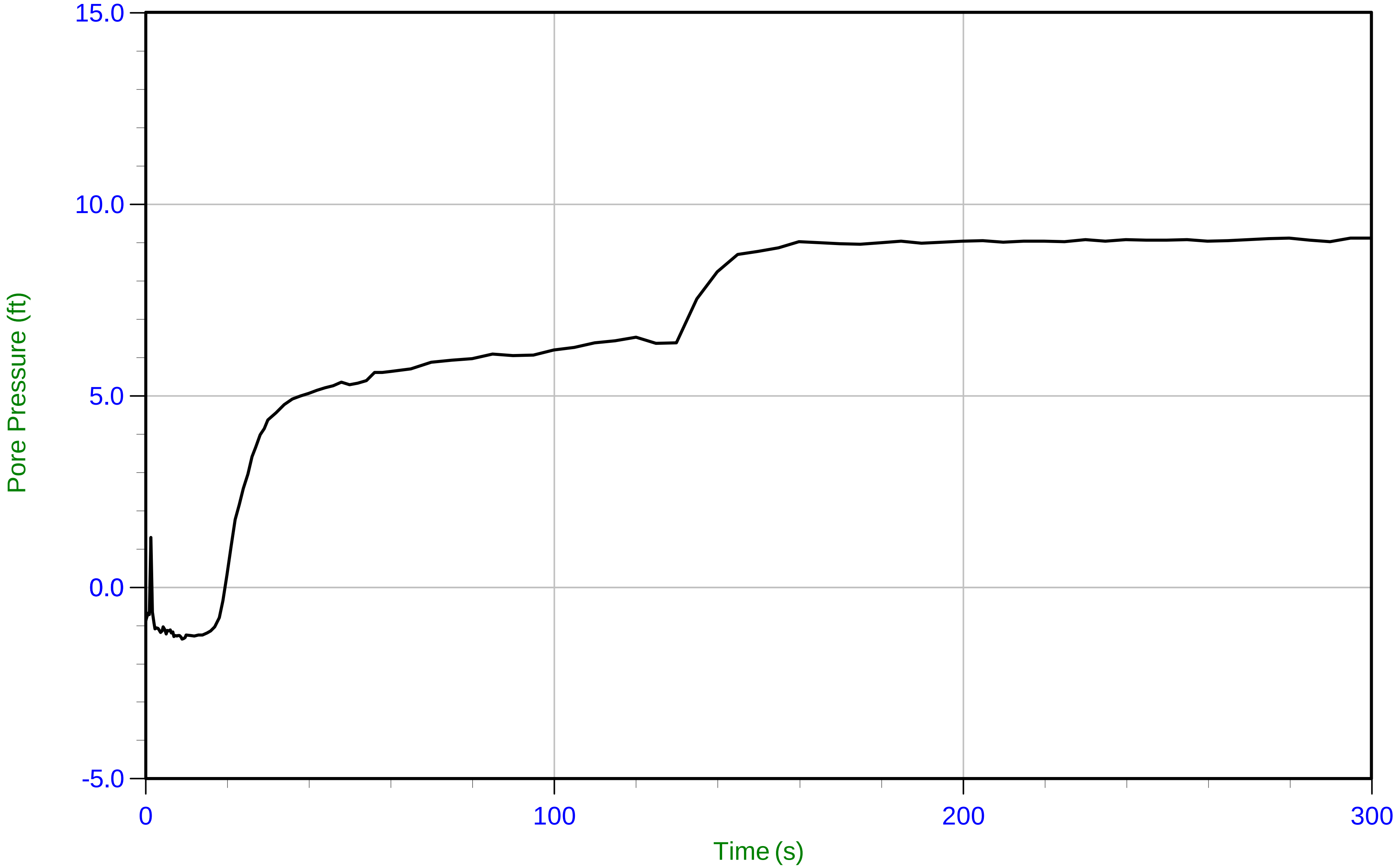
u Min: -7.6 ft
u Max: 9.1 ft
u Final: 9.0 ft

WT: 2.407 m / 7.898 ft
Ueq: 9.0 ft

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 11:58
Site: 4590 Patrick Henry

Sounding: CPT-04
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP04.PPD2
Depth: 4.900 m / 16.076 ft
Duration: 300.0 s

u Min: -1.4 ft
u Max: 9.1 ft
u Final: 9.1 ft

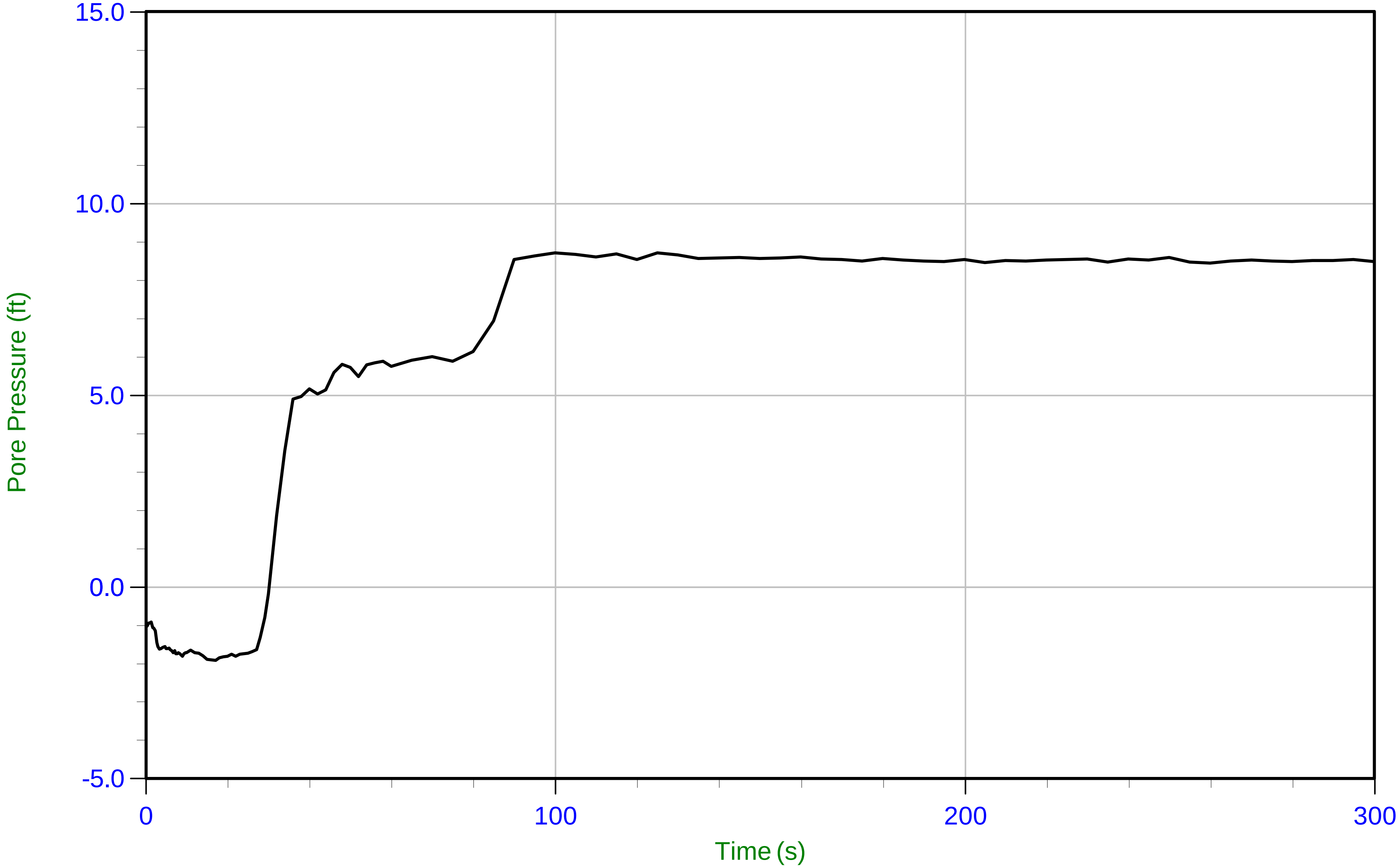
WT: 2.120 m / 6.955 ft
Ueq: 9.1 ft

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 12:56
Site: 4590 Patrick Henry

Sounding: CPT-05
Cone: 811:T1500F15U35 Area=15 cm²



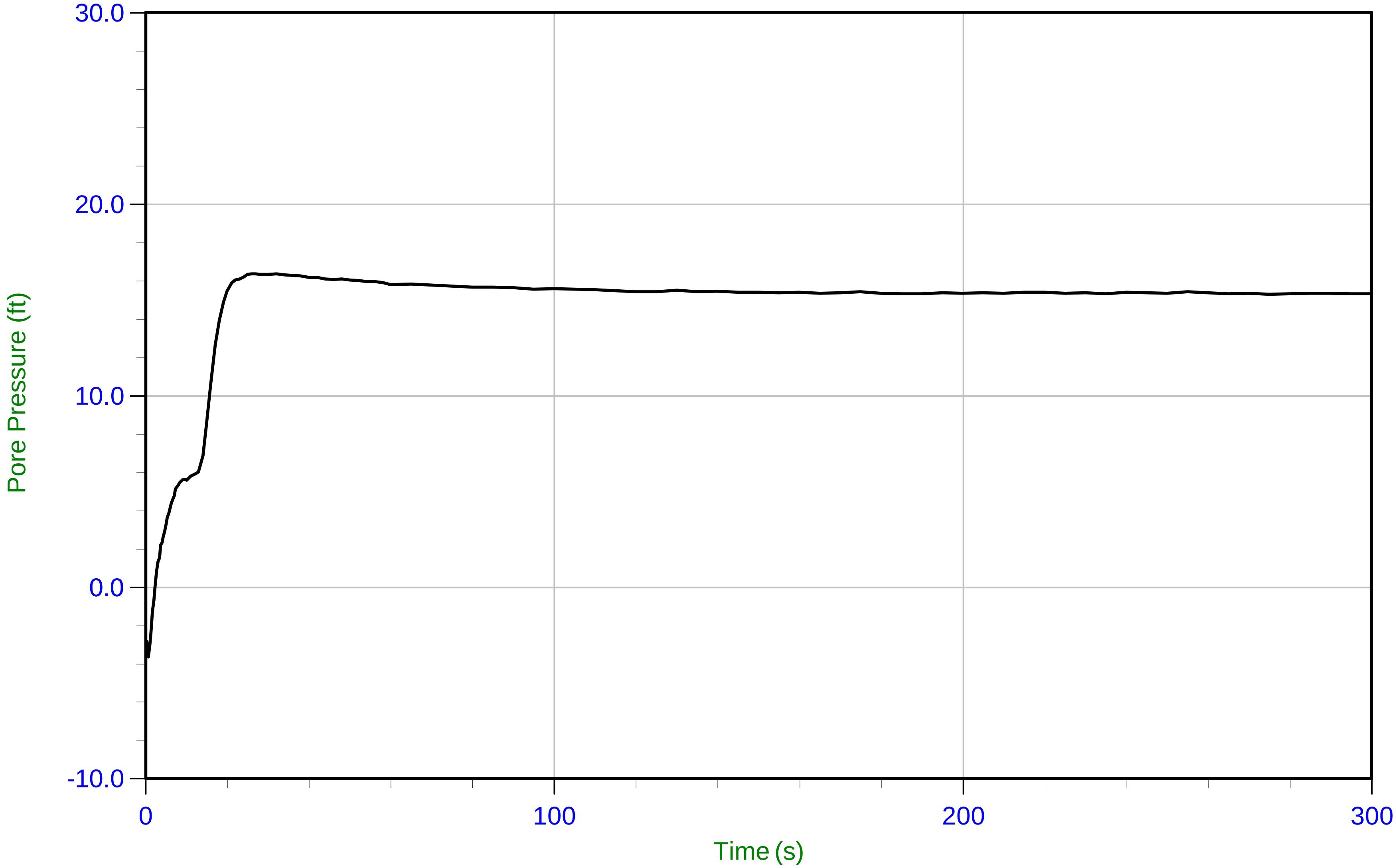
Trace Summary:

