GEOTECHNICAL **FEASIBILITY** REPORT
ARLINGTON APARTMENTS
21117 59TH Avenue Northeast
Arlington, Washington

PROJECT NO. 24-207
July 2024

Prepared for:

Quarterra

PanGEO INCORPORATED

Geotechnical & Earthquake
Engineering Consultants

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July 25, 2024
PanGEO Project No. 24-207

Peter van Overbeek
Quarterra
1325 – 4th Avenue, Suite 1300
Seattle, Washington 98104

Subject: Geotechnical Feasibility Report
Arlington Apartments
21117 59th Avenue Northeast, Arlington, Washington

Dear Peter:

As requested, PanGEO, Inc. is pleased to present this geotechnical feasibility study for the proposed development in Arlington, Washington. In preparing this report, we drilled four test borings, performed four cone penetrometer tests, and conducted our engineering analyses. Because design details are not available at this time, additional geotechnical input may be needed during the final design phase of the project.

At our exploration locations, we encountered interbedded loose to medium dense sand, silty sand, and sand with gravel to the maximum exploration depth of about 52 feet below grade. Groundwater was encountered between 3 and 5 feet below grade in our exploration locations at the time of our subsurface explorations.

We understand it is planned to develop the site with a mixed-use development consisting of nine garden style apartment buildings and three retail buildings in the north and northwest portions of the site. We also understand that 2 to 5 feet will be placed to raise the site grade.

The results of our analysis indicate that the site soils are prone to soil liquefaction during a design level earthquake, which could result in a total liquefaction-induced settlement of two to five inches across the site during a design level seismic event; the differential settlement within each building would likely be less than half of the total settlement. To mitigate the potential for significant differential settlement due to soil liquefaction, we recommend supporting the buildings on the fill that is planned to raise site grades, provided the fill placed below the footings is at least two feet thick and reinforced with a geogrid.

Because groundwater was quite shallow (i.e., 2 to 5 feet deep at the time of subsurface explorations), construction dewatering may be required. In addition, it may impact the feasibility of infiltration.

We appreciate the opportunity to be of service. Should you have any questions, please do not hesitate to call.

Sincerely,

PanGEO, Inc.

Scott D. Dinkelman, LEG
Principal Engineering Geologist
SDinkelman@pangeoinc.com

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ATTACHMENTS:

GEOTECHNICAL FEASIBILITY REPORT
ARLINGTON APARTMENTS
21117 - 59TH AVENUE NORTHEAST
ARLINGTON, WASHINGTON

1.0 GENERAL

As requested, PanGEO, Inc. is pleased to present this geotechnical feasibility report for the proposed apartment and retail development located in Arlington, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated May 13, 2024. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, performing subsurface exploration program with a combination of four test borings and four cone penetrometer tests (CPTs), installing two groundwater monitoring wells, conducting laboratory tests for evaluating infiltration feasibility, providing engineering analyses, and preparing this geotechnical feasibility report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 21117 59th Avenue Northeast in the southeast corner of the intersection of the State Route 530 Northeast and 59th Avenue Northeast, in Arlington, Washington. The approximate location of the site is shown in Figure 1, Vicinity Map.

The rectangular shaped site comprises about 8.8 acres and is bordered to the north by State Route 530, to the west by 59th Avenue Northeast, to the south by vacant land and to the east by residences. The site is currently vacant of structures and the surface grade is relatively flat with less than about five feet of elevation change across the site. The site conditions are included in Plates 1 and 2, following page.

We understand it is planned to develop the site with nine garden style apartment buildings and three retail buildings. The proposed development will also include a clubhouse building, surface parking and driveway areas, landscaping, stormwater, and utility improvements. The proposed layout is shown in the attached Figure 2, Site and Exploration Plan.

We anticipate the garden style apartments will be three stories in height and of lightly loaded wood frame construction while the retail buildings will be one story in height and wood-frame or concrete masonry unit construction with slab on grade floors. It is planned to raise site grades by two to five feet in order to achieve construction subgrade elevations.

Surface water from the planned improvements will be collected and disposed of by infiltration into the site soils if the site conditions are conducive to this application.

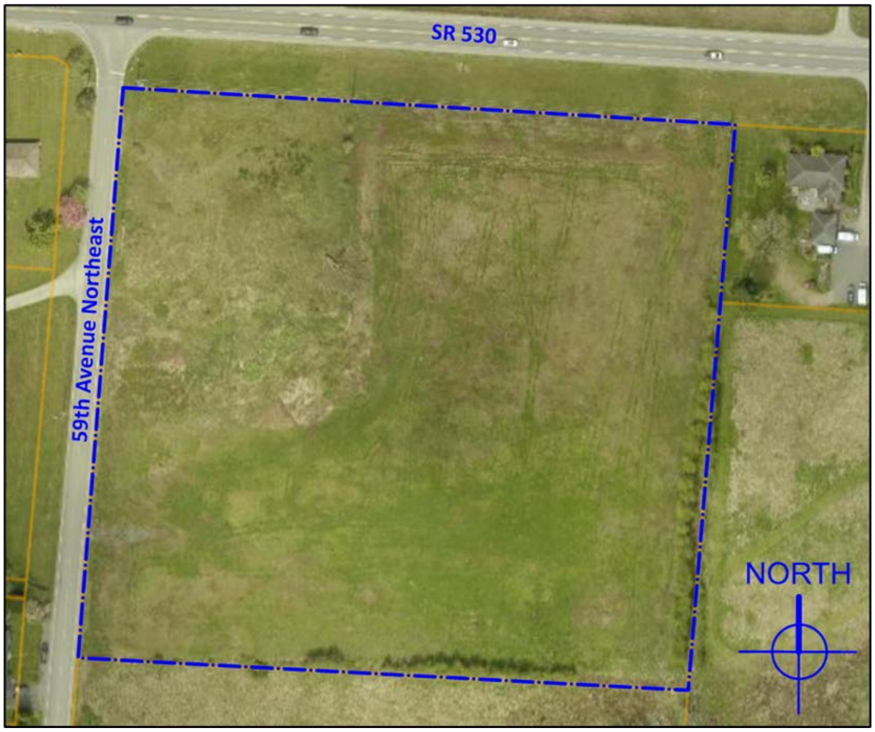


Plate 1: *Aerial view of the site.* - *Snohomish County, 2020*

Plate 2: *Ground level view of the site*

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATION

3.1 Test Borings

A total of four test borings identified as PG-1 through PG-4 were drilled at the project site on June 27 and 28, 2024. The approximate boring locations are indicated in the attached Figure 2 – *Site and Exploration Plan*. The borings were drilled to a depth of 41½ feet below grade using track-mounted drill rig.

The drill rig was equipped with 6-inch diameter hollow stem augers and soil samples were obtained from the borings at 2½- and 5-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches operated with a rope and cathead mechanism. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. Extra soil samples at the ground surface are obtained at these boring logs to have a better understanding of the subsurface soil conditions.

A geologist from PanGEO was present throughout the field exploration program to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings. The completed boreholes were backfilled and sealed with bentonite chips.

The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A, and the summary boring logs are included as Figures A-2 through A-5.

3.2 STANDPIPE PIEZOMETER INSTALLATION

To facilitate future monitoring of groundwater levels, standpipe piezometers were installed in Borings PG-1 and PG-2. The standpipe piezometers consisted of a 2-inch diameter, PVC well casing. The bottoms of the piezometers were generally located at 30 to 40 feet below existing grade and were screened in the lower 10 feet. The annular space around the screen was backfilled with Colorado Silica (10-20) sand to act as a filter against the adjacent soil. The sand filter extended to five feet above the top of the well screen. The annular space above the sand pack was backfilled with bentonite chips to within one-foot of the ground surface to form a water-tight seal. The wells were finished at the surface with a flush-mounted steel monument. Details of the well construction are shown on the appropriate boring logs.

The piezometers were developed to remove drilling mud and foreign material left from drilling and to establish good hydraulic conductivity between the well and the aquifer. Development of the wells was accomplished by a combination of pumping and mechanical surging with a bailer until the well water appeared clear.

Pressure transducer data loggers have been installed in the piezometers to monitor the fluctuations in groundwater levels over time.

The piezometers should be decommissioned prior to construction by a licensed driller in accordance with Washington Department of Ecology requirements.

3.3 CONE PENETRATION TESTS

Four cone penetration tests (CPTs) identified as CPT-1 through CPT-4 were completed on June 28, 2024, at the approximate locations indicated in the attached Figure 2. The CPT probes were advanced to depths of 34 to 53½ feet below the existing ground surface in general accordance with ASTM D-5778, *Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils*.

The CPTs were accomplished by pushing an approximately 1½ inch diameter instrumented cone into the soil at a steady rate to measure tip resistance (Qt), side friction (Fs), pore water pressure (u), soil behavior type, and correlated SPT blow counts. The readings on the soil are taken at about 2-inch vertical intervals, and provide a nearly continuous readout of soil stratigraphy, strength, and other parameters. Summary CPT logs are included in Appendix B of this report.

Three pore water dissipation tests were conducted in the CPTs to estimate the static groundwater levels at the test locations. The dissipation test results are included in Appendix B.

3.4 Laboratory Tests

Laboratory tests were conducted on representative soil samples obtained from the test borings, to verify or modify the field soil classification and to evaluate the general physical properties and engineering characteristics of the soil encountered. Visual field classifications were supplemented by grain size analyses on representative soil samples.

We submitted a total of four samples for particle size distribution testing in accordance with ASTM D-422 *Standard Test Method for Particle-Size Analysis of Soils*. The results of the grain size determinations for the samples were used in classification of the soils and are presented in Appendix C.

Four samples were also submitted to Kuo Testing Laboratories for cation exchange capacity (CEC) and organics content testing. The CEC is a calculated value that estimates the soil's ability to attract, retain, and exchange cation elements. It is reported in milliequivalents per 100 grams of soil (meq/100g). The test results are summarized in Tables 2 and 3 in Section 5.0 of this report.

It is important to note that these test results may not accurately represent the overall in-situ soil conditions. Our geotechnical recommendations are based on our interpretation of these test results and their use in guiding our engineering judgment.

4.0 SUBSURFACE CONDITIONS

4.1 Site Geology

Based on review of the *Distribution and Description of the Geologic Units in the Arlington West Quadrangle, Washington* (Minard, 1980), the geologic units in the vicinity of the site consist of Quaternary younger alluvium (Geologic Map Unit Qyal) and the Quaternary Marysville sand member (Qvrm). Quaternary younger alluvium is comprised of fine to coarse grained sand with silt and clay deposited in stream channels and as overbank flood deposits adjacent to stream channels.

The Marysville sand member consists of well-drained, stratified to massive outwash sand, some fine gravel, and silt and clay. This unit typically underlies the Quaternary younger alluvium.

4.2 Soil Conditions

For a detailed description of the subsurface conditions encountered at each exploration location, please refer to the boring and CPT logs provided in Appendices A and B. The following is a generalized description of the soils encountered in our borings and CPTs and our general understanding of the site geology.

Topsoil: At all of our boring locations, we encountered a surficial layer of topsoil and roots. The topsoil layer consisted of dark silty sand with organics. This layer was typically 6 to 12 inches thick at our test boring locations.

Quaternary Younger Alluvium (Qyal): Below the topsoil, we encountered very loose to medium dense silty fine to coarse sand with gravel and poorly grades sand with silt containing scattered organics. We classified this material as Quaternary younger alluvium which is mapped as underlying this area. This unit extended to about 15 to 26 feet below existing grade.

Marysville Sand Member (Qvrm): Underlying the younger alluvium, we borings encountered poorly graded very fine to coarse grained sand with thin lenses of silty sand. This unit varied from medium dense to dense. We classified this material as Marysville sand member recessional outwash which is mapped as underlying this area. Borings PG-1, PG-2, and PG-3 were terminated in recessional outwash at 41½ feet below grade.

Vashon Till (Qvt): At the location of Boring PG-4, we encountered dense to very dense silty sand with gravel at about 35½ feet below grade. We classified this material as Vashon till, which stratigraphically underlies the Maryville sand member.

Our CPTs were terminated in very dense soils at 34 to 53½ feet below grade due to refusal of equipment, likely on Vashon till.

Our descriptions of subsurface conditions are based on the conditions encountered at the time of our exploration. Soil conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.3 Groundwater

Groundwater was encountered at three to five feet below grade at the time of our subsurface explorations on June 27 and 28, 2024. It should be anticipated that groundwater will be a construction consideration in excavations for utilities and associated structures.

The design team should also be aware there will be fluctuations in groundwater levels depending on the season, amount of rainfall, surface water runoff, and other factors. Generally, the water level is higher and seepage rates are greater in the wetter, winter months (typically October through May).

We will be monitoring groundwater during the permitting process and will provide quarterly summaries of groundwater levels.

5.0 PRELIMINARY INFILTRATION FEASIBILITY

5.1 Infiltration Rate Determination

A preliminary infiltration evaluation was conducted in general accordance with the 2019 Washington Department of Ecology Stormwater Management Manual for Western Washington (WDOE Manual) (WDOE, 2019). Specifically, we used the Soil Grain Size Analysis method outlined in the WDOE Manual, Volume V, Section 5.4 to estimate the infiltration rates.