Filename: 22-56-24292_CP05.PPD2
Depth: 5.475 m / 17.962 ft
Duration: 300.0 s

u Min: -1.9 ft
u Max: 8.7 ft
u Final: 8.5 ft

WT: 2.888 m / 9.473 ft
Ueq: 8.5 ft

Langan Engineering	Job No: 22-56-24292 Date: 06/13/2022 13:55 Site: 4590 Patrick Henry	Sounding: CPT-06 Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP06.PPD2
Depth: 6.975 m / 22.884 ft
Duration: 300.0 s

u Min: -3.6 ft
u Max: 16.4 ft
u Final: 15.3 ft

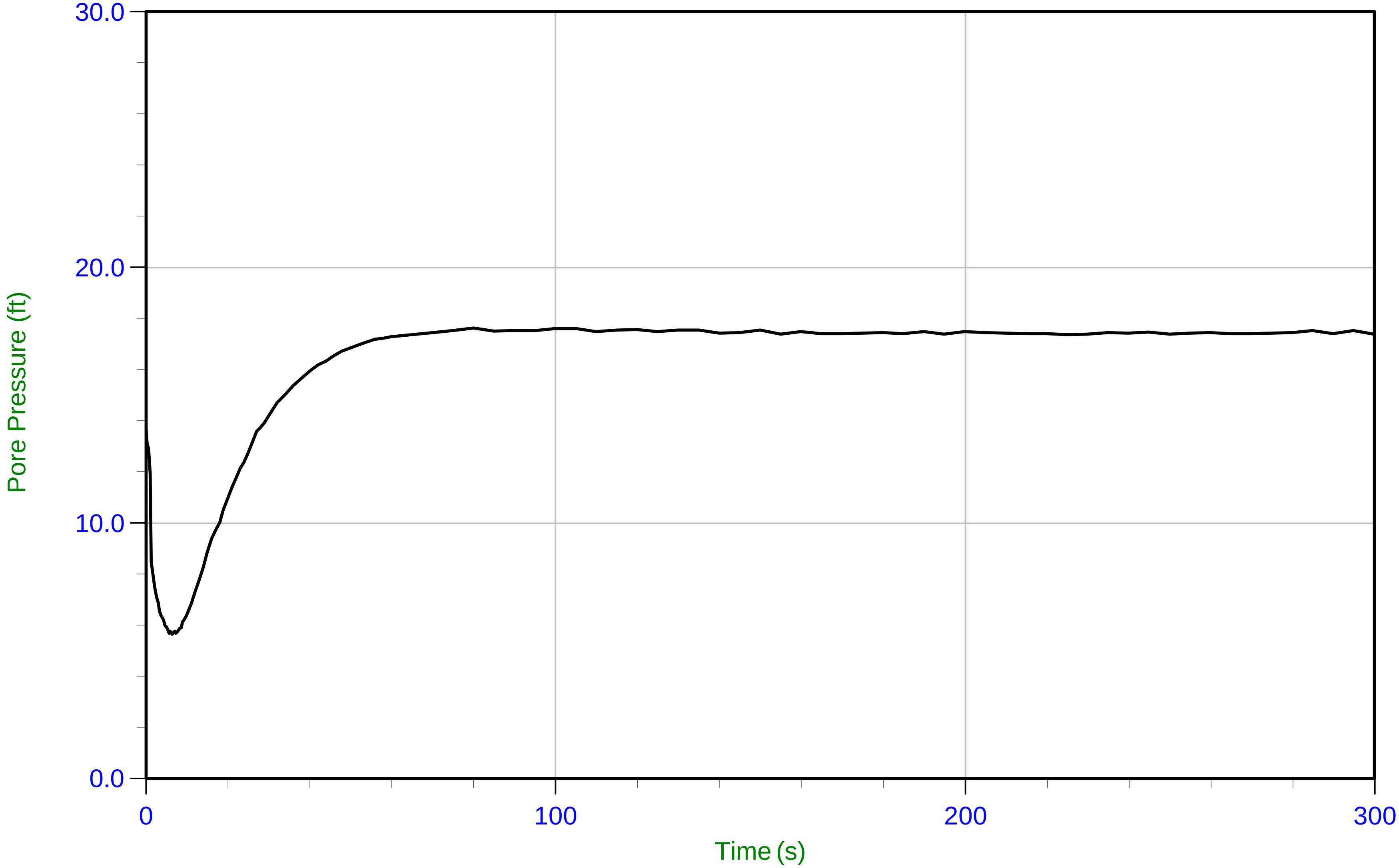
WT: 2.307 m / 7.569 ft
Ueq: 15.3 ft

ConeTec

Langan Engineering

Job No: 22-56-24292
Date: 06/13/2022 15:14
Site: 4590 Patrick Henry

Sounding: CPT-07
Cone: 811:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-24292_CP07.PPD2
Depth: 7.750 m / 25.426 ft
Duration: 300.0 s

u Min: 5.7 ft
u Max: 17.6 ft
u Final: 17.4 ft

WT: 2.448 m / 8.033 ft
Ueq: 17.4 ft

Seismic Cone Penetration Test Tabular Results

Job No: 22-56-24292
Client: Langan Engineering
Project: 4590 Patrick Henry
Sounding ID: CPT-01
Date: 06:13:22 07:49

Seismic Source: Beam
Seismic Offset (ft): 2.10
Source Depth (ft): 0.00
Geophone Offset (ft): 0.81

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.79	1.98	2.88			
6.07	5.26	5.66	2.78	6.89	403
9.25	8.44	8.70	3.04	5.35	567
12.53	11.72	11.91	3.21	5.95	540
15.81	15.00	15.15	3.24	4.26	760
19.19	18.38	18.50	3.35	3.85	872
22.38	21.56	21.67	3.16	3.58	883
25.66	24.84	24.93	3.27	4.60	711
28.97	28.16	28.24	3.30	4.63	713
32.32	31.50	31.57	3.34	3.96	844
35.50	34.69	34.75	3.18	3.67	865
38.88	38.07	38.12	3.37	3.65	926
42.06	41.25	41.30	3.18	4.03	788
45.44	44.63	44.68	3.38	3.84	879
48.72	47.91	47.95	3.28	4.02	816
52.00	51.19	51.23	3.28	3.88	845
55.28	54.47	54.51	3.28	4.06	807
58.56	57.75	57.79	3.28	3.70	886
61.84	61.03	61.07	3.28	3.59	914
65.13	64.31	64.35	3.28	3.72	881
68.24	67.43	67.46	3.12	3.26	957
74.90	74.09	74.12	6.66	6.83	975
78.25	77.44	77.46	3.35	3.28	1019
81.53	80.72	80.74	3.28	3.40	965
84.91	84.10	84.12	3.38	3.72	909
88.19	87.38	87.40	3.28	3.48	943
91.37	90.56	90.58	3.18	2.94	1083
94.59	93.77	93.80	3.22	3.02	1066
97.93	97.12	97.14	3.35	3.02	1106
100.56	99.75	99.77	2.62	2.59	1014

Seismic Cone Penetration Test Plots

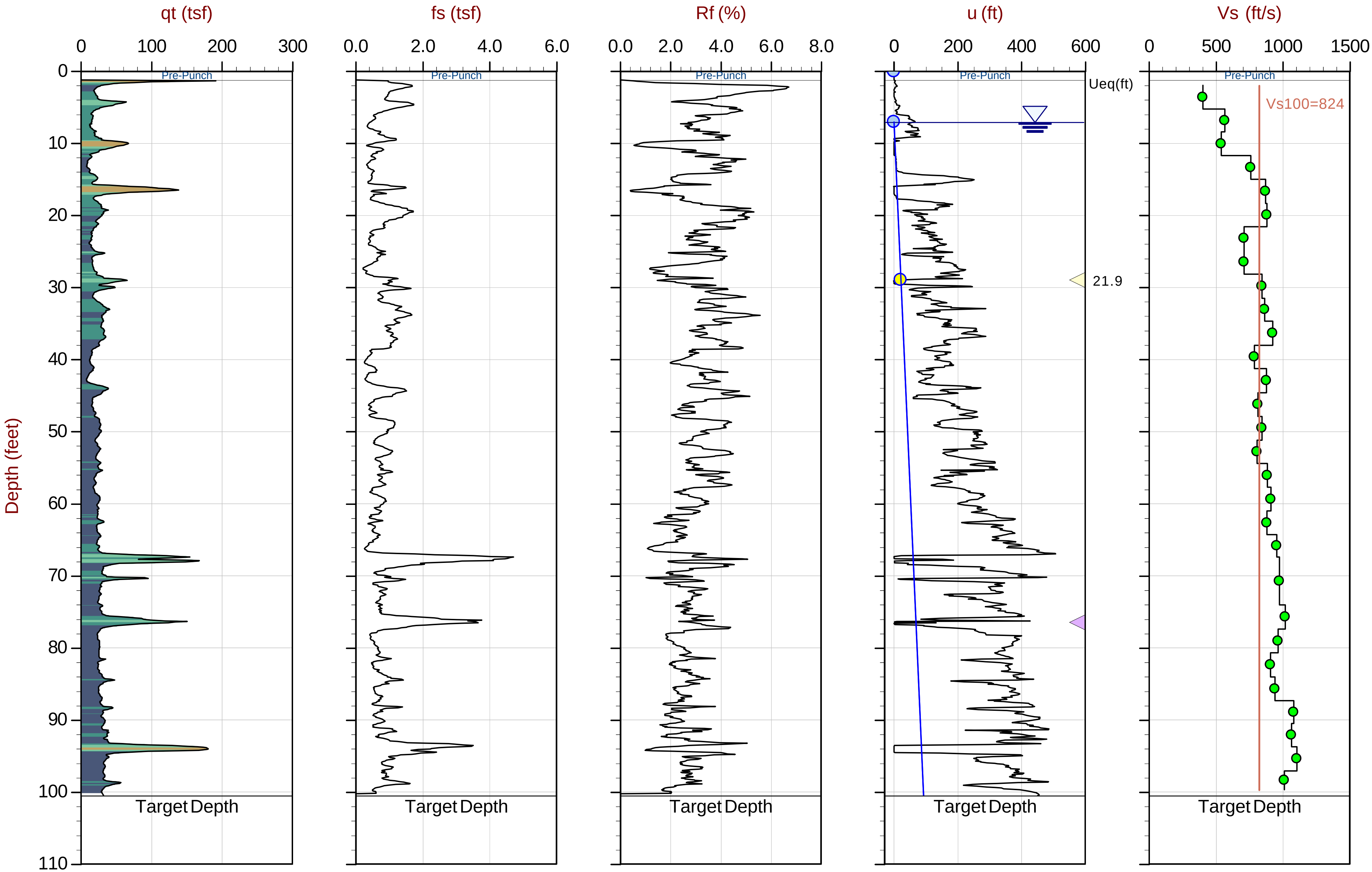
CONETEC

CONETEC

Langan Engineering

Job No: 22-56-24292
Date: 2022-06-13 07:49
Site: 4590 Patrick Henry

Sounding: CPT-01
Cone: 811:T1500F15U35



Max Depth: 30.650 m / 100.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-24292_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139427m E: 589766

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

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

Seismic Cone Penetration Test Shear Wave (Vs) Traces

CONETEC

Job No: 22-56-24292
Date: 06:13:22 07:49

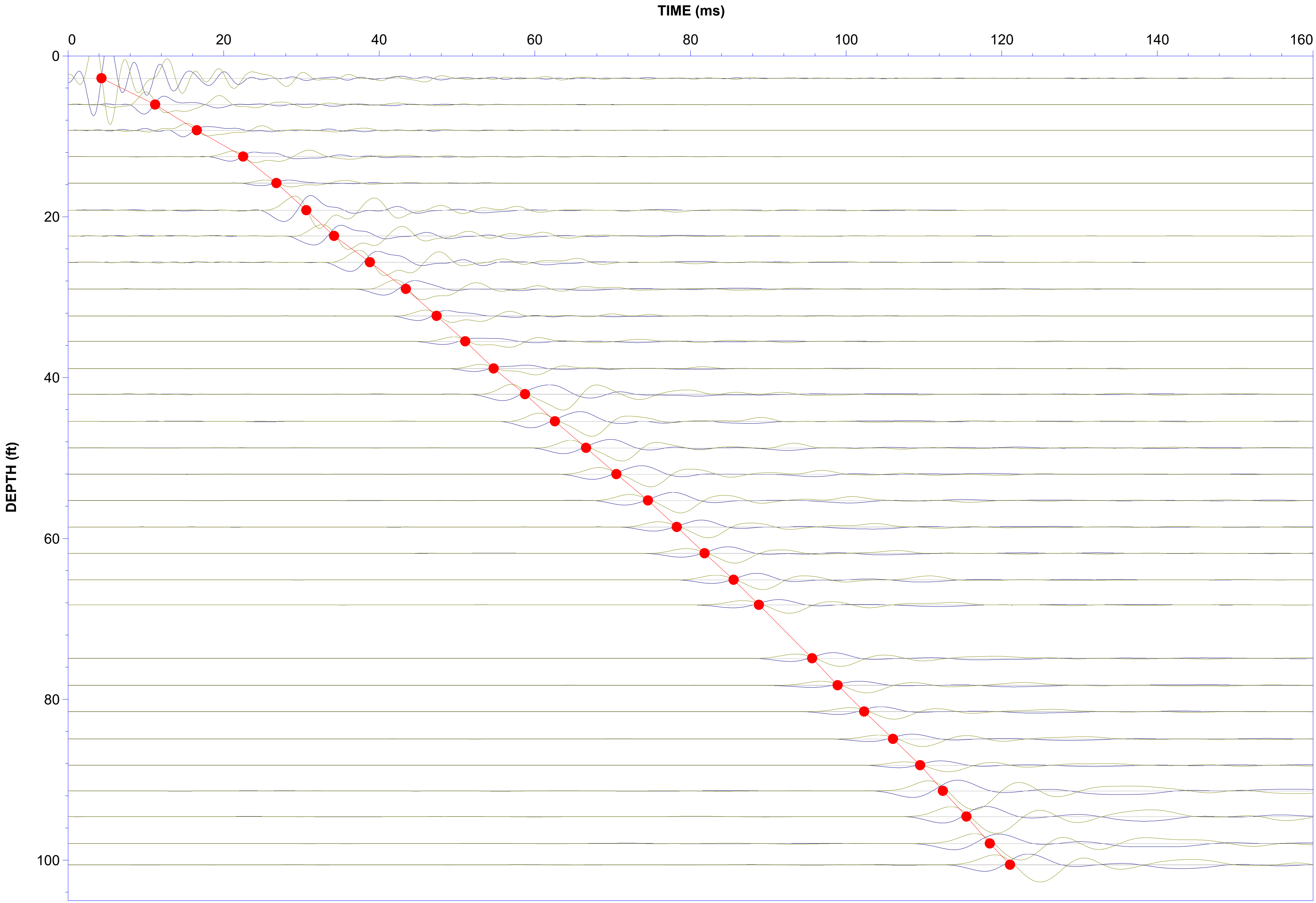
Client: Langan Engineering
Cone: 811:T1500F15U35

Project Title: 4590 Patrick Henry

Filter: BP 0-500 Hz

Hole: CPT-01

Filter: Shear Wave



Methodology Statements and Data File Formats

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 presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 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.

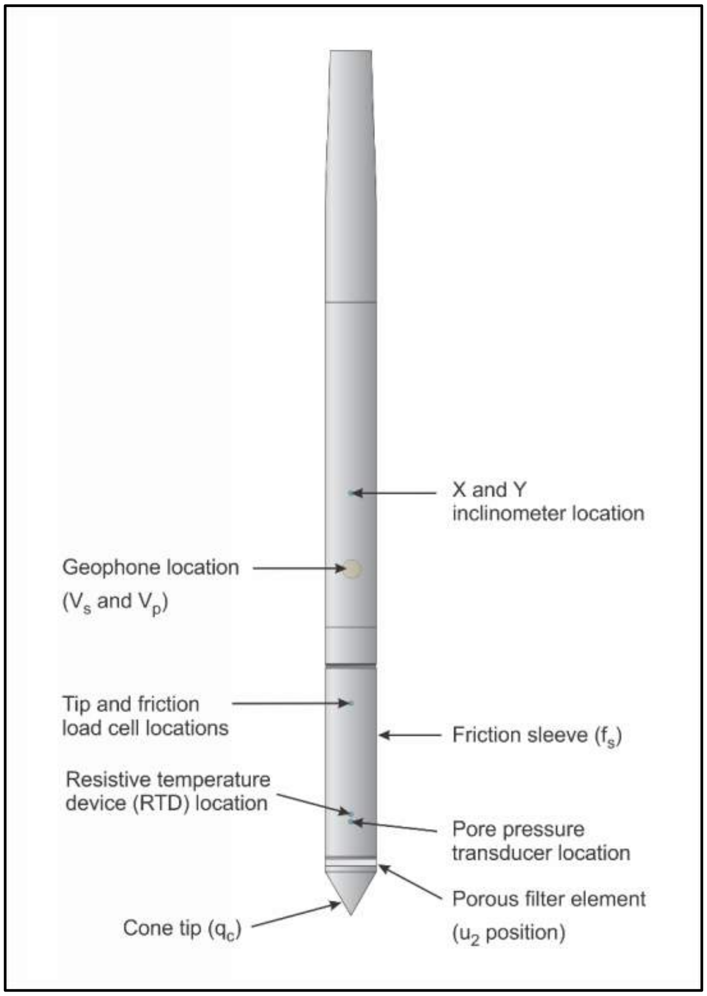


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a 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 wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

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

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

All testing is performed in accordance to ConeTec's 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 (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

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

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

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

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

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

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

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

ConeTec's piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

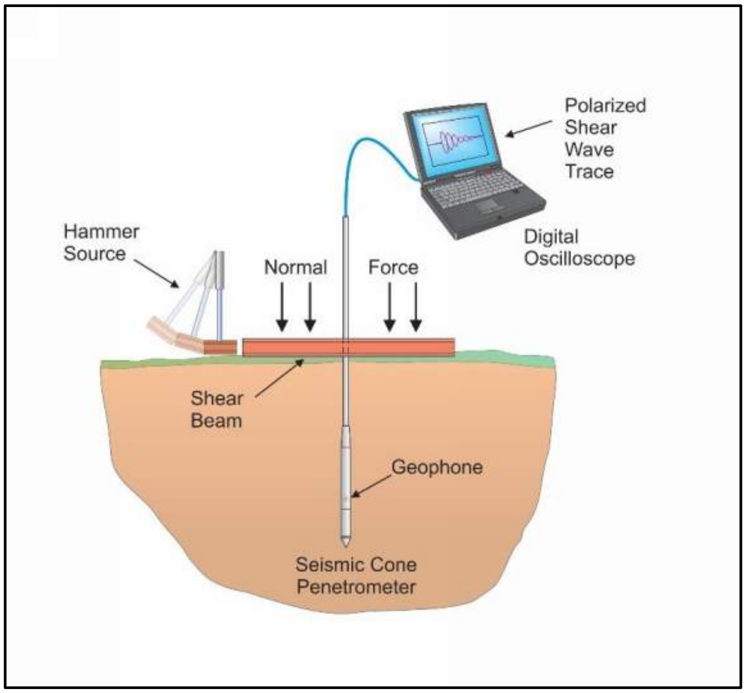


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

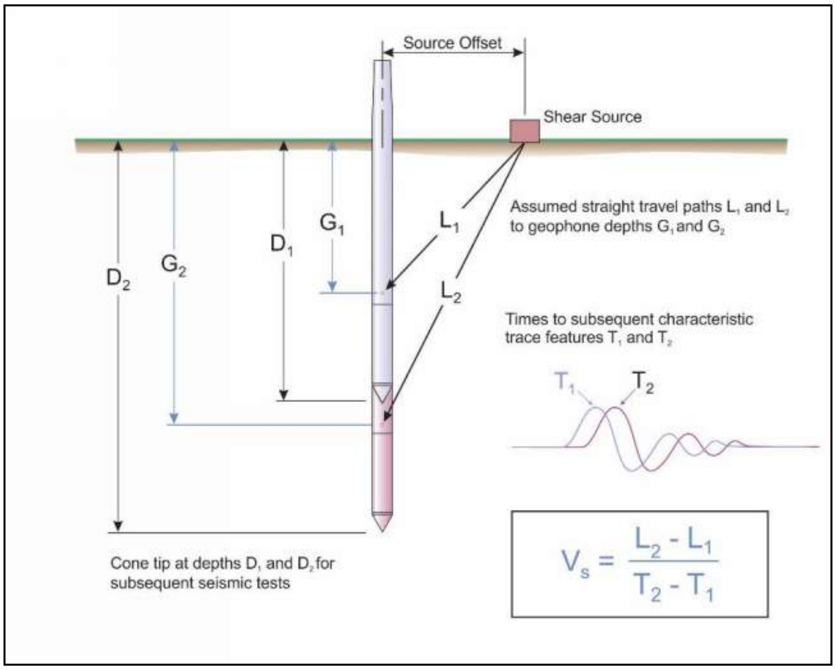


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

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

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

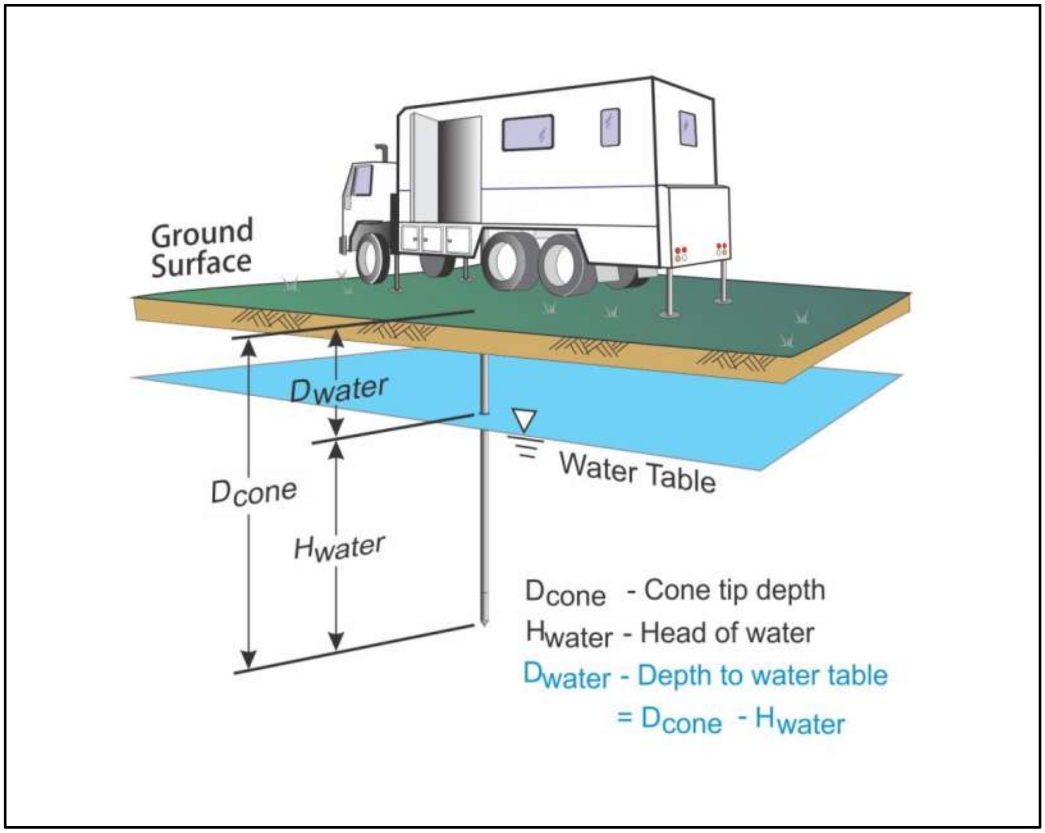


Figure PPD-1. Pore pressure dissipation test setup

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

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

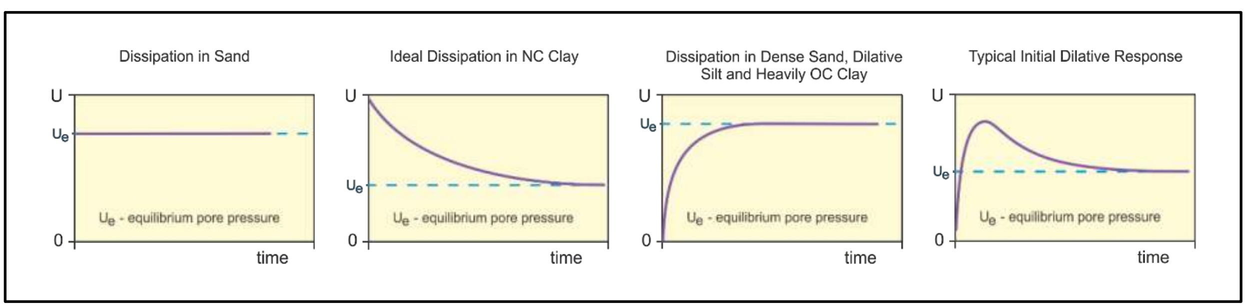


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

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

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

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

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

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

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

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

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

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

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Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

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

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

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.

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Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

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

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381. DOI: 10.1139/T98-105.

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

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

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

CONETEC

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 14

Revised November 26, 2019

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

CONETEC

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 November 26, 2019

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.

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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 of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1-a) \bullet u_2$ (consistent units are implied)

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 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.

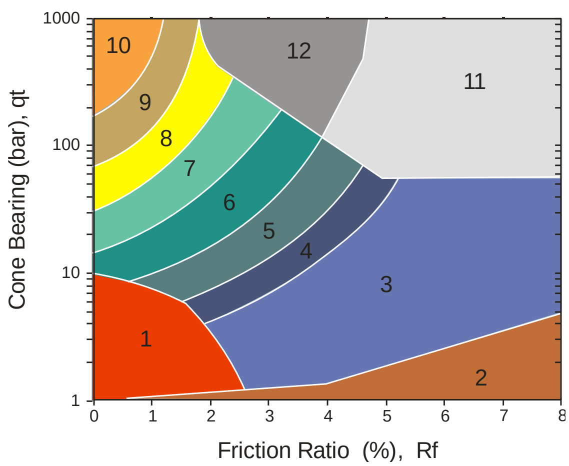
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were 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 or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. 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) SBT 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. Please 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 used 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 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.