Representative samples of the site soils were collected from 2½ to 5 feet below grade. The samples were submitted to a soil testing laboratory for grain size distribution testing in general accordance with ASTM D-422 *Standard Test Method for Particle-Size Analysis of Soils*. The saturated hydraulic conductivity of the materials was then estimated from the grain size distribution testing using the following equation from the WDOE Manual.

$$\log10(Ksat) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines}$$

Where, D_{10}, D_{60} and D_{90} are the grain sizes in mm for which 10 percent, 60 percent and 90 percent of the sample is finer and f_{fines} is the fraction of the soil (by weight) that passes the #200 sieve.

The results of the grain size distribution testing are provided in Appendix c. Table 1 below summarizes the results of our testing and estimated saturated hydraulic conductivities.

5.2 Correction Factors for Design Infiltration Rate

The grain size test method provides an uncorrected, saturated hydraulic conductivity (K_{sat}) of the soil. The results of our infiltration tests are summarized in Table 1 on the next page. To provide a long-term design infiltration rate, the K_{sat} value is factored by applying a series of correction factors (CF) outlined in Table 3.3.1 of the WDOE Manual. As discussed below, the correction factors account for the test method (CF_t), influent control (CF_m) and site variability (CF_v). The value of each of these correction factors are discussed in Sections 5.2.1, 5.2.2 and 5.2.3, below.

5.2.1 Test Method

The correction factor for the test method (CF_t) is used to account for differences between the test method and in-situ infiltration testing. WDOE Manual specifies a CF_t value of 0.4 based on the use of the grain size method.

5.2.2 Influent Control

The influent control correction factor (CF_m) is intended to account for a reduction in infiltration capacity due to clogging from siltation and the build-up of biological material. An influent control factor of 0.9 was used in our calculation, assuming that when the infiltration systems lose 10 percent of their infiltration capacity due to clogging, the system will be maintained or cleaned.

5.2.3 Site Variability

The correction factor for site variability (CF_v) is intended to correct for the number of locations sampled and the consistency of the underlying soil conditions. The value for CF_v ranges from 0.33 to 1.0. Based on the number of exploration locations, relatively uniform soil conditions encountered at our exploration locations and our experience and engineering judgment, we assigned a correction factor of 0.8 for site variability.

5.2.4 Correction Factor

The total correction factor ($CF = CF_v \times CF_t \times CF_m = 0.29$) was then applied to the infiltration rate to obtain a corrected infiltration rate appropriate for long term design purposes.

5.2.5 Design Infiltration Rate

With the Total Correction Factor (CF_T) of 0.29, the long-term design rate can be calculated from the field measured rates. Table 1 on the next page summarizes the estimated infiltration rates based on the sieve testing.

TABLE 1: Summary of Infiltration Rates

Test Location	Sample Depth [feet below grade]	Soils Types	Sieve-Based Infiltration Rate K_{SAT} [inches/hour]	Correction Factor			Design Infiltration Rate [inches/hour]
				CF_v	CF_t	CF_m	
PG-1	5 – 6½	SP-SM	34	0.8	0.4	0.9	9.9
PG-2	2½ – 4	SM	4	0.8	0.4	0.9	1.2
PG-3	5 – 6½	SP-SM	22	0.8	0.4	0.9	6.4
PG-4	2½ - 4	SM	24	0.8	0.4	0.9	7

5.3 Cation Exchange Capacity Test Results

The WDOE Manual specifies that soils used for treatment and infiltration should have a Cation Exchange Capacity (CEC) of greater than or equal to 5 milliequivalents per 100 grams of dry soil (meq/100g). CEC testing was performed on four representative samples from our test borings. Table 2, below, provides a summary of the CEC test results.

TABLE 2: Cation Exchange Capacity Test Results

Location	Soil Sample Depth [feet below grade]	CEC [meq/100g]
PG-1	5 – 6½	4.6
PG-2	2½ – 4	5.5
PG-3	5 – 6½	4.3
PG-4	2½ - 4	7.9

Based on the results of the testing the soils in the infiltration system area may need to be amended to meet the minimum CEC value of 5 meq/100g required if the soils will be relied on for stormwater treatment.

5.4 Organic Content Test Results

Four representative samples collected from our borings were submitted to determine the percentage of organic material in the soils at our infiltration test locations. The testing procedure was performed in general accordance with the ASTM D2974-13 *Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils*. Table 3, below, provides a summary of the organic material test results.

TABLE 3: Organic Matter of Organic Soils Test Results

Location	Soil Sample Depth [feet [below grade]	Organic Content [%]
PG-1	5 – 6½	0.54
PG-2	2½ – 4	0.77
PG-3	5 – 6½	0.52
PG-4	2½ - 4	1.28

Based on the results of our tests, the soils in the infiltration system area may need to be amended if the soils will be relied on for stormwater treatment.

5.5 Infiltration Feasibility

Based on the results of our subsurface exploration, laboratory testing, infiltration of stormwater should generally be feasible at the site. Amendment of the soils in the base of the infiltration systems may be needed if they are going to be relied on for treatment.

However, the infiltration systems will need to maintain at least five feet of separation from the wet season high groundwater elevation. Because the groundwater was encountered at 2 to 5 feet deep at the time of subsurface exploration, this requirement may not be met, unless the site grade is raised. We will be monitoring the groundwater levels in the piezometers installed at the locations of Borings PG-1 and PG-2 over the upcoming wet season.

When the locations and elevations of the infiltration facilities are identified, additional exploration and in-situ infiltration testing will need to be performed.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN PARAMETERS

6.1.1 Seismic Site Class

We anticipate the project design will follow either the 2018 or 2021 edition of the International Building Code (IBC). Both editions of IBC specify a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and both IBC reference ASCE 7-16 for site class determination.

Because the site soil is considered prone to soil liquefaction (see discussions in Section 6.1.2 of this report), the site would be considered Site Class F in accordance with the IBC.

For Site Class F, Section 11.4.8 of ASCE-7-16 states that "*A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless the exception to Section 20.3.1 is applicable*." The exception in Section 20.3.1 (1) of ASCE 7-16 states that "*For structures having fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2*." In other words, for structures with a period of vibration equal to or less than 0.5 second and situated on liquefiable soils, the IBC/ASCE 7-16 exception allows the values of F_a and F_v for liquefiable soils be taken equal to the values of site class determined without regard to soil liquefaction.

Since the site will be developed with one story retail buildings and three-story apartment buildings, we anticipate the fundamental period of vibration of the structures will be less than 0.5 second, and the seismic site coefficients should be determined based on a Site Class D, based on the calculated average equivalent SPT N-values around 24 blows/ft obtained from SPT and CPT testing per Section 20.4.2 of ASCE 7-16.

The fundamental period of the proposed buildings should be confirmed by the structural engineer. If the fundamental period of the proposed buildings is equal to or greater than 0.5 seconds, a site-specific ground response analysis will be needed.

6.1.2 Liquefaction

Liquefaction occurs when saturated, predominately sand and silt are subjected to cyclic loading during a strong seismic event. This causes the porewater pressure to increase in the soil, thereby reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the soil decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil particles will be in suspension and behave like a viscous fluid. Typically, loose, saturated, sand and silt that have a low enough permeability to prevent drainage during cyclic loading have the greatest potential for liquefaction. Soil liquefaction may cause the temporary loss/reduction of foundation capacity and settlement.

To evaluate the risk of soil liquefaction, we performed liquefaction analysis based on the subsurface information from CPT-1, CPT-2, CPT-3 and CPT-4. The analyses were conducted using the computer liquefaction assessment software program CLiq and the method proposed by Boulanger & Idriss (2014). An earthquake with a magnitude of 7.5 and a PGA_M of 0.52g (based on Site Class E) was used in our analysis, consistent with the 2018 and 2021 IBCs.

Our analysis indicates that the risk of soil liquefaction is high and the soils below the groundwater table (about 2 to 5 feet below existing grade) to below 60 feet could liquefy during a seismic event consistent with the 2018 and 2021 IBCs. The results of our liquefaction analysis are included in Appendix D for references.

6.1.3 Liquefaction-Induced Free Field Settlement

Ground settlement should be expected to occur in the event of soil liquefaction. We calculated the potential liquefaction-induced settlement based on the results of CPTs using software program CLiq and the method proposed by Cetin et al. (2009), which is a probabilistically based model for the assessment of cyclically induced straining of saturated cohesionless soils. This approach includes a depth correction factor that assumes contribution of layers to surface settlement diminishes as the depth of layer increases, and settlement deeper than 60 feet should not manifest at the ground surface.

The calculated free-field settlements at the ground surface ranged from two to five inches across the site. Differential settlement is estimated to be one half of the total settlement.

6.2 Building Foundations

Because the proposed buildings will consist of lightweight wood-frame structures and single-story CMU structures and with the plan to raise site grades by two to five feet, there should be a non-liquefied soil crust at the surface. During recent earthquakes in Seattle, including our observations of building performance in areas experienced liquefactions in Seattle during the 2001 Nisqually Earthquake, these types of buildings performed well with no reported severe damages. As such, it is our opinion that the proposed buildings may be supported on conventional footings bearing on the fill used to raise site grades, provided the fill below the building areas is reinforced with a geogrid.

The footings will need to be tied together as indicated in *Section 12.13.8.2 Foundation Ties* of ASCE 7-16.

Based on an analysis by Bray and Macedo (2017), as long as the factor of safety (FS) of post-liquefaction bearing capacity is above 1.5, the building settlements due to liquefaction are not large. As the FS drops below 1.0, the liquefaction-induced building settlement can increase significantly. Based on our analysis, with the foundation approach described above, a FS of 1.5 can be achieved.

Recommendations for subgrade preparation and design parameters are described below.

6.2.1 Building Subgrade Preparation

Prior to placing the structural fill below the building footprints, the topsoil layer should be stripped and removed, and the building footprints cut to an elevation of at least two feet below the foundation subgrade elevation. The exposed subgrade should then be proofrolled with a minimum of a 10-ton roller or a fully load dump truck. Any loose/soft areas identified during the proof roll should be overexcavated and replaced with structural fill. A representative from PanGEO should observe the proofrolling and subgrade preparation operation to identify any excessively loose or soft areas that may require additional overexcavation or other treatment measures.

After the subgrade is adequately prepared, a layer of geogrid should be placed over the footprints of the individual buildings. The geogrid should consist of Tensar UX-1400 or an approved alternative. The intent of the geogrid is to mitigate the risk of differential settlement. The geogrid should extend at least two feet beyond the footprints of the building areas.

Structural fill should then be placed on the geogrid and compacted in accordance with Section 7.2 of this report. The structural fill should consist of imported granular soils with a maximum dimension of 4 inches, less than 30 percent passing the U.S. Standard No. 40 sieve, and less than 7 percent passing the U.S. Standard No. 200 sieve. The fine-grained portion of the structural fill should consist of non-plastic material.

6.2.2 Allowable Bearing Pressure & Subgrade Modulus

We recommend a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf) and a subgrade modulus of 150 pci be used to size the foundation elements.

The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind; however, the one-third increase should not be applied for seismic forces due to potential for soil liquefaction.

For frost protection considerations, exterior foundation elements should be placed at a minimum depth of 18 inches below final exterior grade. Interior spread foundations should be placed at a minimum depth of 12 inches below the top of concrete slabs.

6.2.3 Estimated Settlement

In general, with conventional foundations supported on at least two feet of structural fill designed and constructed in accordance with the above recommendations should experience total settlement of about one inch and differential settlement of about ½ inch. Most of the anticipated settlement should occur during construction as dead loads are applied.

Additional settlements could occur during the design seismic event. Estimation of foundation settlement during seismic conditions and liquefaction are difficult at best. According to Bray and Macedo (2017), as long as the factor of safety (FS) of post-liquefaction bearing capacity is above 1.5, the building settlements due to liquefaction are not large. As the FS drops below 1.0, the liquefaction-induced building settlement can increase significantly. Based on our analysis, with the foundation approach described above, an FS of 1.5 can be achieved, and hence we anticipate the proposed buildings to perform adequately.

With the building supported on the geogrid reinforced fill and the relatively uniform subsurface conditions indicated in the boring and CPT logs, we anticipate the differential settlement to be less than one inch across the length of the proposed buildings.

6.2.4 Lateral Resistance

Lateral loads on the foundation elements may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils.

- A frictional coefficient of 0.45 may be used to evaluate sliding resistance developed between the foundation and the structural fill subgrade soil. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.
- Passive soil resistance may be calculated using an equivalent fluid weight 350 pcf, assuming foundations are backfilled with properly compacted structural fill and level ground surface.

The above values include a factor of safety of 1.5.

6.2.4 Perimeter Foundation Drains

Footing drains should be installed around the perimeter of the buildings, at or just below the invert of the footings. As a minimum, 4-inch diameter perforated drainpipes should be installed next to the base of the footings and embedded in 12 to 18 inches of pea or washed gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet and should not be allowed to daylight over the slope.

Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.3 FLOORS SLABS

The floor slabs for the proposed buildings should be supported on the structural fill recommended in Section 7.2 of this report.

Interior concrete slab-on-grade floors should be underlain by a capillary break placed below the slab. The capillary break should meet the gradational requirements provided in Table 4, below.

TABLE 4: Capillary Break Gradation

Sieve Size	Percent Passing
¾-inch	100
No. 4	0 – 10
No. 100	0 – 5
No. 200	0 – 3

A 10-mil polyethylene vapor barrier should also be placed directly below the slab. Construction joints should be incorporated into the floor slab to control cracking.