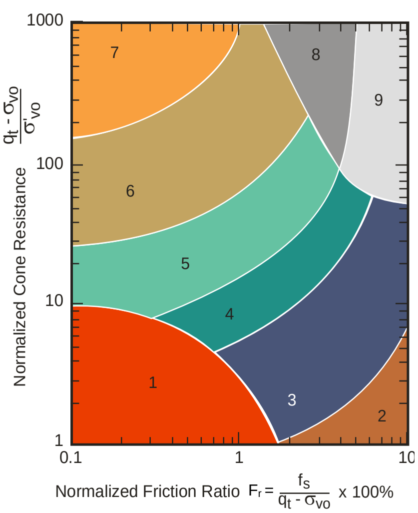


Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

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

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



Zone	Normalized Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

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

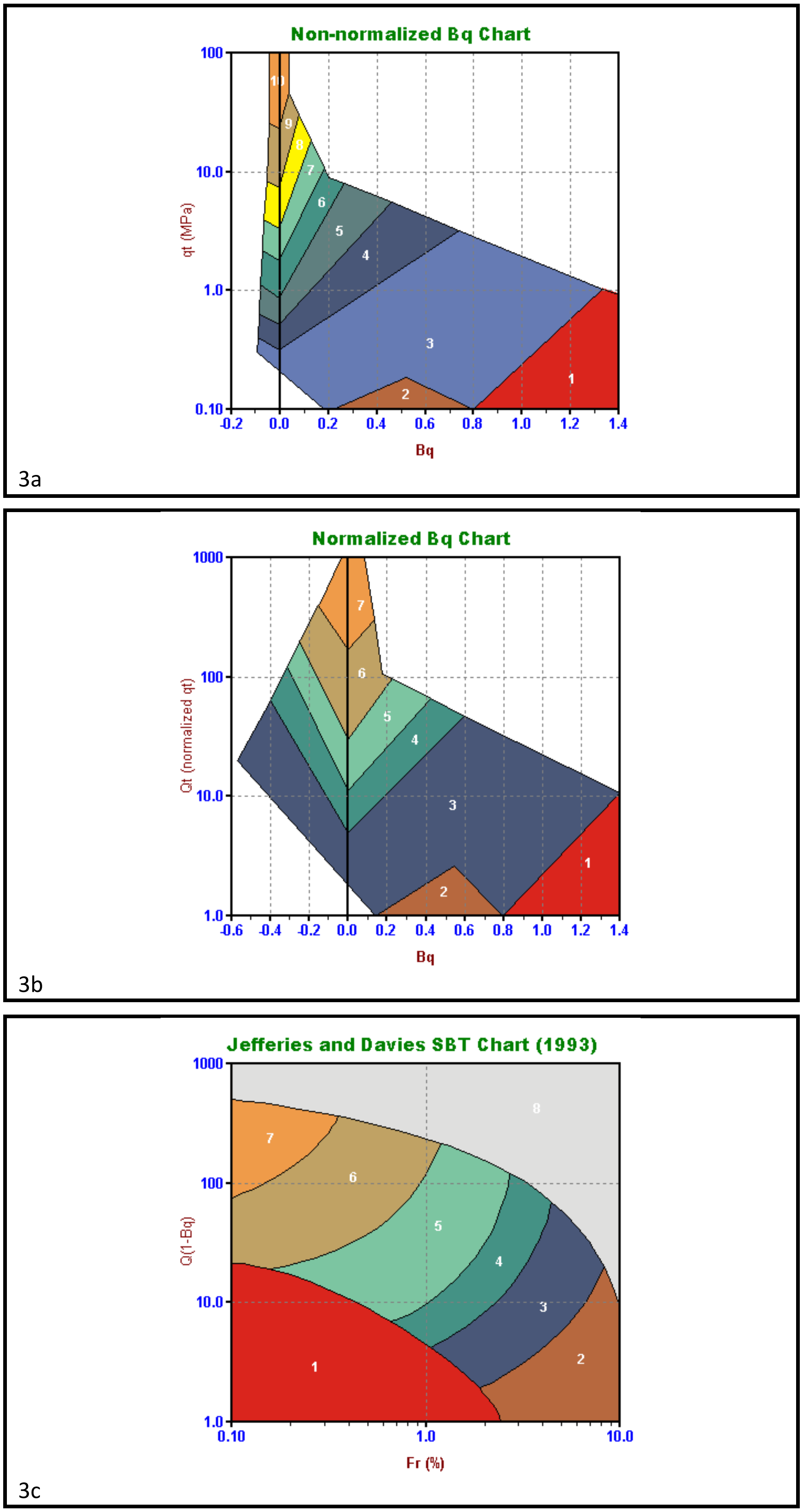


Figure 3. Alternate Soil Behavior Type Charts

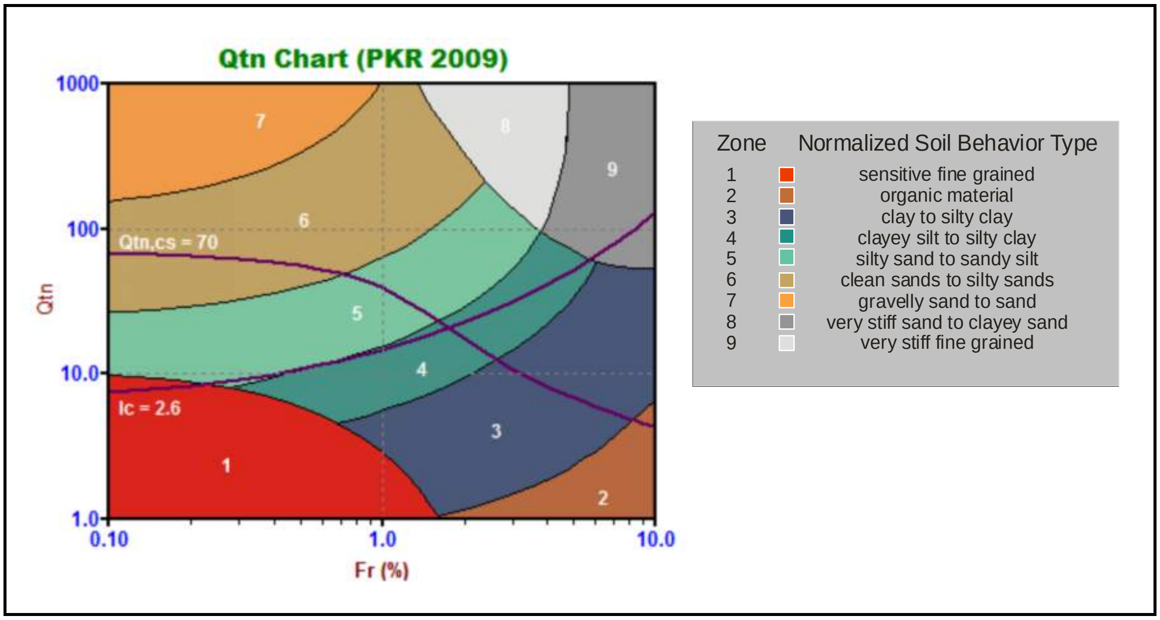


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

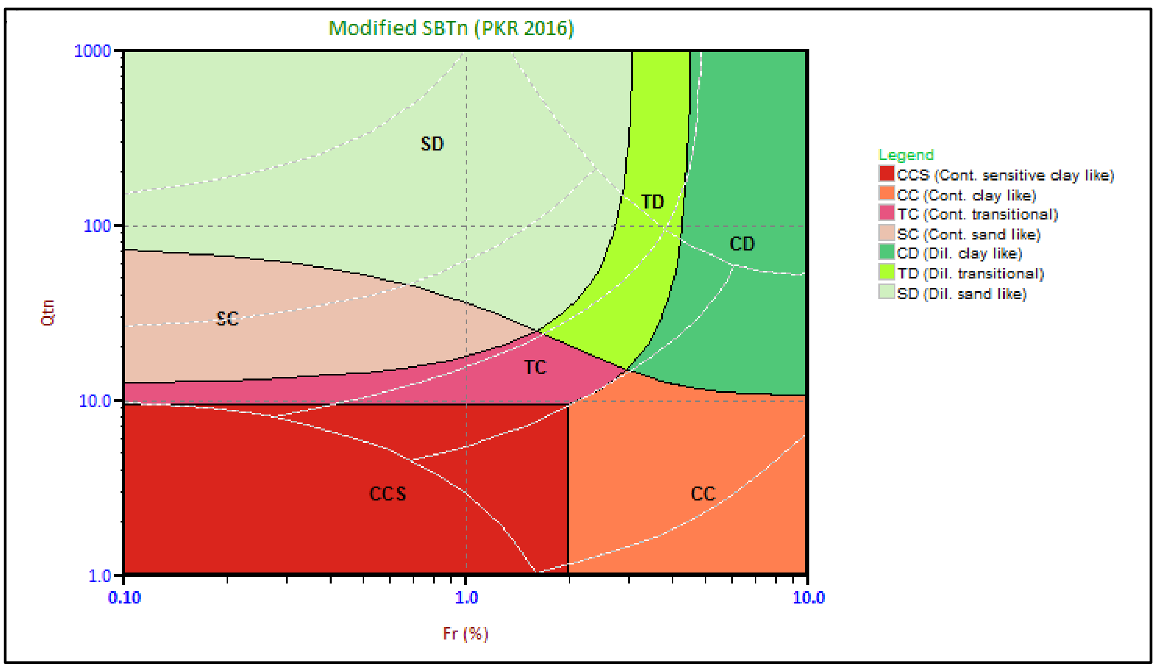


Figure 5. Modified SBTn Behavior Based Chart

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 the user should 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 as an undrained material (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 Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) 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

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 based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
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: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	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*

Calculated Parameter	Description	Equation	Ref
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 since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *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)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	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) Mayne fs (sleeve friction) method 7) Robertson 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

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 pressure determined from one of the following user selectable options: 1) hydrostatic below 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 hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is 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 1.7)* *P_a is atmospheric pressure (100 kPa)*	12
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)*	3, 12

Calculated Parameter	Description	Equation	Ref
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
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	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
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	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
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	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*	CK*
Bq	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 qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	CK*
Q_t or Norm: Qt	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}.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5
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-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	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: Pa = atmospheric pressure	21
qc1 (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: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	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
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 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, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *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 varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (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

Calculated Parameter	Description	Equation	Ref
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 Ic (PKR 2009).	*n (PKR 2009) = 0.381 (I_c) + 0.05 (σ_v'/P_a) – 0.15*	15
Qtn (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 Qtn (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
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) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	 19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	 a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) OCR = $0.25\cdot(Qt)^{1.25}$	 9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es 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. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	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, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su\ (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	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

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	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
Kc (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
$(N_1)_{60cs}Ic$	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
Qtn,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	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\, [(N_1)_{60}]^{4.79}$ *qc1* is calculated from specified qt(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	Small strain Stiffness Ratio Factor, Kg	$[G_{max}/q_t]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

No.	Reference
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10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
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18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
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25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
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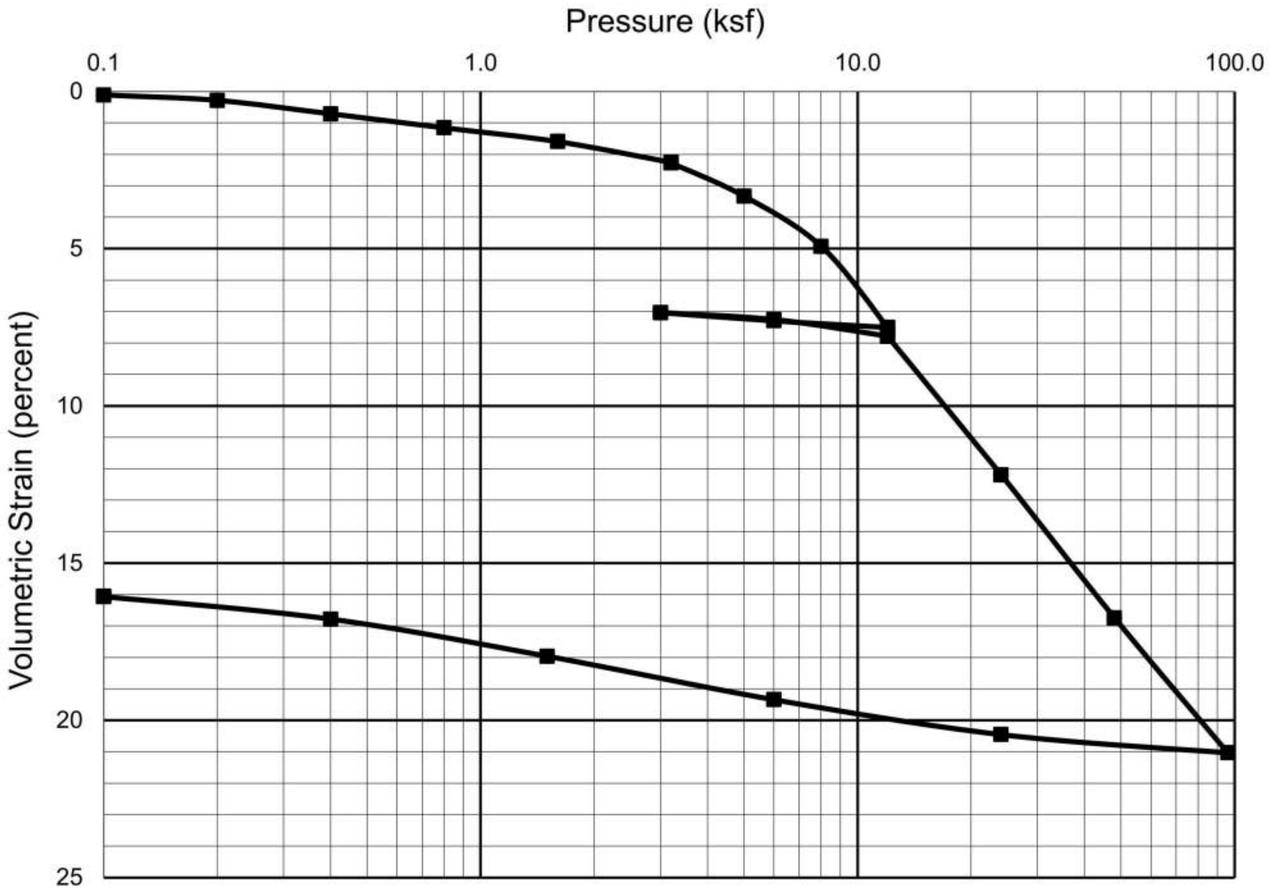
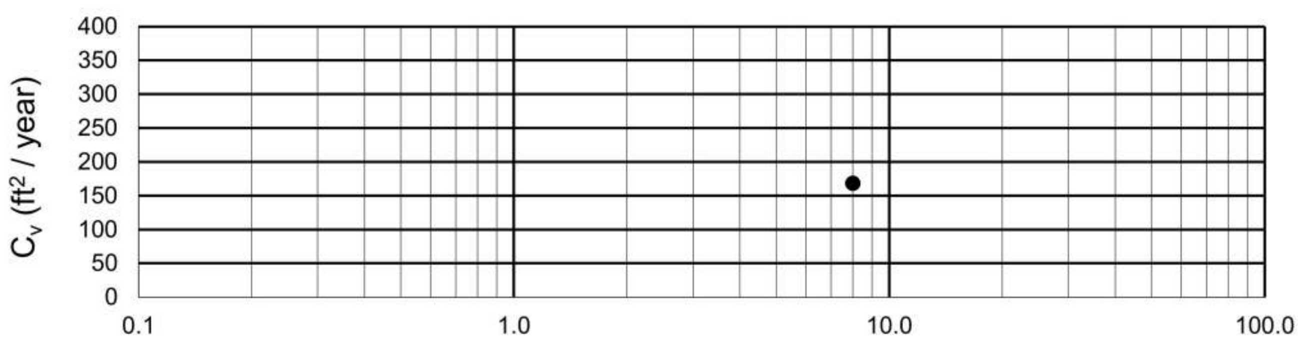
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.

APPENDIX C

LABORATORY TEST RESULTS





Sampler Type						Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00		Water Content	W_o	25.0%	W_f	17.2%
Overburden Pressure	P_o	1,750 psf				Void Ratio	e_o	0.74	e_f	0.46
Preconsol. Pressure	P_c	7,500 psf				Saturation	S_o	91%	S_f	100%
Compression Ratio	$C\varepsilon_c$	0.15				Dry Density	γ_d	97 pcf	γ_d	115 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)			
Classification:	CLAY (CL), gray					Source:	LB-1 at 45 feet			

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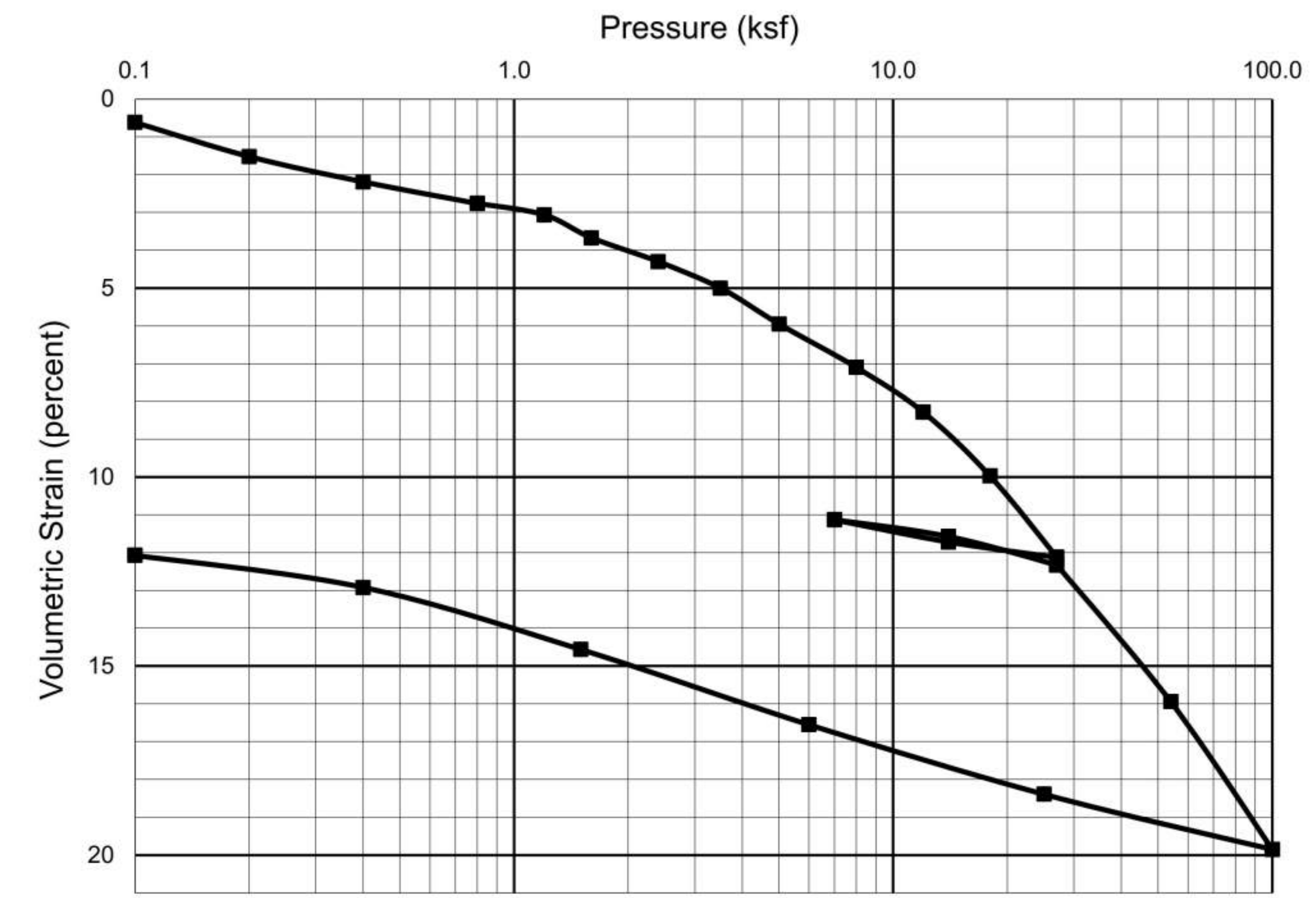
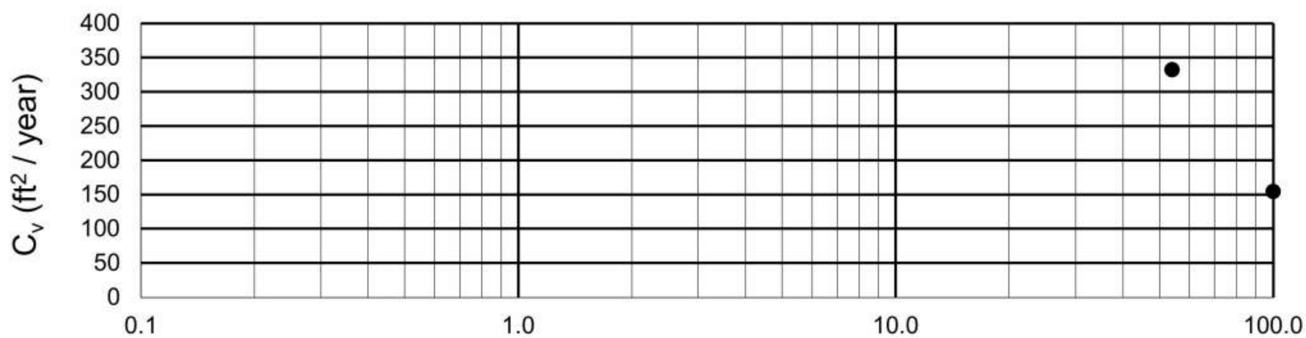
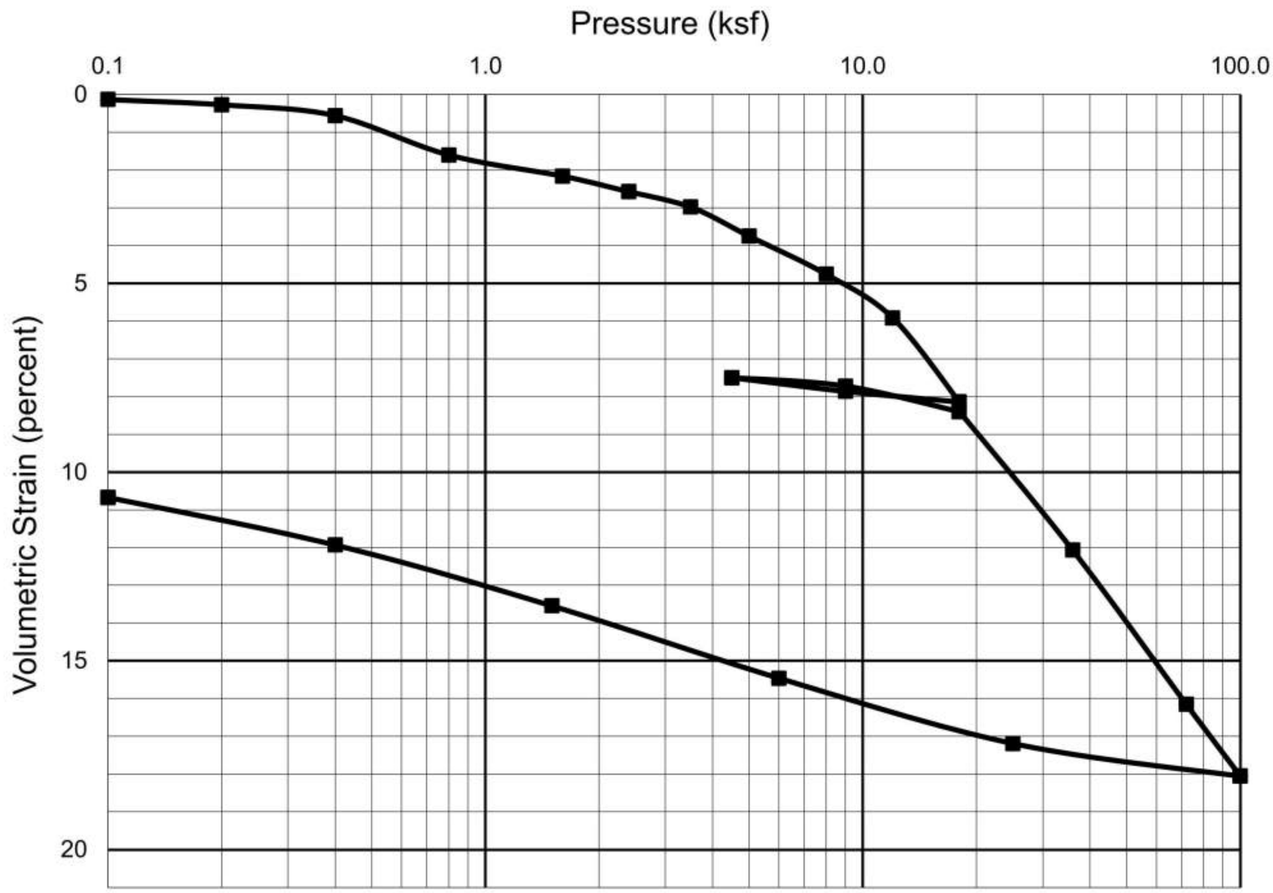
Project	Figure Title	Project No.	Figure
4590 PATRICK HENRY DRIVE SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	**CONSOLIDATION TEST REPORT**	Project No. 750664902 Date 07/31/2023 Drawn By AG Checked By PB	C-1

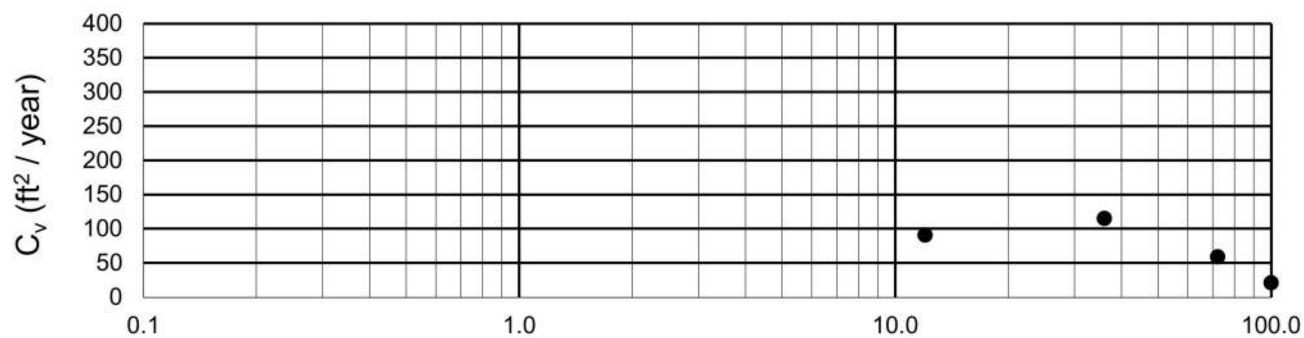




Sampler Type						Condition Before Test			After Test	
Shelby Tube	Diameter (in)		2.42	Height (in)	1.00	Water Content	w_o	24.3%	w_f	17.7%
Overburden Pressure		P_o	3,210 psf			Void Ratio	e_o	0.68	e_f	0.48
Preconsol. Pressure		P_c	10,200 psf			Saturation	S_o	96%	S_f	100%
Compression Ratio		$C\varepsilon_c$	0.14			Dry Density	γ_d	100 pcf	γ_d	114 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s		2.70 (assumed)		
Classification:	CLAY (CL), light brown					Source:	LB-2 at 40 feet			

LANGAN Langan Engineering and Environmental Services, Inc. 1814 Franklin Street, Suite 505 Oakland, CA 94612 T: 510.874.7000 F: 510.874.7001 www.langan.com	Project **4590 PATRICK HENRY DRIVE** SANTA CLARA SANTA CLARA COUNTY CALIFORNIA	Figure Title **CONSOLIDATION TEST REPORT**	Project No. 750664902 Date 07/31/2023 Drawn By AG Checked By PB	Figure C-2