6.4 Retaining Wall Design Parameters

Retaining walls should be designed to resist the lateral earth pressure of the retained soils and hydrostatic pressures, if applicable. We assume that retaining walls for this project will be located above groundwater.

We recommend cantilevered walls be designed for an equivalent fluid weight of 35 pcf, and braced wall be designed for an equivalent fluid weight of 50 pcf.

For the seismic condition, we recommend including an incremental uniform lateral earth pressure of 8H psf (where H is the height of the below grade portion of the wall) as an ultimate seismic load.

6.4.1 Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. We recommend a lateral load coefficient of 0.4 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half the wall height.

6.4.2 Wall Foundation

For walls less than about 5 feet, the recommendations outlined in Section 6.2 of this report are also appropriate for designing wall foundations. For taller walls, the foundation design requirements should be evaluated individually, and the use of flexible walls such as Mechanically Stabilized Earth (MSE) walls may be more appropriate than cats-in-place concrete walls due its higher tolerance for settlements.

6.4.3 Wall Drainage

Footing drains should be installed at the base of wall footings. As a minimum, 4-inch diameter perforated drainpipes should be installed next to the base of the footings and embedded in 12 to 18 inches of pea or washed gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet and should not be allowed to daylight over the slope.

For site retaining walls, in lieu of conventional footing drains, weepholes may be installed. The weep holes should be at least 1 ½ inch in diameter, spaced no more than 10 feet apart, and located no more than one foot above the ground surface in front of the walls.

6.4.4 Wall Backfill

Retaining wall backfill should consist of free draining granular material. The site soils consist of relatively fine sand with varying amounts of silt. We recommend importing a free draining granular material, such as Seattle Type 17 or a soil meeting the requirements of Gravel Borrow as defined in Section 9-03.14(1) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2024). In areas where space is limited between the wall and the face of excavation, pea gravel may be used as backfill without compaction.

Wall backfill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than 8 to 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557 (Modified Proctor). Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

6.5 PAVEMENT

We anticipate that most of the paved areas will be limited to light passenger vehicles. Pavement for light traffic loads should have a minimum thickness of 2 inches of hot-mixed asphalt over 4 inches of crushed surfacing base course (CSBC) and 12 inches of structural fill placed of properly compacted subgrade. However, a thicker asphalt layer (i.e., 3 inches) would significantly increase the design life and reduce the need for long term maintenance of the pavement.

In areas where the pavement will be subjected to higher loading conditions such as fire trucks and garbage trucks, we recommend a minimum pavement section of 3 inches of hot-mixed asphalt, 4 inches of CSBC and 12 inches of structural fill placed on a properly compacted subgrade.

6.6 PERMANENT CUT AND FILL SLOPES

Permanent cut and fill slopes should be inclined no steeper than 2H:1V (Horizontal:Vertical). Cut slopes should be observed by PanGEO during excavation to verify that conditions are as anticipated. Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

6.7 FILL FOR RAISING GRADE

We understand that, according to comments from the City of Burlington, the west portion of the site grade is lower than the base flood elevation of 30.4-foot NGVD-29, and as much as 2 feet of fill may be needed to raise the site/floor elevation to at least Elevation 31.4 feet (NGVD-29). More than 2 feet of new fill will be needed if it is desire to raise the site grade above Elevation 31.4 feet (NGVD-29).

We recommend that organic soils were stripped removed prior to placing the new fill. Based on the results of our test pits, it appears that stripping depth should be no more than about a foot thick.

Placement of significant amount of fill at the site could lead to settlement of the native soils below the site. However, because of the mostly sandy and non-plastic nature of the site soils, we expect the settlement to occur quickly. If more than 4 feet of fill will be placed at the site to raise the grade, we recommend the fill be placed at least 4 weeks before commencing on other construction activities such as paving, foundation construction, and installation utilities.

7.0 EARTHWORK CONSIDERATIONS

7.1 TEMPORARY EXCAVATION SLOPES

Temporary excavations should be constructed in accordance with Part N of the WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, for temporary excavation up to 8 feet deep, the temporary excavations may be sloped as steep as 1½H:1V (Horizontal:Vertical). For excavation deeper than 8 feet, PanGEO should be consulted for evaluation.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. If seepage is encountered, excavation slope inclinations may need to be reduced. During wet weather, the cut slopes may need to be flattened to reduce potential erosion or should be covered with plastic sheeting.

7.2 STRUCTURAL FILL AND COMPACTION

Soil to be used as structural fill should be free of organic and inorganic debris, be near the optimum moisture content, and be capable of being compacted to the recommendations provided below.

Structural fill should consist of imported granular soils with a maximum dimension of 4 inches, less than 30 percent passing the U.S. Standard No. 40 sieve, and less than 7 percent passing the U.S. Standard No. 200 sieve. The fine-grained portion of structural fill soils should consist of non-plastic material.

Structural fill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than about a foot in thickness and compacted to a dense and unyielding condition. If density testing will be performed on structural fill, the fill should be compacted to at least 95 percent maximum density, determined using ASTM D-1557 (Modified Proctor). The procedure to achieve proper density of a compacted fill depends on the size and type of compaction equipment, the number of passes, thickness of the lifts being compacted, and certain soil properties. If the excavation to be backfilled is constricted and limits the use of heavy equipment, smaller equipment can be used, but the lift thickness will need to be reduced to achieve the required relative compaction.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

7.3 Material Reuse

The native soils underlying the site primarily consist of silty fine to medium sand. These soils are moisture sensitive and will become disturbed and soft when exposed to inclement weather conditions. We do not recommend reusing the native soils as structural fill. If it is planned to use the native soil in non-structural areas, the excavated soil should be stockpiled and protected with plastic sheeting to prevent it from becoming saturated by precipitation or runoff.

7.4 Wet Weather Construction

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.75-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be installed at strategic locations around the site to control erosion and the movement of soil.

- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

7.5 EROSION CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area leaving the immediate work site. Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

Permanent control of surface water should be incorporated into the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is collected and directed away from the structure to a suitable outlet. Potential issues associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

8.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed development, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. The City, as part of the permitting process, will also require geotechnical construction inspection services. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

9.0 CLOSURE

We have prepared this report for Quarterra and the project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of services.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our services specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

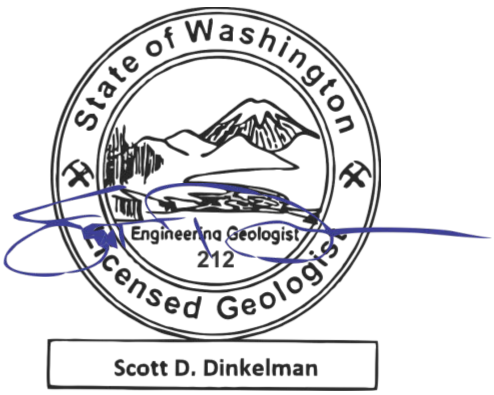
This report has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use of this report.

Sincerely,

PanGEO, Inc.



Scott D. Dinkelman, LEG, LHG
Principal Engineering Geologist

Siew L Tan, P.E.
Principal Geotechnical Engineer

10.0 REFERENCES

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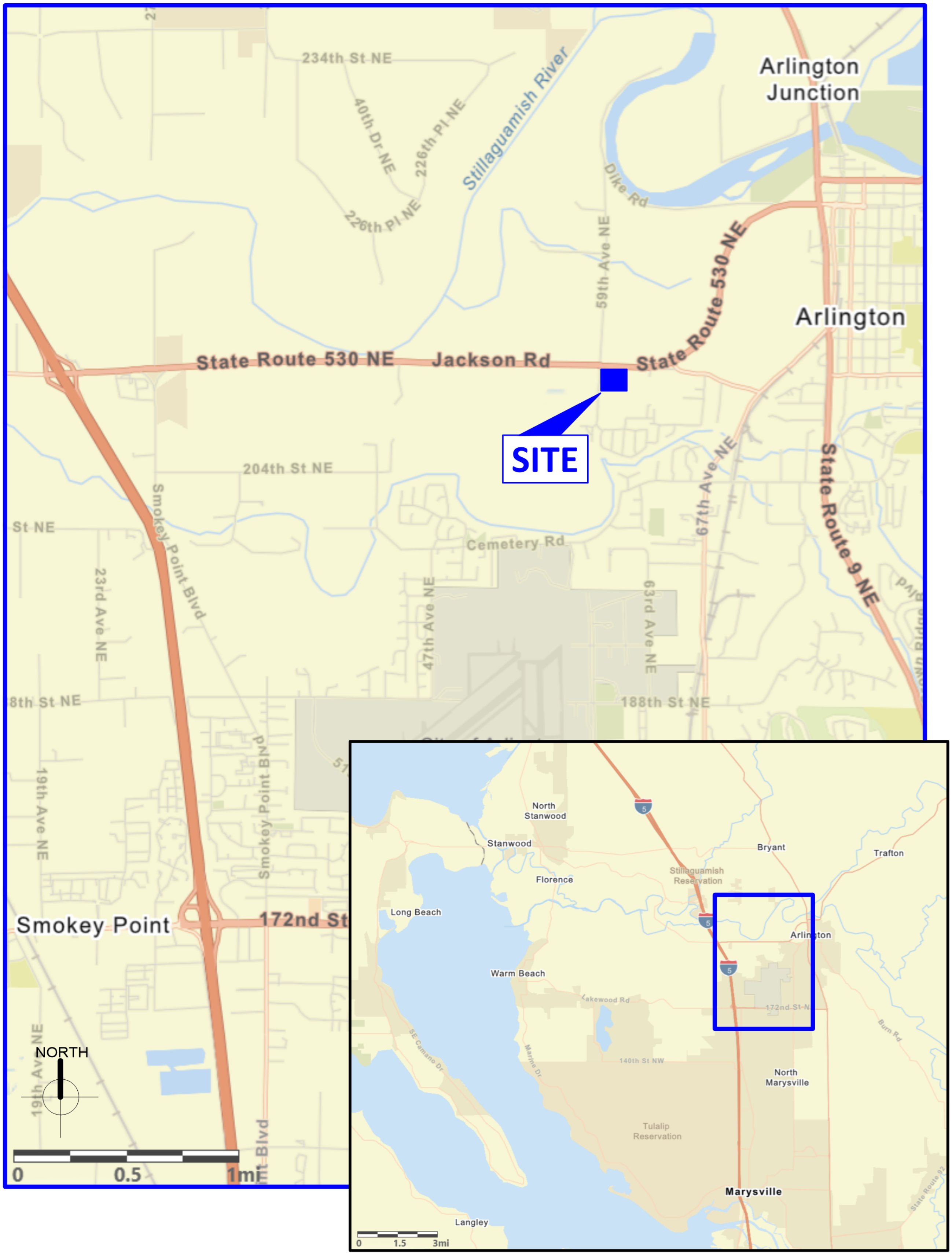
International Code Council (IBC), 2018. *International building code 2018*. Country Club Hills, IL: International Code Council, Inc.

International Code Council (IBC), 2021. *International building code 2021*. Country Club Hills, IL: International Code Council, Inc.

WSDOT, 2024, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10.*



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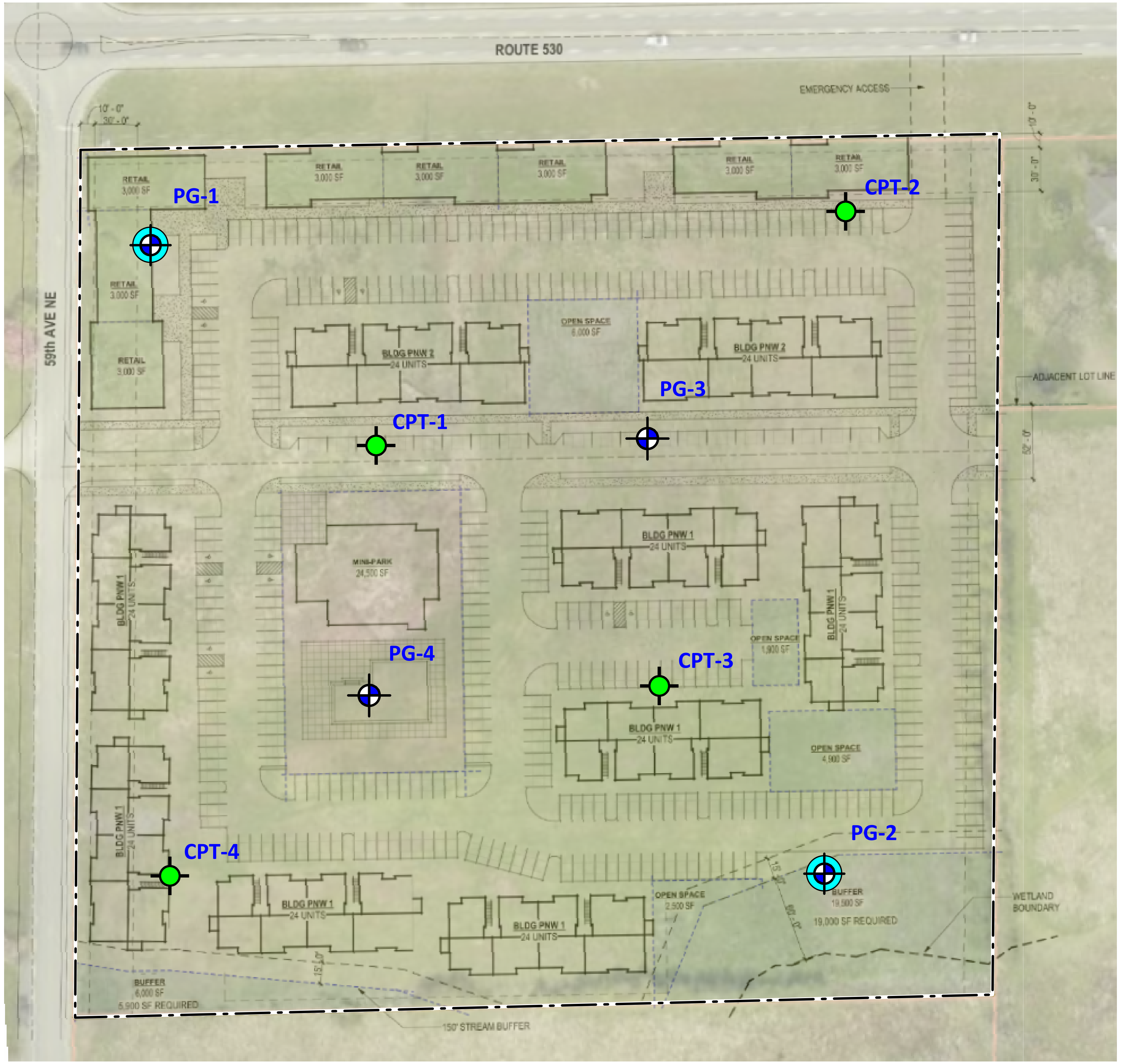
21117 59th Avenue Northeast Arlington, WA

VICINITY MAP

Project No.	Figure No.
24-207	1



LEGEND:

- Subject Site
- Approximate Boring Location
- Approximate Boring Location with Monitoring Well, PanGEO, Inc.
- Cone Penetrometer Test Location

N

Approx. Scale (feet)

0 40 80

Note:
Base map modified from GIS map obtained from Quarterra's Prelimary Site Plan for the project, provided to our representative on 6/26/2024.

PanGEO INCORPORATED

Arlington Apartments
21117 59th Ave NE
Arlington, WA

SITE AND EXPLORATION PLAN

Project No.	Figure No.
24-207	2

APPENDIX A

LOGS OF TEST BORINGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:
1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS
for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS
Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

LOG KEY 09-118 LOG.GPJ PANGEO.GDT 11/12/13

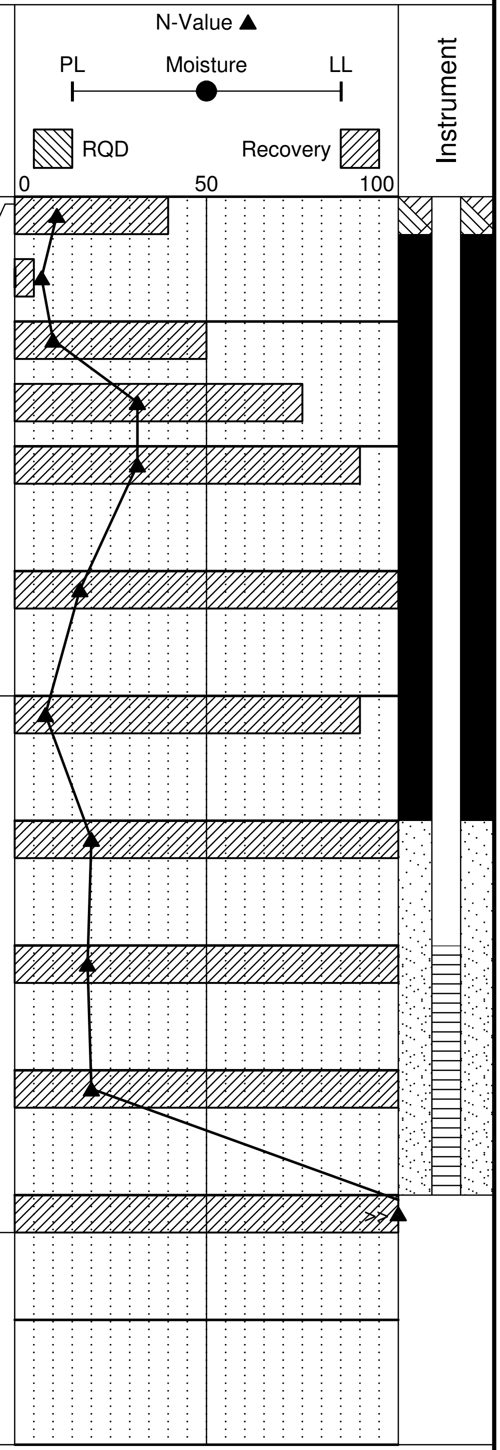
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	21117 59th Avenue Northeast	Surface Elevation:	64.0ft
Job Number:	24-207	Top of Casing Elev.:	64.0ft
Location:	21117 59th Ave NE, Arlington, WA	Drilling Method:	EC55 Track Drill Rig, Hollow Stem Auger
Coordinates:	Northing: 48.1879372, Easting: -122.1504602	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S1		7 6 5			Loose, dark brown silty fine sand with organics -- Topsoil.
	S2		4 3 4	GS		**ALLUVIUM (Qyal)** Loose to medium dense, tan to brown, poorly graded SAND with silt, scattered organics; moist to saturated, stratified soil structure, alternating layers of fine- and coarse-grained materials.
5.0	S3		3 4 6			
	S4		15 16 16			-- No organics present below 3 feet.
10.0	S5		7 16 16			
						-- Audible grinding on gravels at approximately 12- and 17-feet below the existing ground surface.
15.0	S6		6 6 11			-- Heaving sands encountered from 15-feet below the existing ground surface to boring termination depth.
20.0	S7		3 3 5			**MARYSVILLE SAND MEMBER (Qvrm)** Loose to dense, gray to tan, very fine to coarse SAND with trace gravel and silt; wet to saturated, interbedded coarse sand, thin lenses of silty sand, overall massive soil structure.
25.0	S8		7 9 11			
30.0	S9		6 8 11			
35.0	S10		5 7 13			-- Strongly heaving sands encountered below 35-feet; N-value at 40 feet likely inflated due to sand heave.
40.0	S11		11 50/3			
45.0						Boring terminated at 41.5 feet below existing grade. Groundwater was encountered at approximately 7 feet below existing grade at the time of drilling, and was measured at 5.9 feet below grade in the installed monitoring well following the conclusion of drilling. Ecology Well ID BPG-954.
50.0						



Completion Depth:	41.5ft
Date Borehole Started:	6/27/24
Date Borehole Completed:	6/27/24
Logged By:	E. Eckles
Drilling Company:	Boretec1 Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Approximate state plane coordinates and elevation are based on Google Earth. This information is provided for relative information only and is not a substitution for field survey. **Datum: Washington North / NAVD88**

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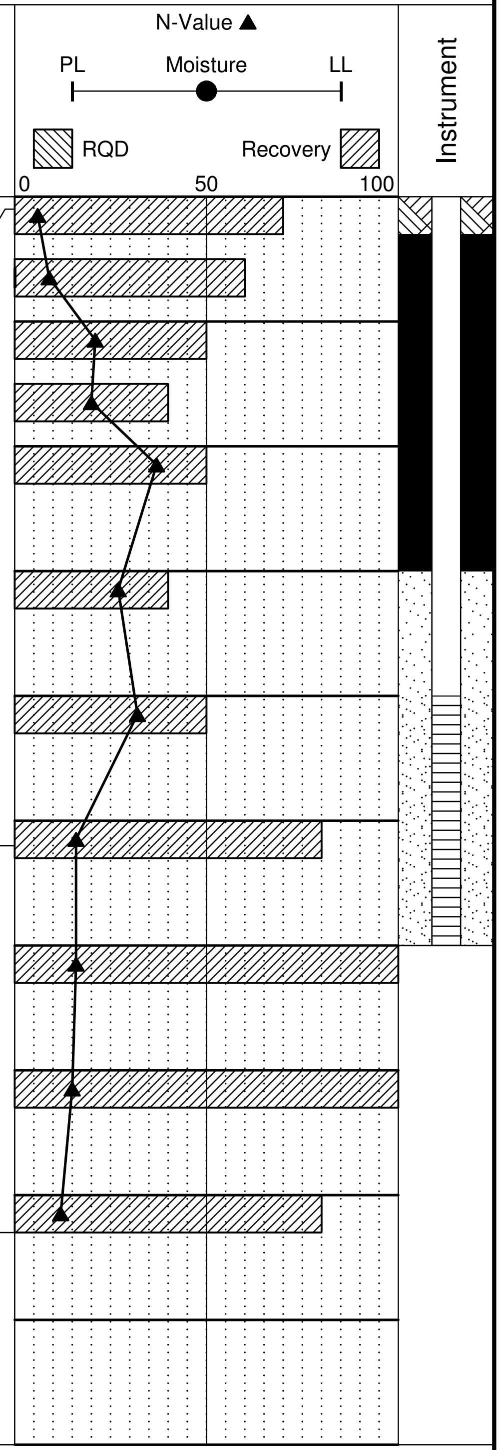
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	21117 59th Avenue Northeast	Surface Elevation:	62.0ft
Job Number:	24-207	Top of Casing Elev.:	62.0ft
Location:	21117 59th Ave NE, Arlington, WA	Drilling Method:	EC55 Track Drill Rig, Hollow Stem Auger
Coordinates:	Northing: 48.18686, Easting: -122.1487876	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	MATERIAL DESCRIPTION
0.0	S1		2, 3, 3		Loose, dark brown silty fine sand with organics -- Topsoil.
	S2		2, 4, 5	GS	**ALLUVIUM (Qyal)** Loose to medium dense, tan to brown, poorly graded fine to coarse SAND with silt and scattered organics; moist to saturated, stratified soil structure, alternating layers of fine- and coarse-grained materials.
5.0	S3		7, 11, 10		
	S4		5, 9, 11		-- No organics present below 4 feet.
10.0	S5		16, 17, 20		-- Audible grinding on gravels at approximately 11-feet below the existing ground surface.
15.0	S6		10, 13, 14		
20.0	S7		10, 15, 17		-- Heaving sands encountered from 20-feet below the existing ground surface to boring termination depth.
25.0	S8		5, 7, 9		**MARYSVILLE SAND MEMBER (Qvrm)** Medium dense to dense, gray to tan, very fine to coarse SAND with trace gravel and silt; wet to saturated, interbedded coarse sand, thin lenses of silty sand, overall massive soil structure.
30.0	S9		5, 7, 9		
35.0	S10		5, 7, 8		
40.0	S11		3, 7, 5		Boring terminated at 41.5 feet below existing grade. Groundwater was encountered at approximately 4 feet below existing grade at the time of drilling, and was measured at 3.7 feet below grade in the installed monitoring well following the conclusion of drilling. Ecology Well ID BPG-955.
45.0					
50.0					



Completion Depth:	41.5ft
Date Borehole Started:	6/27/24
Date Borehole Completed:	6/27/24
Logged By:	E. Eckles
Drilling Company:	Boretec1 Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Approximate state plane coordinates and elevation are based on Google Earth. This information is provided for relative information only and is not a substitution for field survey. **Datum: Washington North / NAVD88**

PanGEO INCORPORATED

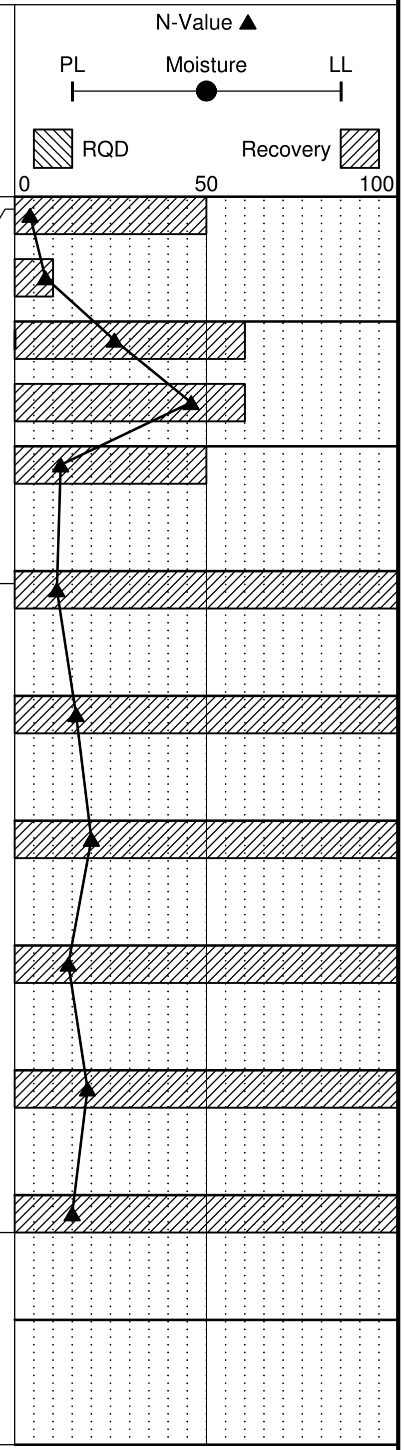
LOG OF TEST BORING PG-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	21117 59th Avenue Northeast	Surface Elevation:	65.0ft
Job Number:	24-207	Top of Casing Elev.:	
Location:	21117 59th Ave NE, Arlington, WA	Drilling Method:	EC55 Track Drill Rig, Hollow Stem Auger
Coordinates:	Northing: 48.1874709, Easting: -122.1491178	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	MATERIAL DESCRIPTION
0.0	S1	X	2 2 2		Loose, dark brown silty fine sand with organics -- Topsoil.
	S2	X	1 3 5		**ALLUVIUM (Qyal)** Loose to medium dense, tan to brown, poorly graded fine to coarse SAND with silt and gravel; moist to saturated, stratified soil structure, alternating layers of fine- and coarse-grained materials.
5.0	S3	X	6 12 14	GS	-- Scattered organics. -- No organics present below 4 feet.
	S4	X	20 27 19		-- Gravel in tip of split spoon sampler at the 7.5-foot interval; N-value at 7.5-feet inflated by pushing gravel.
10.0	S5	X	5 6 6		-- Audible grinding on gravels at approximately 15-feet below the existing ground surface.
15.0	S6	X	4 5 6		**MARYSVILLE SAND MEMBER (Qvrm)** Medium dense to dense, gray to tan, very fine to coarse SAND with trace gravel and silt; wet to saturated, thinly interbedded lenses of silty sand, overall massive soil structure.
20.0	S7	X	4 6 10		-- Heaving sands encountered from 20-feet below the existing ground surface to boring termination depth.
25.0	S8	X	5 8 12		
30.0	S9	X	5 6 8		
35.0	S10	X	6 8 11		
40.0	S11	X	5 6 9		Boring terminated at 41.5 feet below existing grade. Groundwater was encountered at approximately 5 feet below grade at the time of drilling.
45.0					
50.0					