Sampler Type					Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	22.7%	w_f	17.6%
Overburden Pressure	P_o	4,583 psf			Void Ratio	e_o	0.65	e_f	0.47
Preconsol. Pressure	P_c	10,200 psf			Saturation	S_o	94%	S_f	100%
Compression Ratio	$C\varepsilon_c$	0.14			Dry Density	γ_d	102 pcf	γ_d	114 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY with SAND (CL), olive-gray				Source:	LB-2 at 60 feet			

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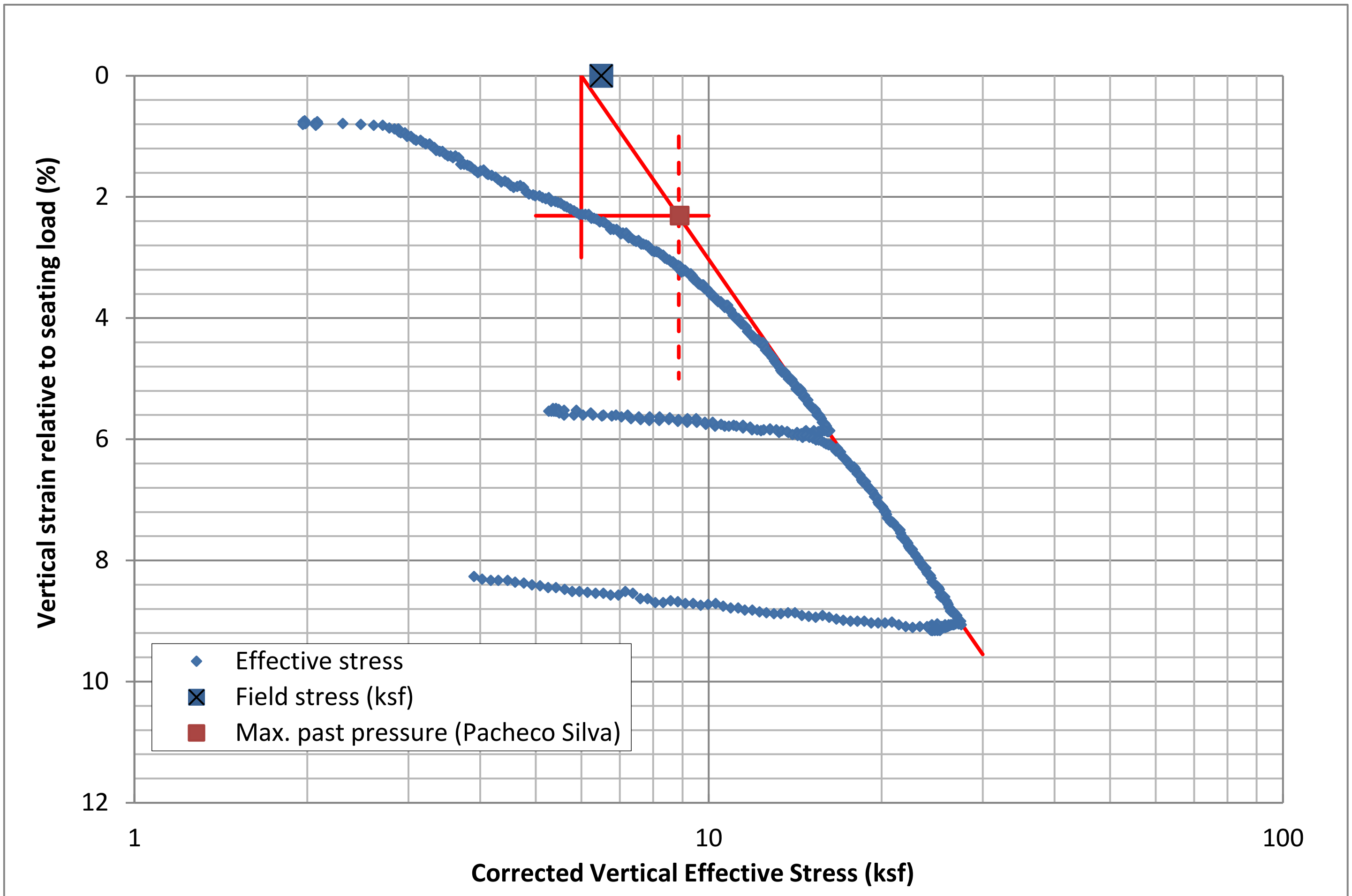
SANTA CLARA COUNTY CALIFORNIA

Figure Title
CONSOLIDATION TEST REPORT

Project No. 750664902
Date 07/31/2023
Drawn By AG
Checked By PB

Figure
C-3

Constant Rate of Strain Consolidation Test
ASTM D4186



Specimen	Depth	W.C.	Atterberg Limits			Fines content		
	(ft)	(%)	LL	PL	PI	(%)	Description	USCS
	56.5	21.9					gray-green clayey silt	

Initial Specimen Properties	
Height (mm)	20.40
Diameter (mm)	60.9
Volume (cm3)	59.42
Moist mass (g)	118.56
Moist density, ρ (g/cm3)	1.995
Total unit weight (pcf)	124.5
Gs (assumed)	2.66
Void Ratio e	0.622
Saturation	92.6

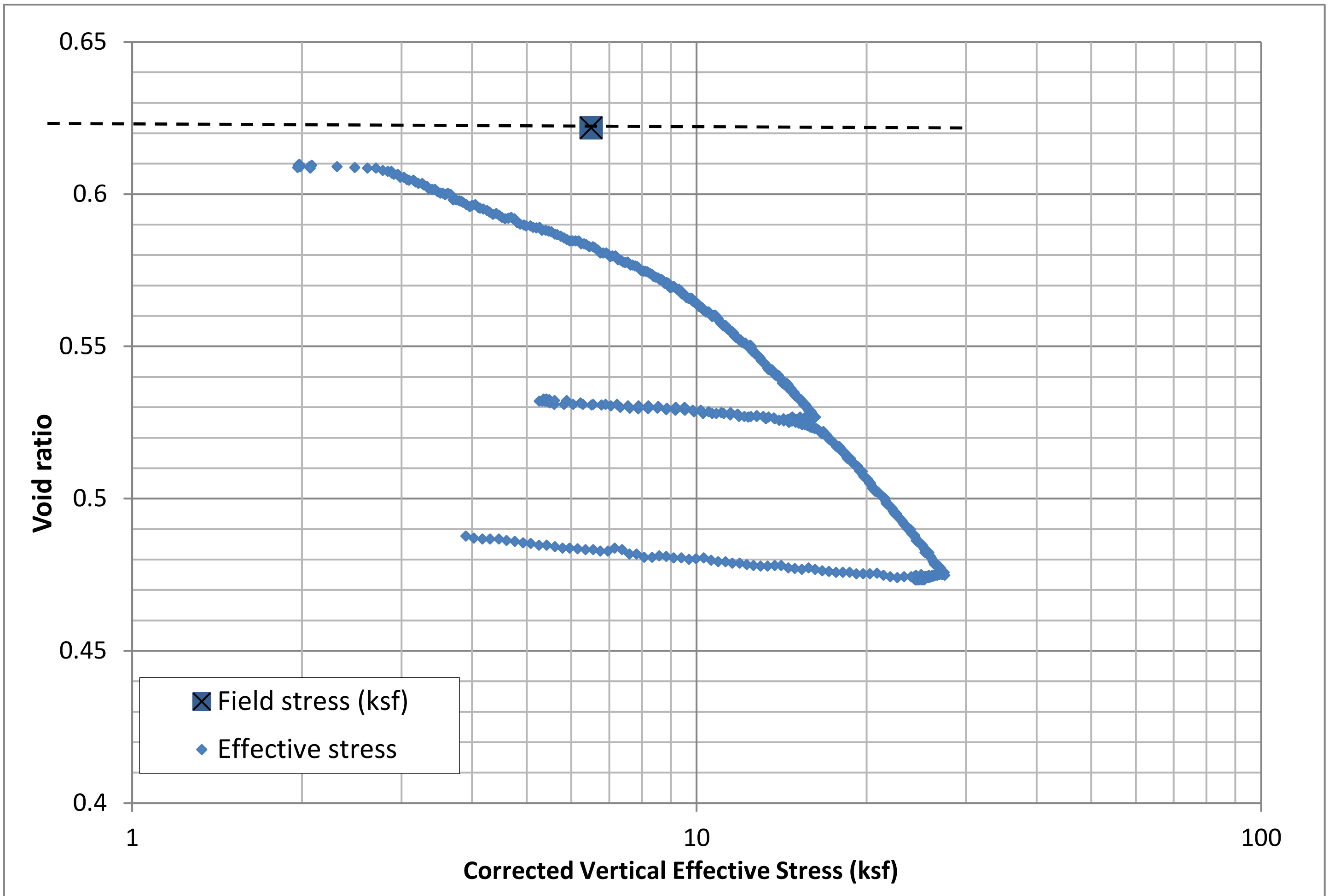
Stresses	(ksf)
Estimated vertical field effective	6.50
Maximum past (Pacheco Silva)	8.9
Maximum past (Work method)	8.9

Disturbance	
Δe / eo (%)	6.3
Sample quality (Lunne, 1997)	Good to Fair

Project:	4590 Patrick Henry Drive	**Test:**	**CRS1**
Location:	Santa Clara		Boring LB-2 @55ft
Project Number:	750664902		
Axial strain v. log (vertical effective stress)		**Figure:**	**C-4a**

Performed by: M. Riemer
at UCB Geotech labs

Constant Rate of Strain Consolidation Test
ASTM D4186



Specimen	Depth	W.C.	Atterberg Limits			Fines content		
	(ft)	(%)	LL	PL	PI	(%)	Description	USCS
	56.5	21.9					gray-green clayey silt	

Initial Specimen Properties	
Height (mm)	20.40
Diameter (mm)	60.9
Volume (cm3)	59.42
Moist mass (g)	118.56
Moist density, ρ (g/cm3)	1.995
Total unit weight (pcf)	124.5
Gs (assumed)	2.66
Void Ratio e	0.622
Saturation	92.6

Stresses	(ksf)
Estimated vertical field effective	6.50
Maximum past (Pacheco Silva)	8.9
Maximum past (Work method)	8.9

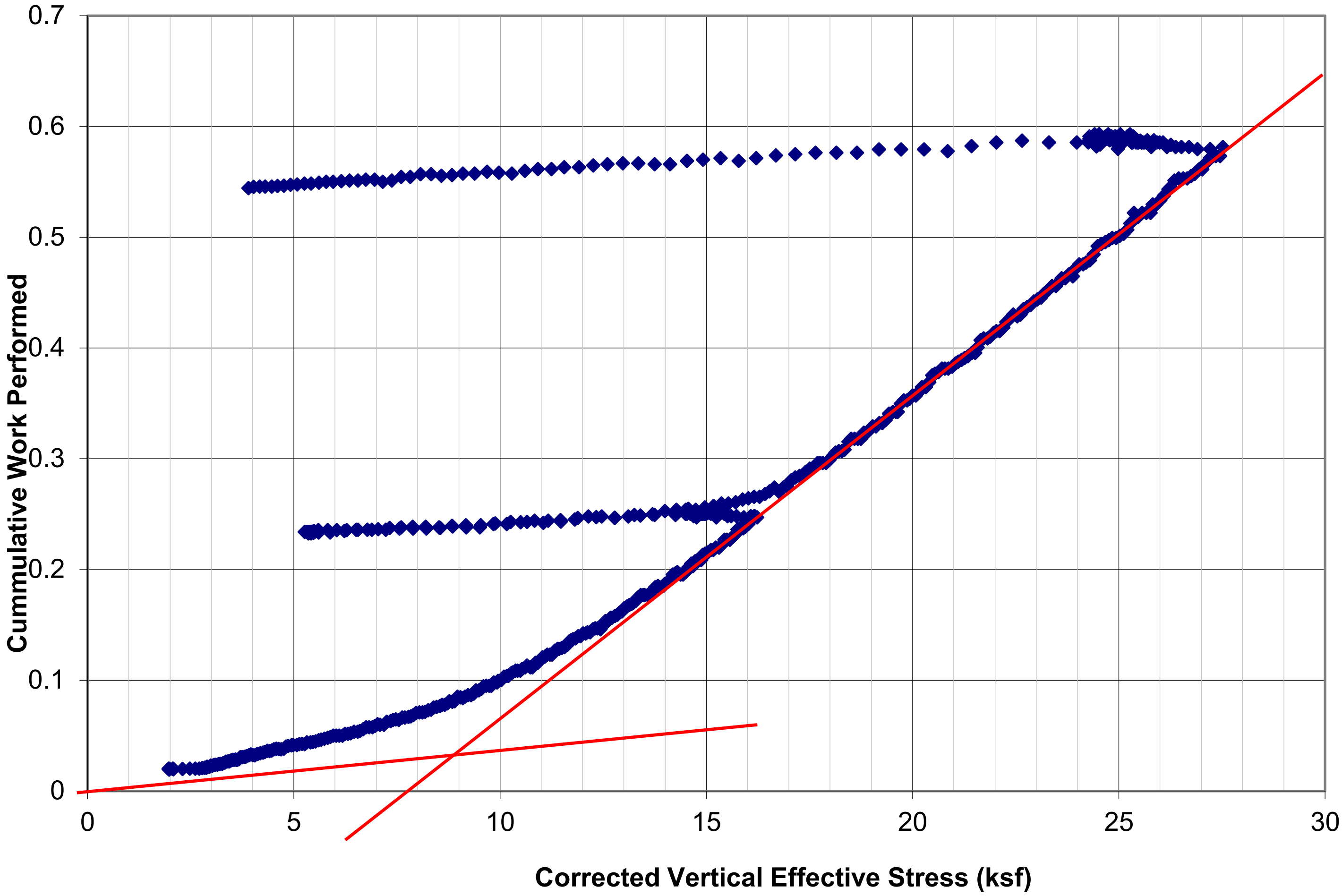
Disturbance	
Δe / eo (%)	6.3
Sample quality (Lunne, 1997)	Good to Fair