Completion Depth:	41.5ft
Date Borehole Started:	6/28/24
Date Borehole Completed:	6/28/24
Logged By:	E. Eckles
Drilling Company:	Boretec1 Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Approximate state plane coordinates and elevation are based on Google Earth. This information is provided for relative information only and is not a substitution for field survey. **Datum: Washington North / NAVD88**

PanGEO INCORPORATED

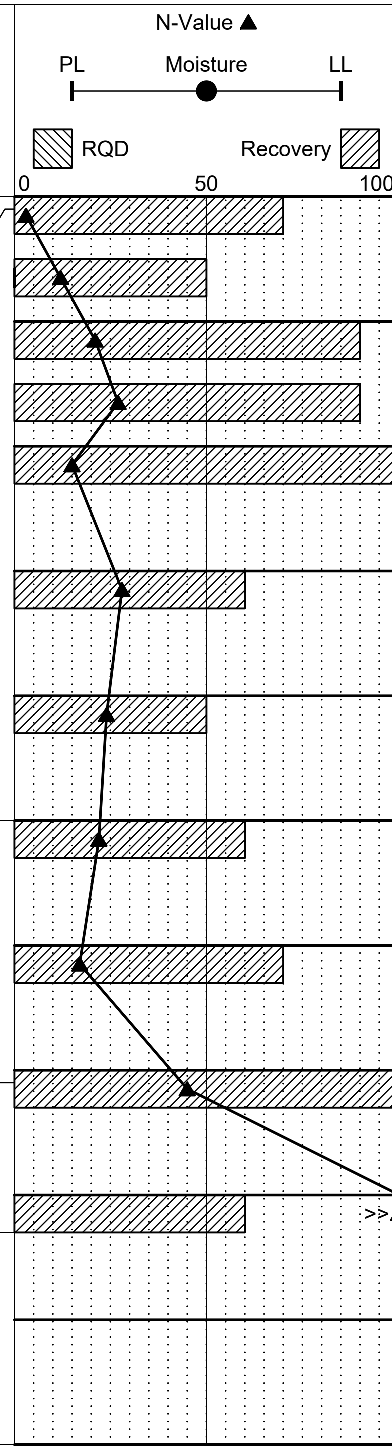
LOG OF TEST BORING PG-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	21117 59th Avenue Northeast	Surface Elevation:	63.0ft
Job Number:	24-207	Top of Casing Elev.:	
Location:	21117 59th Ave NE, Arlington, WA	Drilling Method:	EC55 Track Drill Rig, Hollow Stem Auger
Coordinates:	Northing: 48.1871629, Easting: -122.1499584	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S1	⊠	1 2 1			Loose, dark brown silty fine sand with organics -- Topsoil.
	S2	⊠	2 5 7	GS		**ALLUVIUM (Qyal)** Very loose to medium dense, tan to brown, silty fine to coarse SAND with scattered organics; moist to saturated, stratified soil structure, alternating layers of fine- and coarse-grained materials.
5.0	S3	⊠	7 10 11			-- No organics present below 3 feet.
	S4	⊠	14 12 15			
10.0	S5	⊠	6 7 8			-- Audible grinding on gravels at approximately 6-, 9-, 13-, and 17-feet below the existing ground surface.
15.0	S6	⊠	10 11 17			-- Heaving sands encountered from 15-feet below the existing ground surface to boring termination depth.
20.0	S7	⊠	6 12 12			
25.0	S8	⊠	9 11 11			**MARYSVILLE SAND MEMBER (Qvrm)** Medium dense to dense, gray to tan, very fine to coarse SAND with trace gravel and silt; wet to saturated, thinly interbedded lenses of silty sand, overall massive soil structure.
30.0	S9	⊠	6 8 9			
35.0	S10	⊠	20 26 19			**VASHON TILL (Qvt)** Dense to very dense, gray to blue-gray, silty very fine to medium SAND with gravel; moist, diamict soil structure.
40.0	S11	⊠	36 50/5			
45.0						Boring terminated at 41.5 feet below grade. Groundwater was encountered at approximately 3 feet below grade at the time of drilling.
50.0						



Completion Depth:	41.5ft
Date Borehole Started:	6/28/24
Date Borehole Completed:	6/28/24
Logged By:	E. Eckles
Drilling Company:	Boretec1 Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Approximate state plane coordinates and elevation are based on Google Earth. This information is provided for relative information only and is not a substitution for field survey. **Datum: Washington North / NAVD88**

PanGEO INCORPORATED

LOG OF TEST BORING PG-4

Figure A-5

The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX B

CONE PENETROMETER TESTS

PRESENTATION OF SITE INVESTIGATION RESULTS

Arlington Parcel

Prepared for:

PanGEO

ConeTec Job No: **24-59-27779**

Project Start Date:	2024-06-28
Project End Date:	2024-06-28
Release Date:	2024-07-03

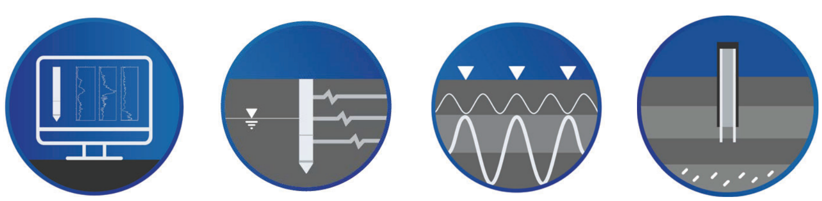
Report Prepared by:

ConeTec, Inc.

1237 S Director Street, Seattle, WA 98108

Tel: (253) 397-4861

ConeTecWA@conetec.com
www.conetec.com
www.conetecdataservices.com



ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. for PanGEO.

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. Please refer to the list of attached documents following the text of this report. A site map, test summaries, and test plots are all included in the body of the report.

Project	
Client	PanGEO
Project	Arlington Parcel
ConeTec Project Number	24-59-27779
Test Types	CPTu/SCPTu
Additional Comments	None

Contents

The following listed below are included in the body of this report:

- **Site Map**
- **Limitations and Closure**
- **Project Information**
- **Report Appendices**
- **Supporting Documents and Materials**

SITE MAP

All locations are approximate unless otherwise stated in the body of the report.

ConeTec Job Number: 24-59-27779

Client: PanGEO

Project: Arlington Parcel

Date: 2024-07-03

CONETEC

LIMITATIONS

3rd Party Disclaimer

The “Report” refers to this report titled: Arlington Parcel

The Report was prepared by ConeTec for: PanGEO

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: PanGEO

The “Report” refers to this report titled: Arlington Parcel

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.

Closure

Thank you for the opportunity to work on this project. The equipment used as well the field procedures followed, all complied with current accepted best practice standards.

Report prepared by: Ashlan Joyce
Jesse Martinez

PROJECT INFORMATION

Rig		
Description	**Deployment System**	**Test Type**
C02-023 CPT Truck Rig	Twin mounted cylinders	CPTu/SCPTu

Coordinates		
Test Type	**Collection Method**	**EPSG Number**
CPTu/SCPTu	Consumer Grade GPS	4326 (WGS84 / LatLong)

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC855:T1500F15U35	855	15	225	1500	15	35

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.
Additional Comments	None

Calculated Geotechnical Parameters	
Additional information	The Normalized Soil Behavior 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 behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

REPORT APPENDICES

The appendices listed below are included in the report:

- **Cone Penetration Test (CPTu) Summary and Standard CPTu Plots**
- **Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic**
- **Normalized Cone Penetration Test Plots**
- **Soil Behavior Type (SBT) Scatter Plots**
- **Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots**
- **Seismic Cone Penetration Test (SCPTu) Tabular Results**
- **SCPTu Test Plots**
- **SCPTu Velocity Wave Traces**
- **Supplementary Documents and Materials**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No: 24-59-27779
Client: PanGEO
Project: Arlington Parcel
Start Date: 2024-06-28
End Date: 2024-06-28

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Seismic Intervals	Latitude[2]	Longitude[2]	Refer to Notation Number
CPT-01	24-59-27779_SP01	2024-06-28	855:T1500F15U35	15	6.4	53.48	17	48.18761	-122.14984	
CPT-02	24-59-27779_CP02	2024-06-28	855:T1500F15U35	15	4.6	51.51		48.18795	-122.14851	
CPT-03	24-59-27779_CP03	2024-06-28	855:T1500F15U35	15	6.4	49.46		48.18714	-122.14897	
CPT-04	24-59-27779_CP04	2024-06-28	855:T1500F15U35	15	6.4	34.04		48.18680	-122.15056	
CPT-04B	24-59-27779_CP04B	2024-06-28	855:T1500F15U35	15	6.4	12.63		48.18676	-122.15055	3
Totals	5 Soundings					201.11 ft	17			

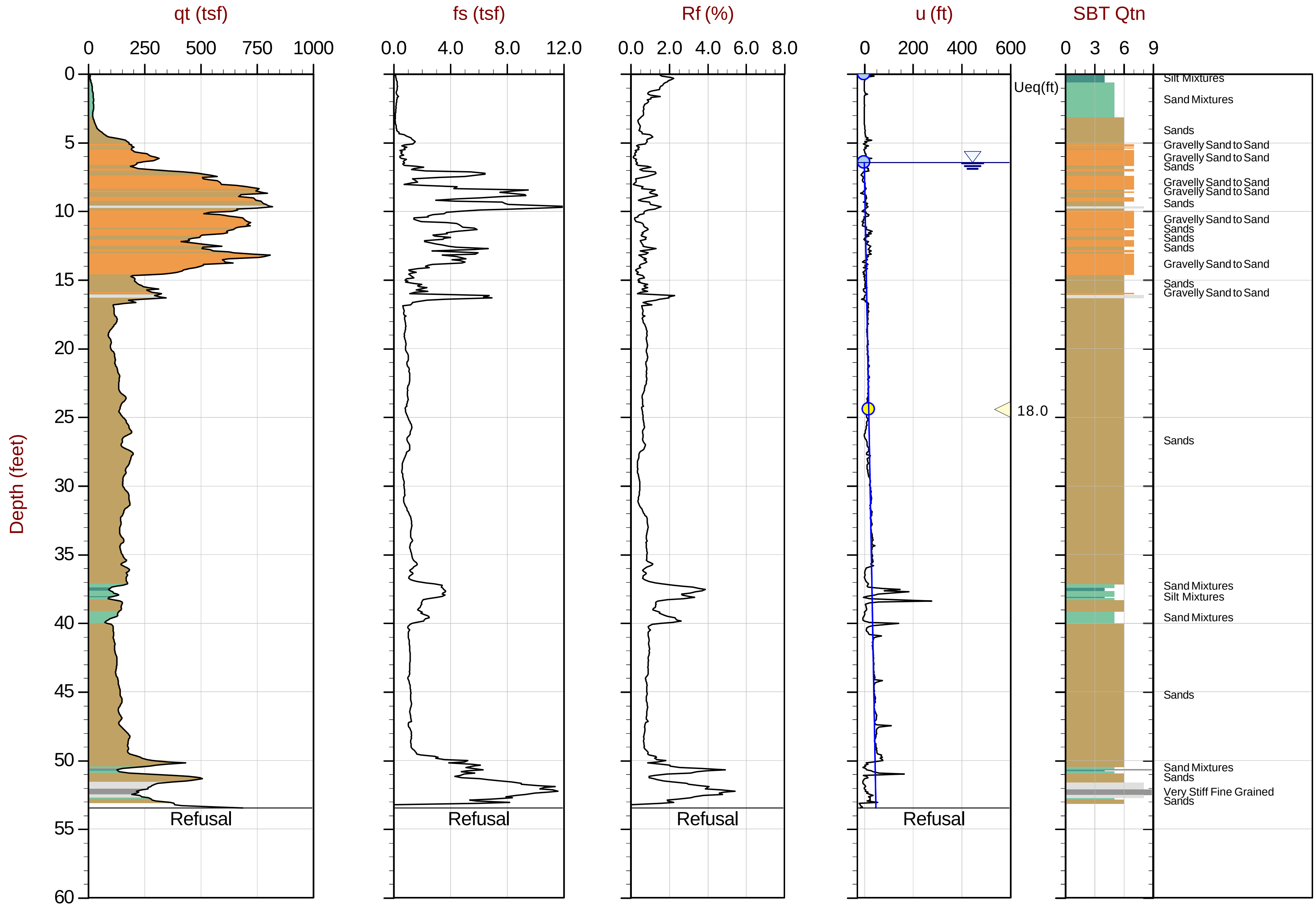
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).
3. The assumed phreatic surface is based off of the nearest pore pressure dissipation test that reached equilibrium.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel

Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 16.300 m / 53.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18761 Long: -122.14984

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

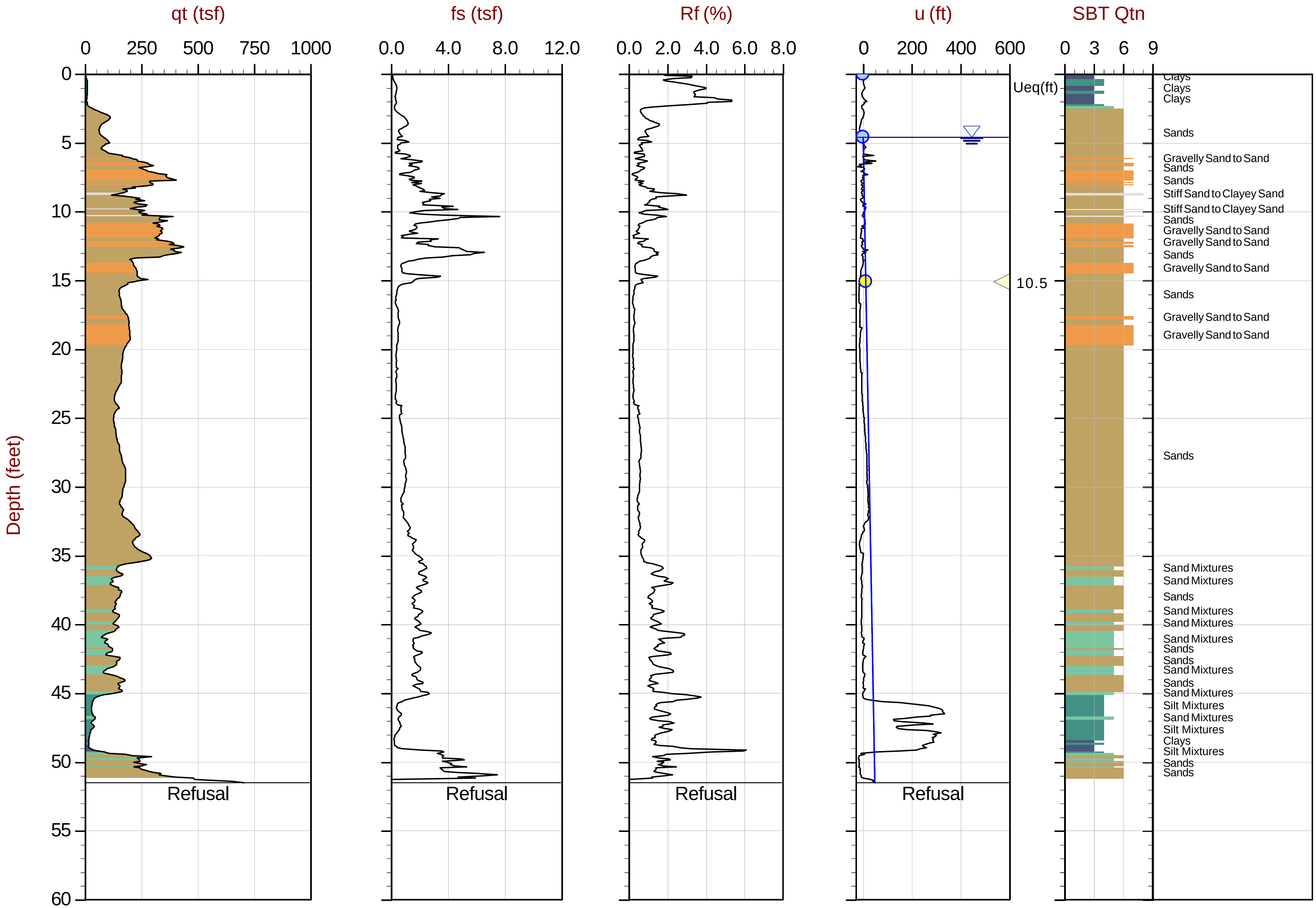
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 09:37
Site: Arlington Parcel

Sounding: CPT-02
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.700 m / 51.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18795 Long: -122.14851

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

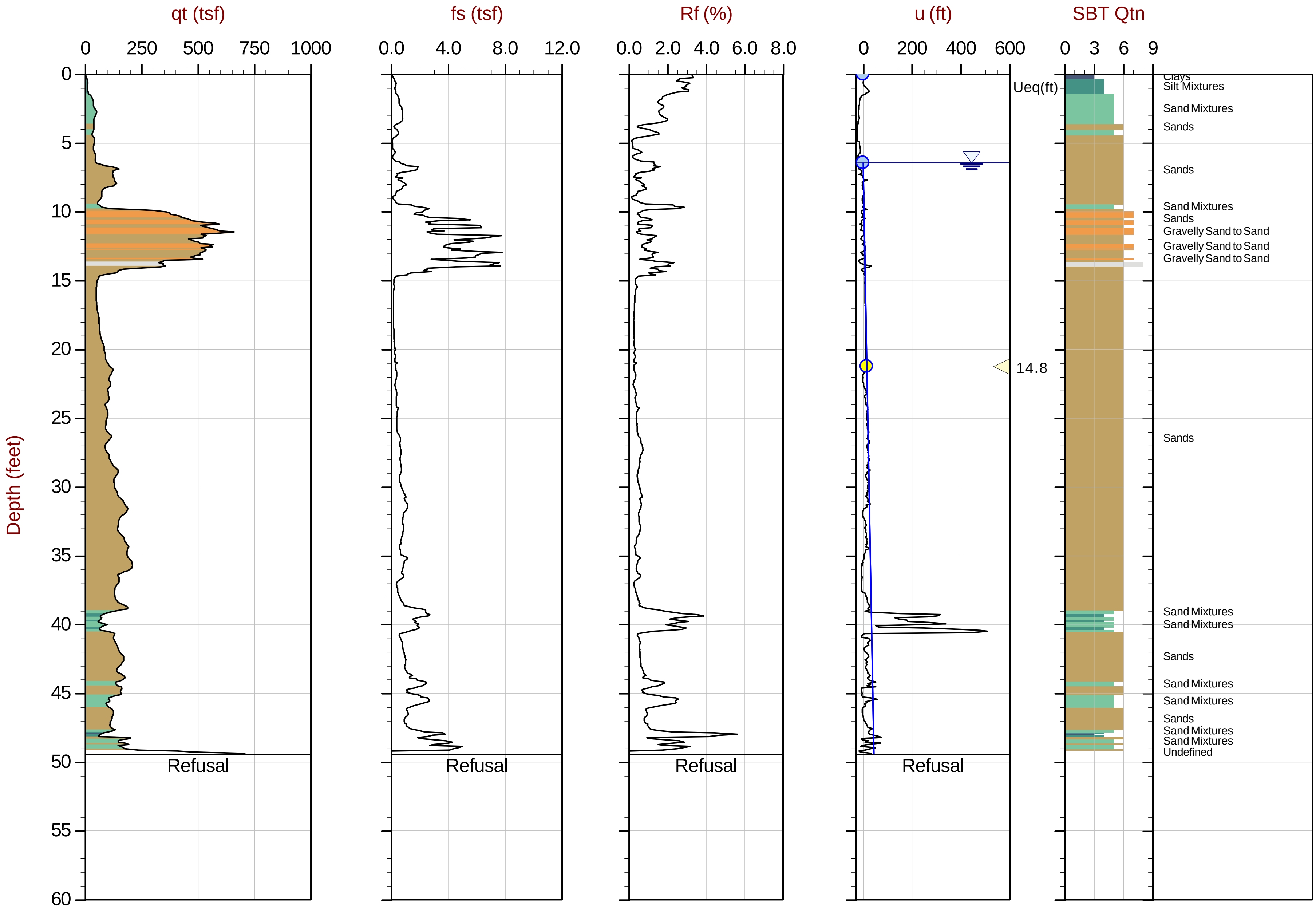
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 10:46
Site: Arlington Parcel

Sounding: CPT-03
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18714 Long: -122.14897

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

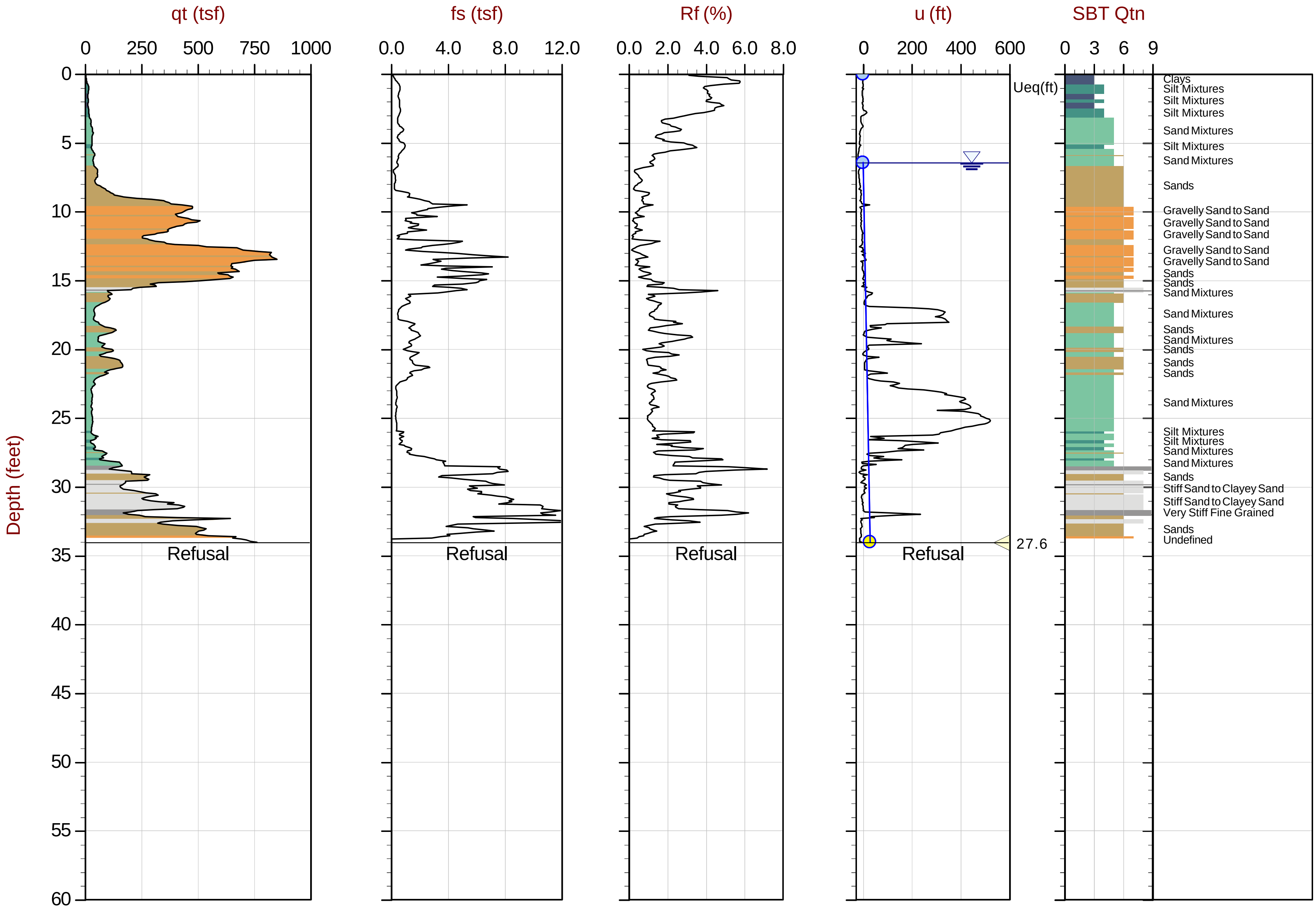
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 12:03
Site: Arlington Parcel

Sounding: CPT-04
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 10.375 m / 34.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18680 Long: -122.15056