Project: 4590 Patrick Henry Drive **Test:** **CRS1**
Location: Santa Clara Boring LB-2 @55ft
Project Number: 750664902

Void ratio v. log (vertical effective stress) **Figure: C-4b**

Performed by: M. Riemer
at UCB Geotech labs

Constant Rate of Strain Consolidation Test
ASTM D4186



Specimen	Depth	W.C.	Atterberg Limits			Fines content	Description	USCS
	(ft)	(%)	LL	PL	PI	(%)		
	56.5	21.9					gray-green clayey silt	

Initial Specimen Properties	
Height (mm)	20.40
Diameter (mm)	60.9
Volume (cm3)	59.42
Moist mass (g)	118.56
Moist density, ρ (g/cm3)	1.995
Total unit weight (pcf)	124.5
Gs (assumed)	2.66
Void Ratio e	0.622
Saturation	92.6

Stresses	(ksf)
Estimated vertical field effective	6.50
Maximum past (Pacheco Silva)	8.9
Maximum past (Work method)	8.9

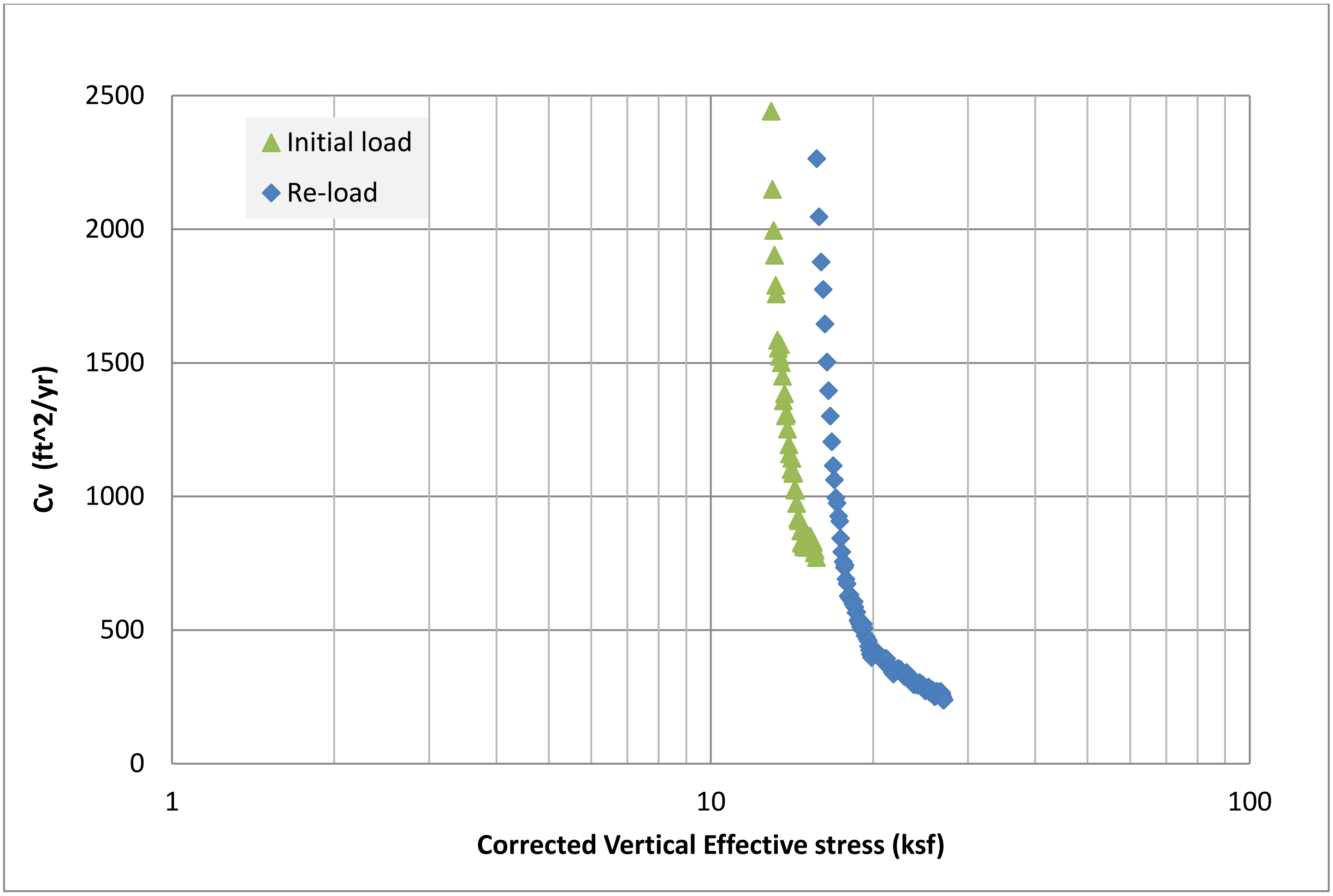
Disturbance	
Δe / eo (%)	6.3
Sample quality (Lunne, 1997)	Good to Fair

Project:	4590 Patrick Henry Drive	**Test:**	**CRS1**
Location:	Santa Clara		Boring LB-2 @55ft
Project Number:	750664902		

Cumulative work v. vertical effective stress **Figure: C-4c**
(Becker Method)

Performed by: M. Riemer
at UCB Geotech labs

Constant Rate of Strain Consolidation Test
ASTM D4186



Specimen	Depth	W.C.	Atterberg Limits			Fines content	Description	USCS
	(ft)	(%)	LL	PL	PI	(%)		
	56.5	21.9					gray-green clayey silt	

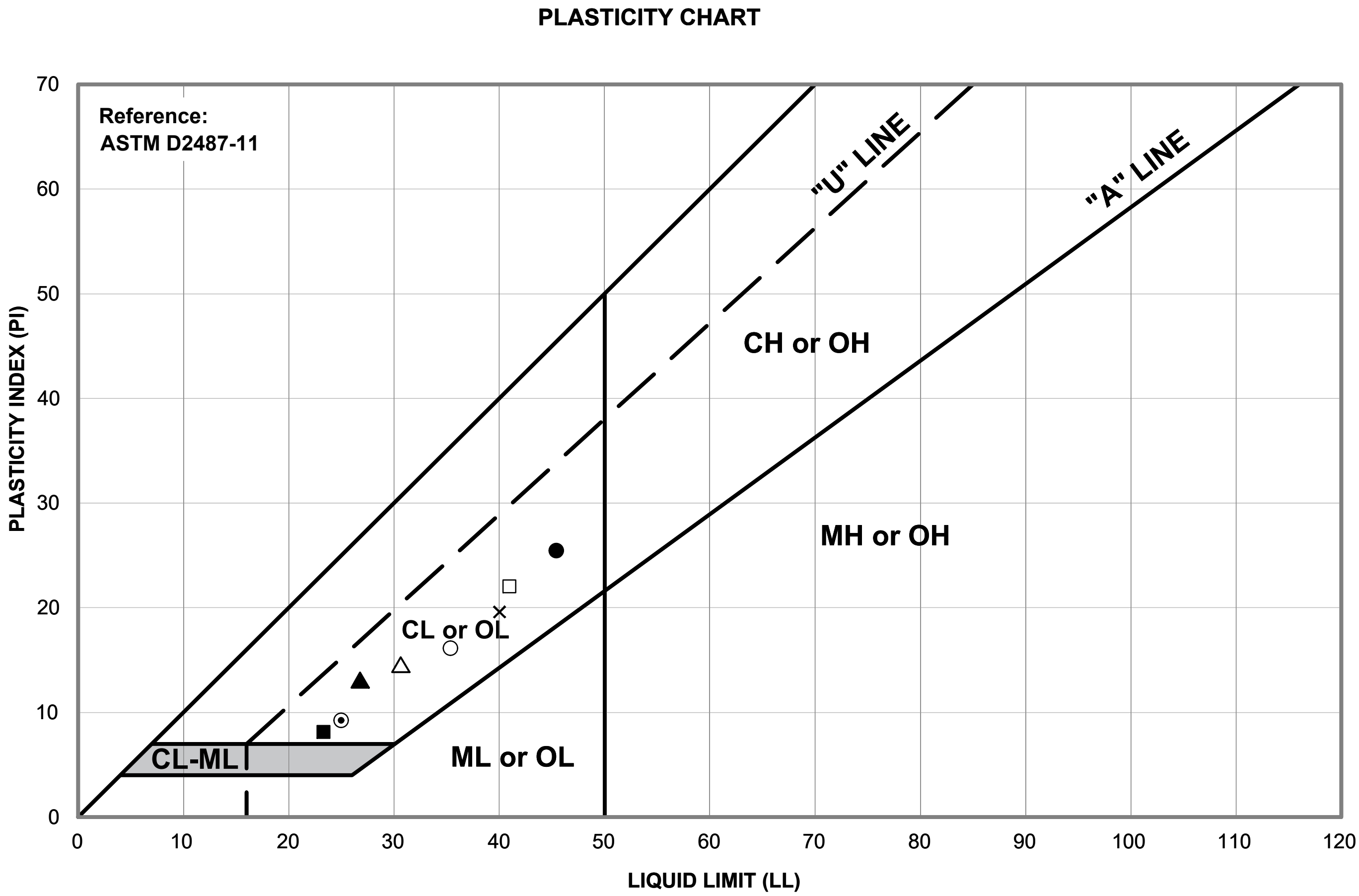
Initial Specimen Properties	
Height (mm)	20.40
Diameter (mm)	60.9
Volume (cm3)	59.42
Moist mass (g)	118.56
Moist density, ρ (g/cm3)	1.995
Total unit weight (pcf)	124.5
Gs (assumed)	2.66
Void Ratio e	0.622
Saturation	92.6

Stresses	(ksf)
Estimated vertical field effective	6.50
Maximum past (Pacheco Silva)	8.9
Maximum past (Work method)	8.9

Disturbance	
Δe / eo (%)	6.3
Sample quality (Lunne, 1997)	Good to Fair

Project:	4590 Patrick Henry Drive	**Test:**	**CRS1**
Location:	Santa Clara		Boring LB-2 @55ft
Project Number:	750664902		
Coeff.of Consol v. log(effective stress)		**Figure:**	**C-4d**

Performed by: M. Riemer
at UCB Geotech labs



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	LB-1, LB-2, LB-3 at 1 foot	CLAY (CL), dark brown to gray	--	46	25	--
■	LB-1 at 41 feet	SAND (SP), gray	19.1	23	8	33.2
□	LB-1 at 61 feet	SANDY CLAY (CL), olive-gray	22.7	41	22	74.3
○	LB-2 at 6 feet	CLAY with SAND (CL), olive-gray	--	36	17	--
▲	LB-2 at 20 feet	CLAYEY SAND (SC), light brown	11.1	27	13	15.4
△	LB-2 at 71 feet	CLAY (CL), olive-gray with yellow-brown mottling	20.1	31	14	83.0
×	LB-3 at 6 feet	CLAY (CL), olive-gray with yellow-brown and brown mottling	31.4	40	19	51.3
▣	LB-3 at 30 feet	SAND with CLAY (SP-SC), brown	16.7	--	--	12.7
⊙	LB-3 at 65 feet	CLAYEY SAND (SC), olive-gray	22.0	25	9	44.8

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Figure Title
PLASTICITY CHART

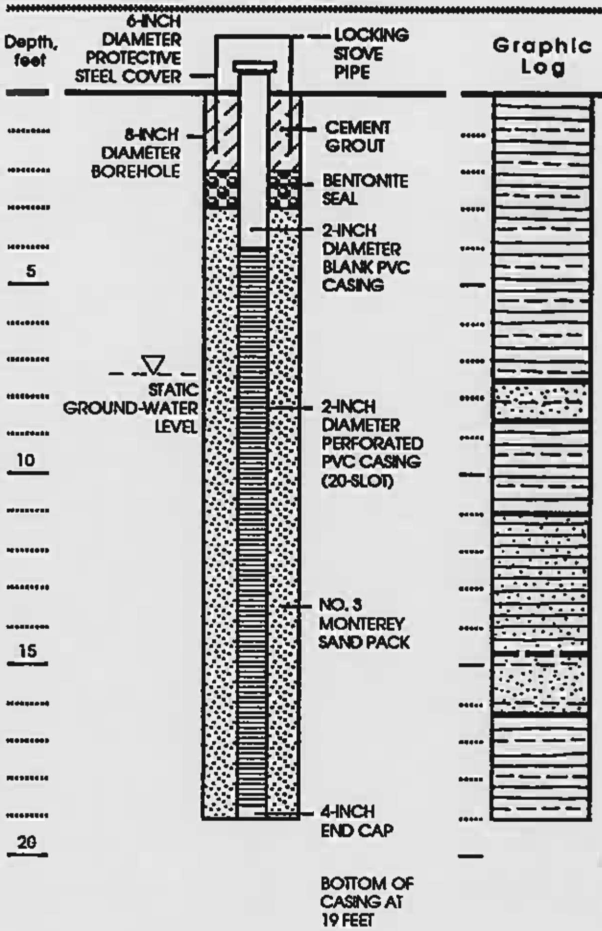
Project No. 750664902
Date 07/07/2022
Drawn By JDF
Checked By ECE

Figure
C-5

APPENDIX D

LOGS OF BORINGS BY OTHERS

WELL CONSTRUCTION



LITHOLOGY

Description

SILTY CLAY (CL), black (2.5Y N2/), dry, friable, medium stiff, low plasticity, massive. Sample slightly moist at 1 foot.