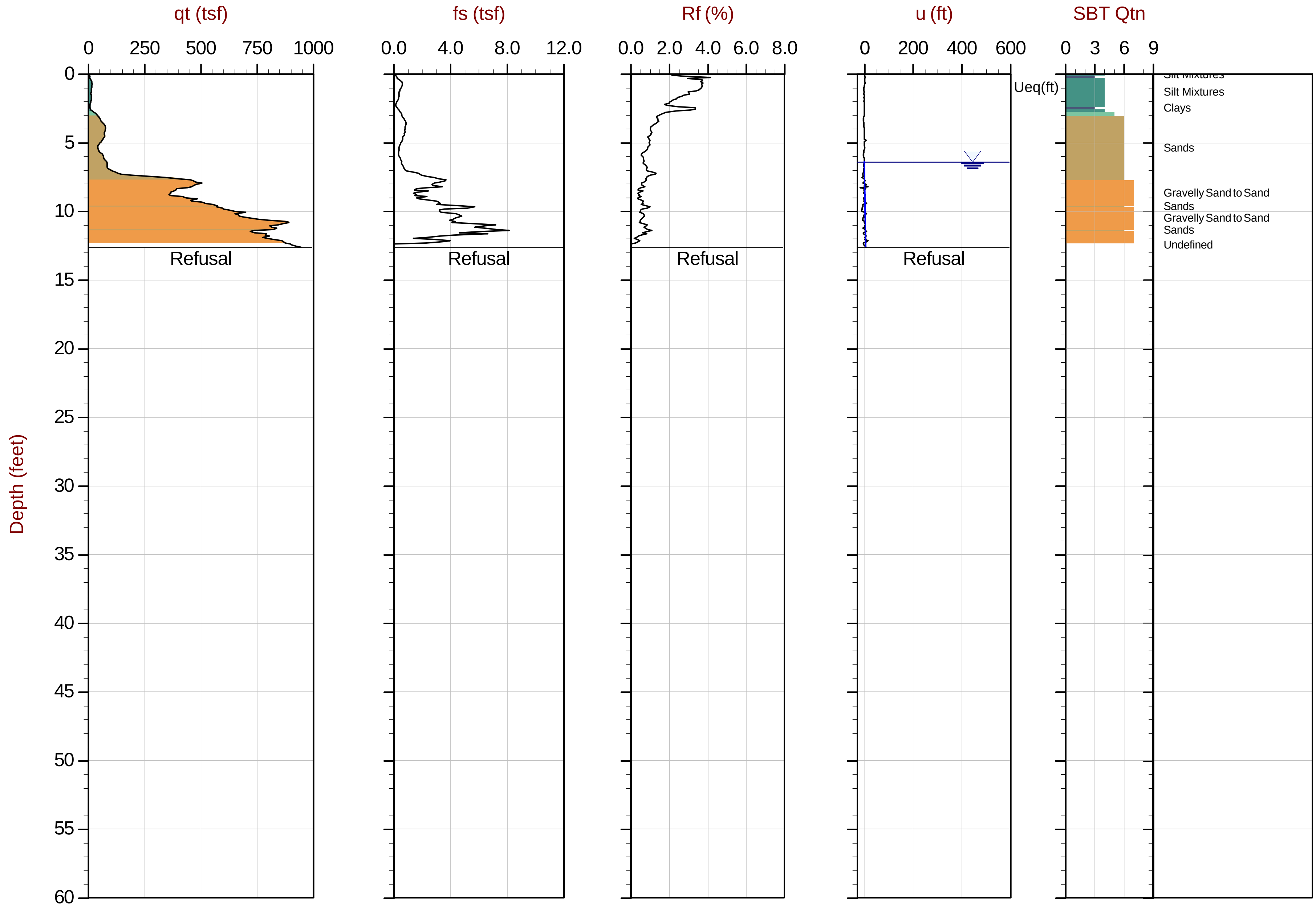
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 13:34
Site: Arlington Parcel

Sounding: CPT-04B
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 3.850 m / 12.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18676 Long: -122.15055

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.

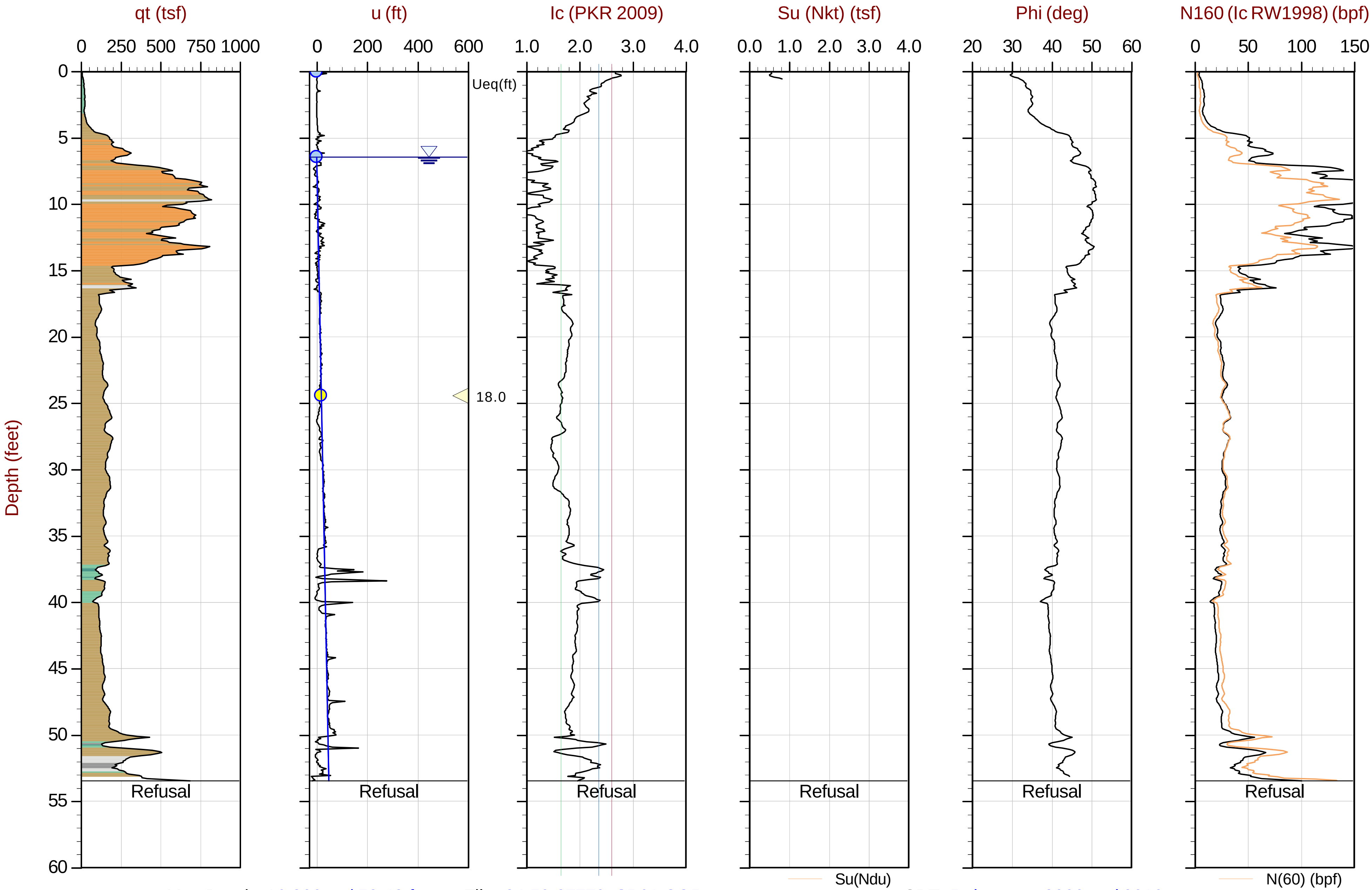
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel

Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 16.300 m / 53.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18761 Long: -122.14984

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

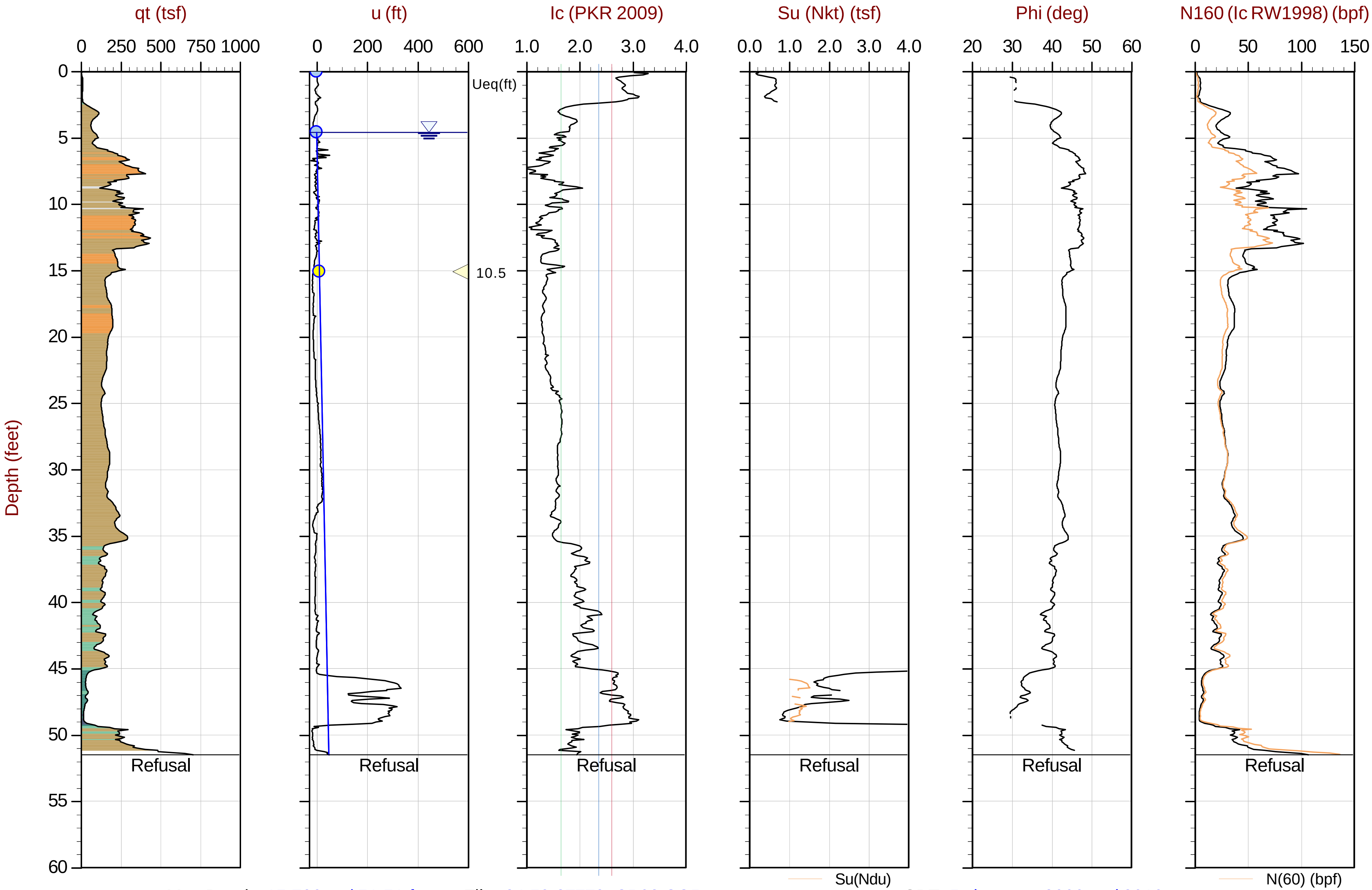
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 09:37
Site: Arlington Parcel

Sounding: CPT-02
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.700 m / 51.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18795 Long: -122.14851

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

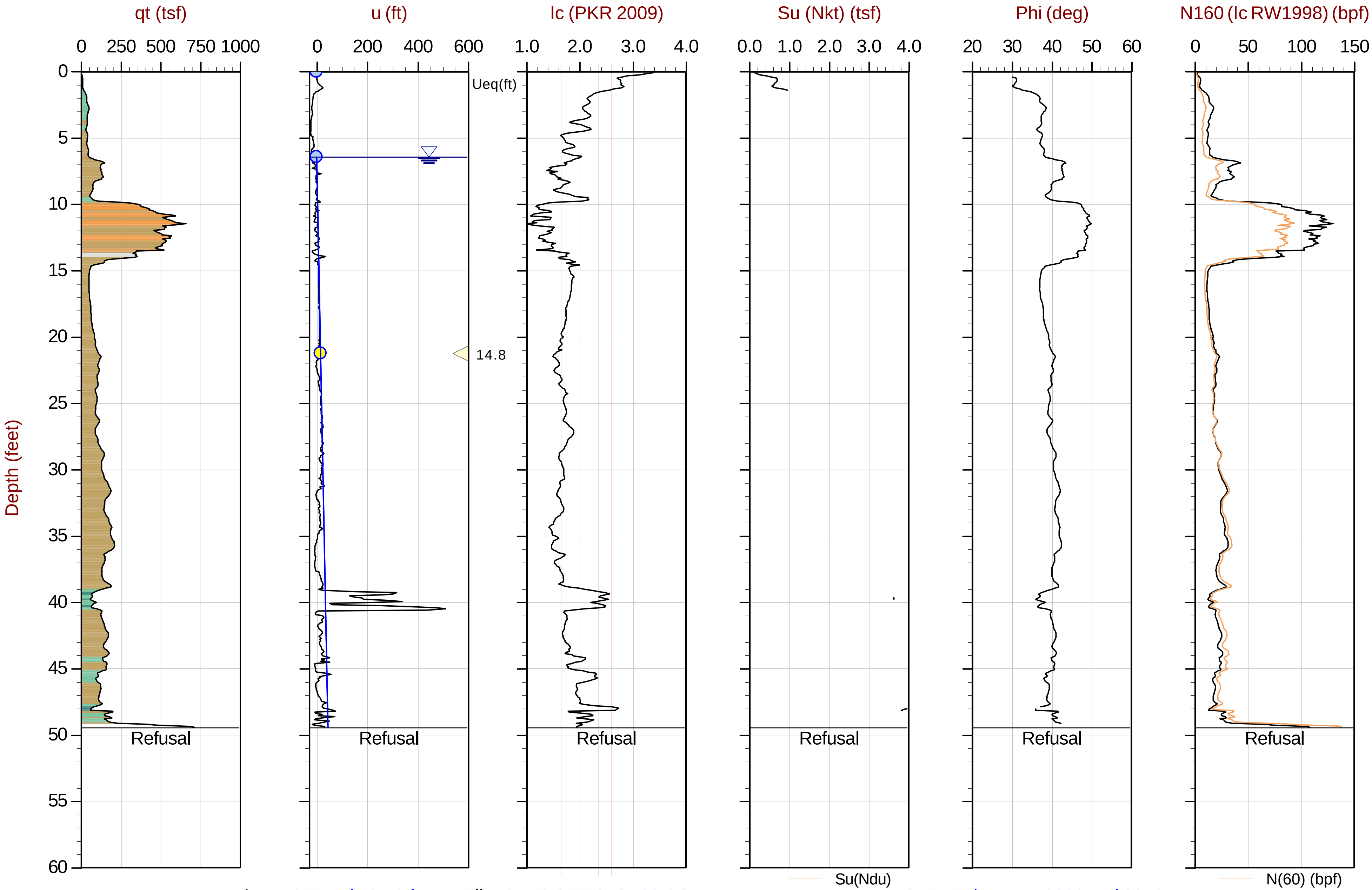
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 10:46
Site: Arlington Parcel