Color change to very pale brown (10YR 7/3) at 2.5 feet.

Color change to mottled brownish yellow (10YR 6/6) at 5 feet.

First water at 7.5 feet.

SILTY SAND (SM), mottled brown (10YR 5/3), wet, very loose, medium- to very fine-grained sand, poorly sorted.

SILTY CLAY (CL), mottled grayish brown (2.5Y 5/2), wet, soft, low plasticity, massive.

SANDY CLAY (CL), yellowish brown (10YR 5/4), wet, soft, low plasticity, massive.

SILTY SAND (SM), brown (10YR 4/3), wet, loose, coarse- to very fine-grained sand, minor amount of fine gravel in upper 0.3-foot, poorly sorted.

SILTY CLAY (CL), mottled dark yellowish brown (10YR 4/3), wet, medium stiff, high plasticity, massive.

BOTTOM OF BORING AT 19 FEET.

Sample No. and Interval	Penetration Rate (Blows/ft.)
LF-1-1	7
LF-1-2	20
	9
	4
	5
	7
	7
	16

Well Permit No.	89W1489
Date well drilled:	10 August 1989
Date water level measured:	11 August 1989
Well elevation:	Not Surveyed
Hammer weight:	140 lbs./30-inch drop
LF Geologist:	Mike Bombard

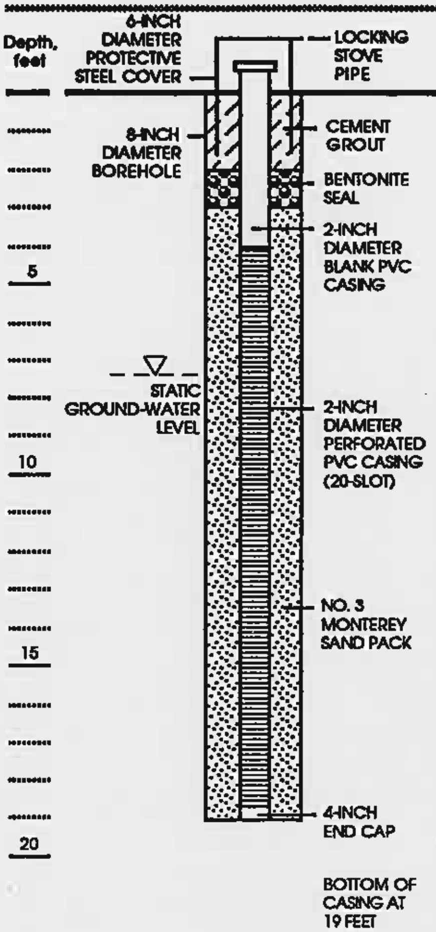
EXPLANATION

- Clay
- Silt
- Sand
- Gravel
- Modified California Sampler
- Sample retained for chemical analysis

Approved by: [signature]

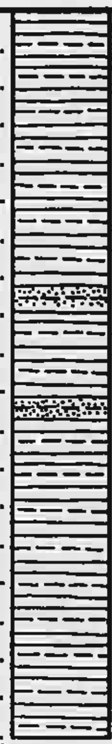
Figure A1 : WELL CONSTRUCTION AND LITHOLOGY FOR WELL LF-1

WELL CONSTRUCTION



LITHOLOGY

Graphic Log



Description	Sample No. and Interval	Penetration Rate (Blows/ft.)
SILTY CLAY (CL), very dark grayish brown (10YR 3/2), dry, slightly moist at 1.5 feet, stiff, low plasticity, massive, minor amount of plant roots.	LF-2-1	9
Color change to very pale brown (10YR 7/3) at 2.5 feet.	LF-2-2	13
Color change to light yellowish brown (10YR 6/4) at 5 feet, sample moist.		13
0.3-foot thick SILTY SAND lens at 7.7 feet to 8.0 feet, fine- to very fine-grained. Color change to mottled light brownish gray (2.5Y 6/2) at 8 feet. First water at 8 feet.		4
0.15-foot thick SILTY SAND lens at 10.8 feet to 10.95 feet.		
Color change to mottled dark yellowish brown (10YR 4/4) at 11 feet, sample wet.		6
		5
		8
Color change to mottled light olive-brown (2.5Y 5/4) at 17.5 feet, sample wet.		5
BOTTOM OF BORING AT 19 FEET.		

Well Permit No. 89W1490

Date well drilled: 10 August 1989

Date water level measured: 11 August 1989

Well elevation: Not Surveyed

Hammer weight: 140 lbs./30-inch drop

LF Geologist: Mike Bombard

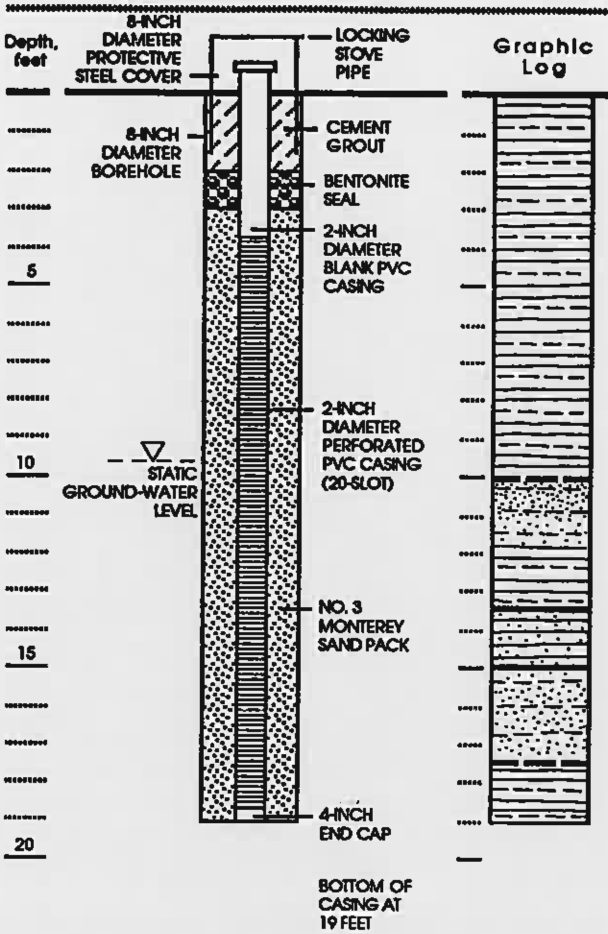
EXPLANATION

- Clay
- Silt
- Sand
- Gravel
- Modified California Sampler
- Sample retained for chemical analysis

Approved by: [signature]

Figure A2 : WELL CONSTRUCTION AND LITHOLOGY FOR WELL LF-2

WELL CONSTRUCTION	LITHOLOGY



Description	Sample No. and Interval	Penetration Rate (Blows/ft.)
SILTY CLAY (CL), black (2.5 N2/), dry, friable, medium stiff, low plasticity, massive. Sample slightly moist at 1 foot.	LF-3-1	10
Color change to light gray (10YR 7/2) at 2.5 feet.	LF-3-2	20
Color change to pale brown (10YR 6/3) at 3 feet.		
Color change to yellowish brown (10YR 5/4) at 5 feet.		5
Sample mottled at 7.5 feet.		5
First water at 9.8 feet.		
SILTY SAND to SANDY SILT (SM-ML), mottled light olive-brown (2.5Y 5/4), wet, very loose, fine- to very fine-grained sand, poorly sorted.		4
SILTY CLAY (CL), mottled dark yellowish brown (10YR 4/6), wet, soft, low plasticity, massive.		
SANDY CLAY (CL), mottled dark yellowish brown (10YR 4/4), wet, soft, low plasticity, massive, coarse-grained sand and trace amount of fine gravel.		7
SILTY SAND (SM), dark yellowish brown (10YR 4/4), wet, loose, coarse- to very fine-grained sand, poorly sorted, trace amount of fine gravel - well rounded.		9
SILTY CLAY (CH), mottled yellowish brown (10 YR 5/4), moist, medium stiff, high plasticity, massive.		17
BOTTOM OF BORING AT 19 FEET.		

Well Permit No.	89W1491
Date well drilled:	10 August 1989
Date water level measured:	11 August 1989
Well elevation:	Not Surveyed
Hammer weight:	140 lbs./30-inch drop
LF Geologist:	Mike Bombard

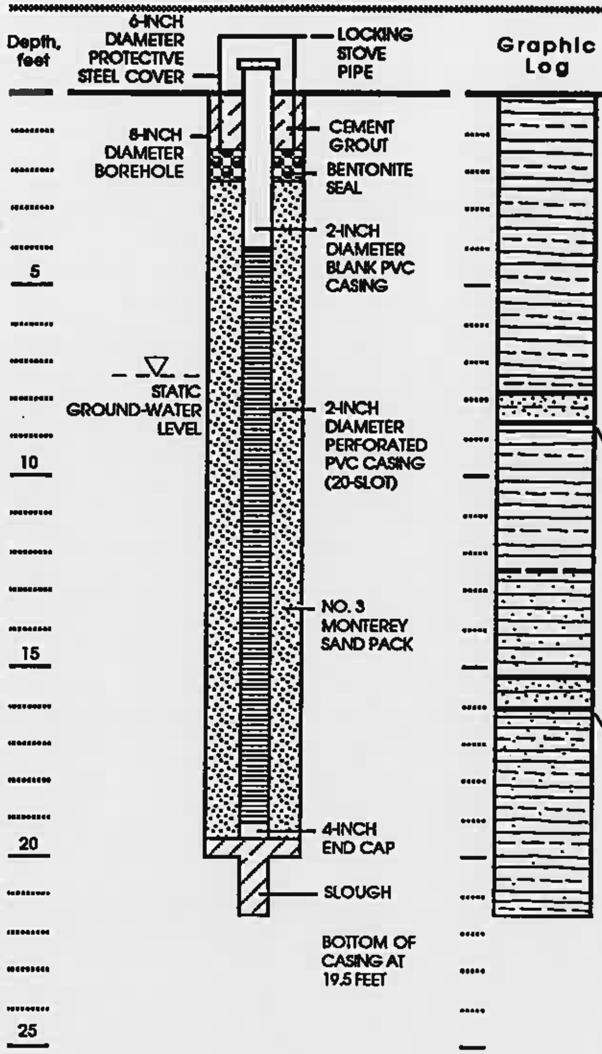
EXPLANATION

- Clay
- Silt
- Sand
- Gravel
- Modified California Sampler
- Sample retained for chemical analysis

Approved by:

Figure A3 : WELL CONSTRUCTION AND LITHOLOGY FOR WELL LF-3

• WELL CONSTRUCTION	LITHOLOGY



Description

SILTY CLAY (CL), black (2.5Y N2/), dry, friable, medium stiff, low plasticity, massive.

Color change to pale brown (10YR 6/3) at 2 feet, slightly moist.

Color change to mottled light brownish gray (10YR 6/2) at 5 feet.

Color change to mottled brown (10YR 4/3) at 7.5 feet. First water at 7.5 feet.

SILTY SAND (SM), brown (10YR 4/3), wet, loose, medium- to fine-grained sand, minor amount of clay, poorly sorted.

SILTY CLAY (CL), mottled olive brown (2.5Y 4/4), wet, soft, low plasticity, massive.
Color change to mottled brown (10YR 4/3) at 10 feet.

SANDY CLAY (CL), light olive-brown (2.5Y 5/4), wet, soft, low plasticity, massive.

CLAYEY SAND (SC), mottled yellowish brown (10YR 5/4), wet, medium loose, medium- to very fine-grained sand, poorly sorted.

SANDY SILTY CLAY (CL), mottled yellowish brown (10YR 5/4), wet, medium stiff, low plasticity, massive.
Sand very little below 17.5 feet.
Color change to dark yellowish brown (10YR 4/4) at 17.5 feet.

BOTTOM OF BORING AT 20 FEET.

BOTTOM OF SAMPLE BORING AT 21.5 FEET.

Sample No. and Interval	Penetration Rate (Blows/ft.)
LF-4-1	19
LF-4-2	17
	10
	5
	4
	5
	11
	13
	5

Well Permit No.	89W1492
Date well drilled:	10 August 1989
Date water level measured:	11 August 1989
Well elevation:	Not Surveyed
Hammer weight:	140 lbs./30-inch drop
LF Geologist:	Mike Bombard

EXPLANATION

Clay

Silt

Sand

Gravel

Modified California Sampler

Sample retained for chemical analysis

Approved by:

Figure A4 : WELL CONSTRUCTION AND LITHOLOGY FOR WELL LF-4

DISTRIBUTION

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