Sounding: CPT-03
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18714 Long: -122.14897

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

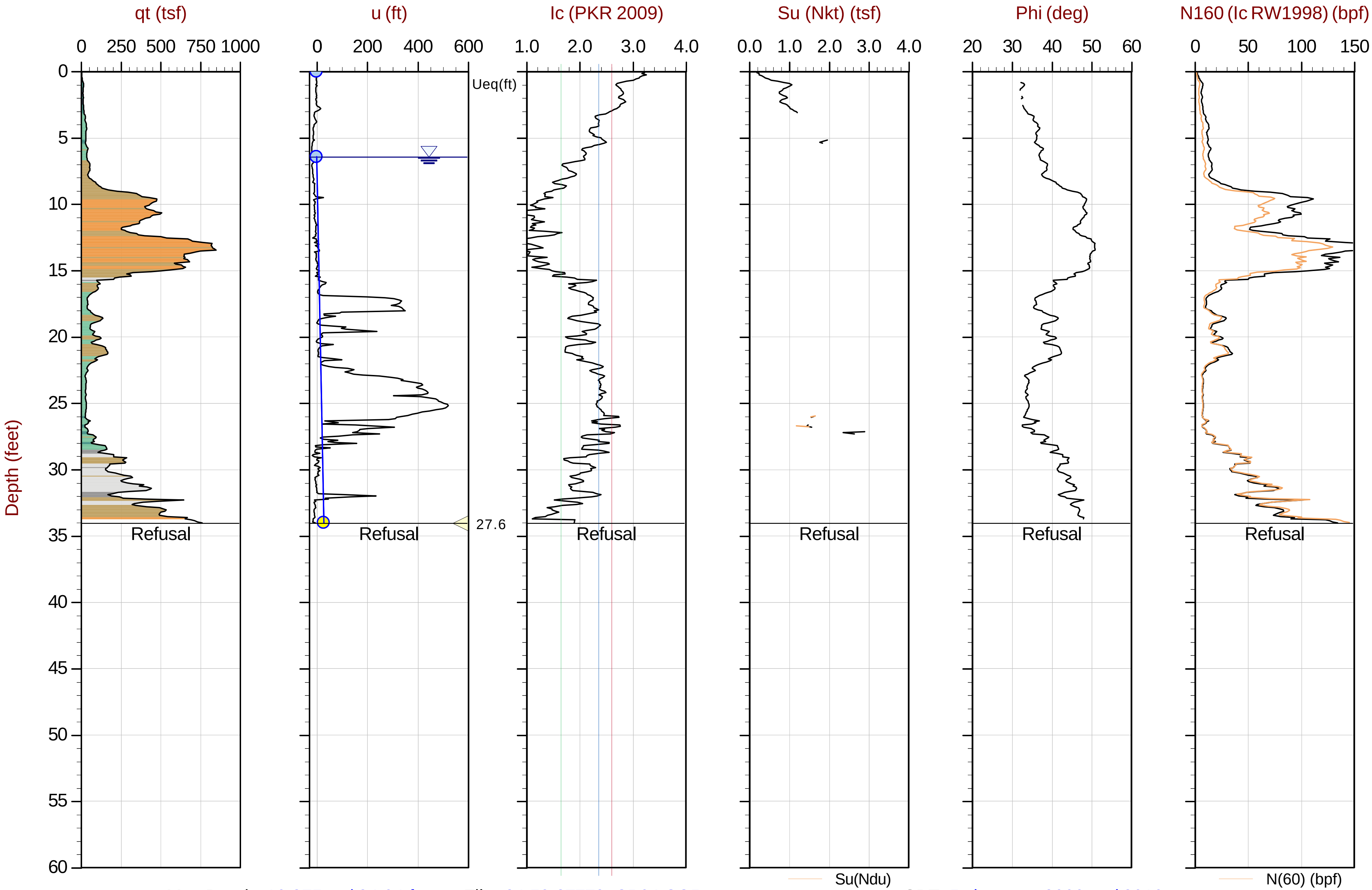
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 12:03
Site: Arlington Parcel

Sounding: CPT-04
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 10.375 m / 34.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18680 Long: -122.15056

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

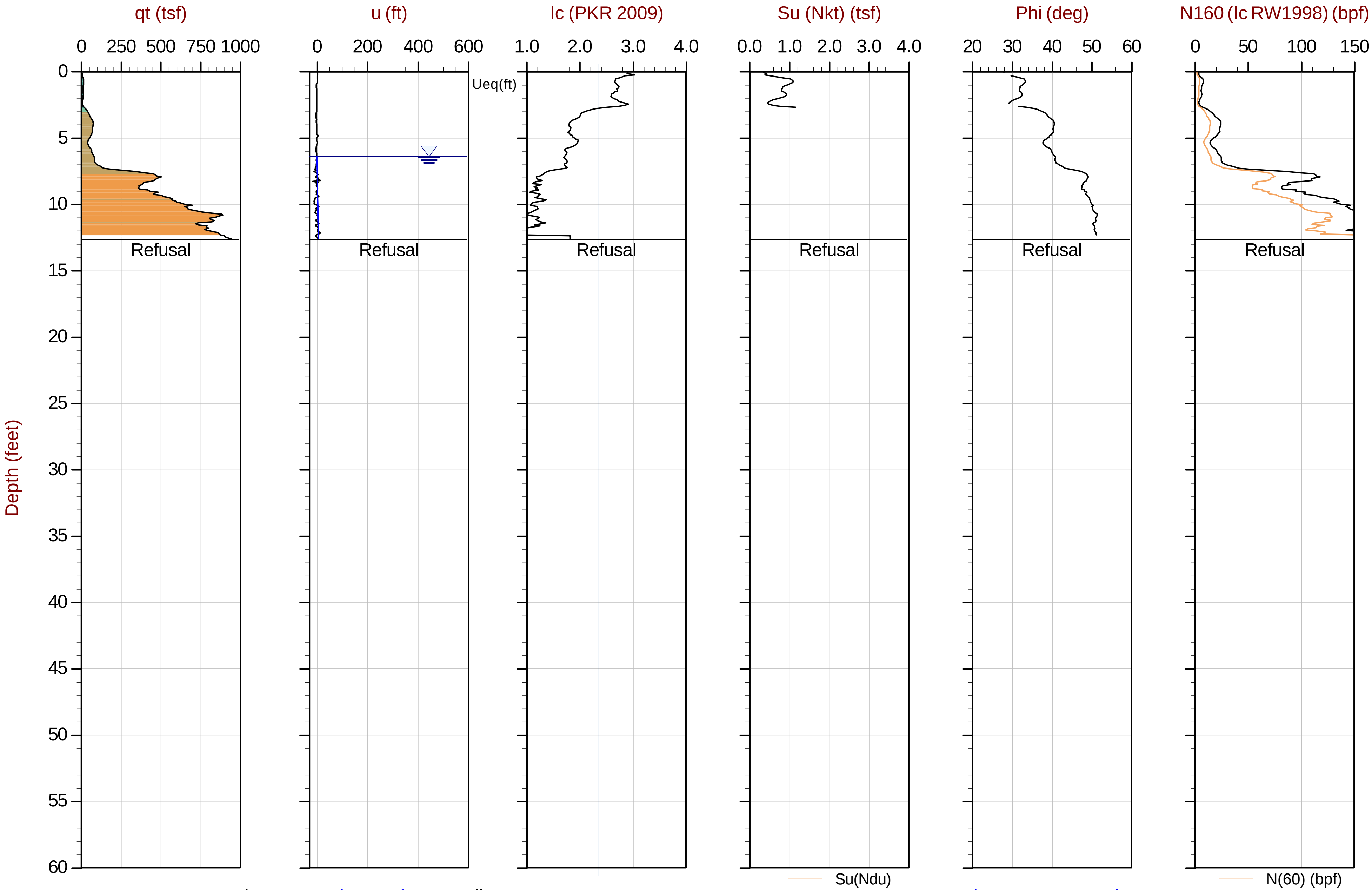
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 13:34
Site: Arlington Parcel

Sounding: CPT-04B
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 3.850 m / 12.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18676 Long: -122.15055

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

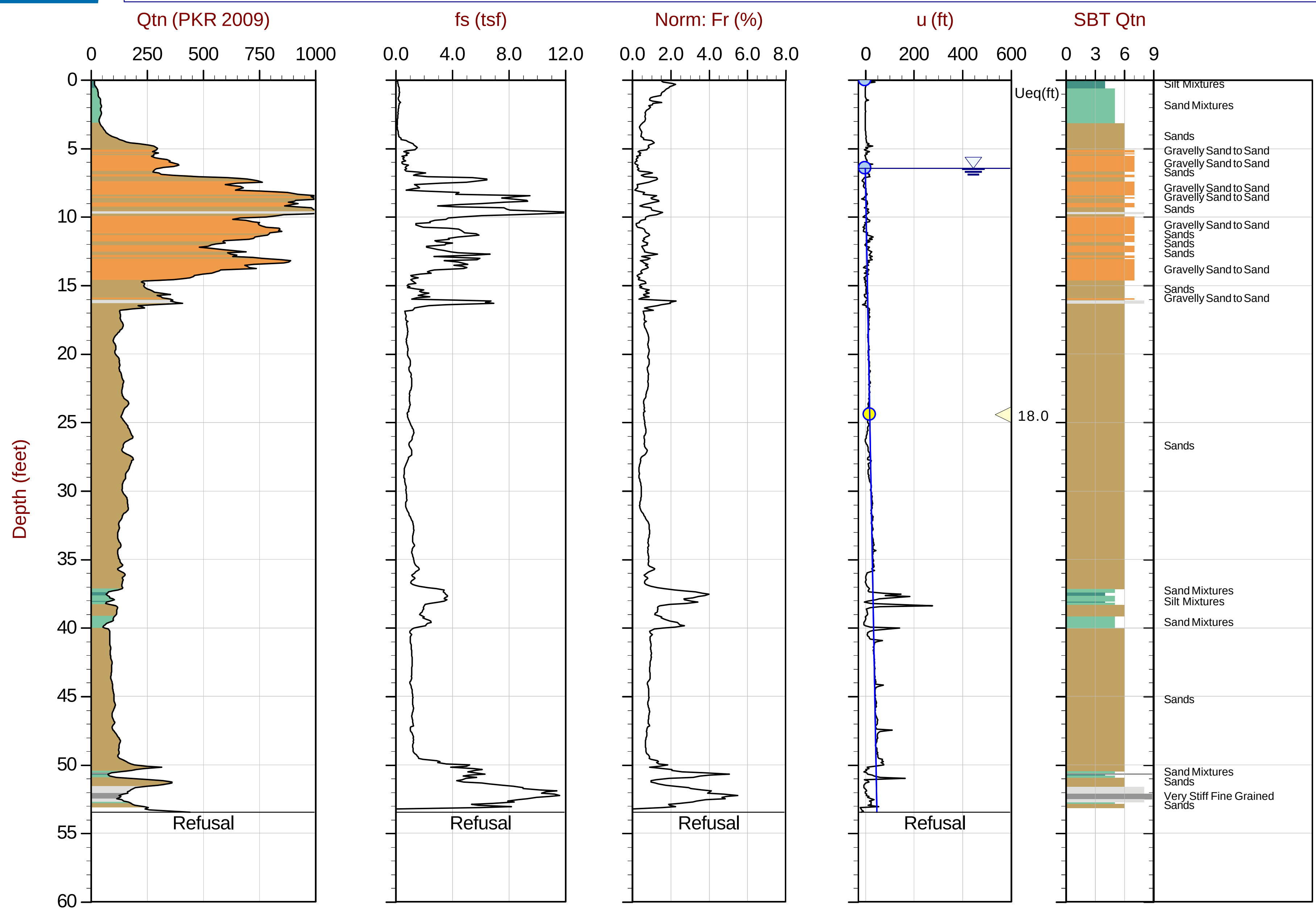
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Normalized Cone Penetration Test Plots

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel

Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 16.300 m / 53.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18761 Long: -122.14984

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

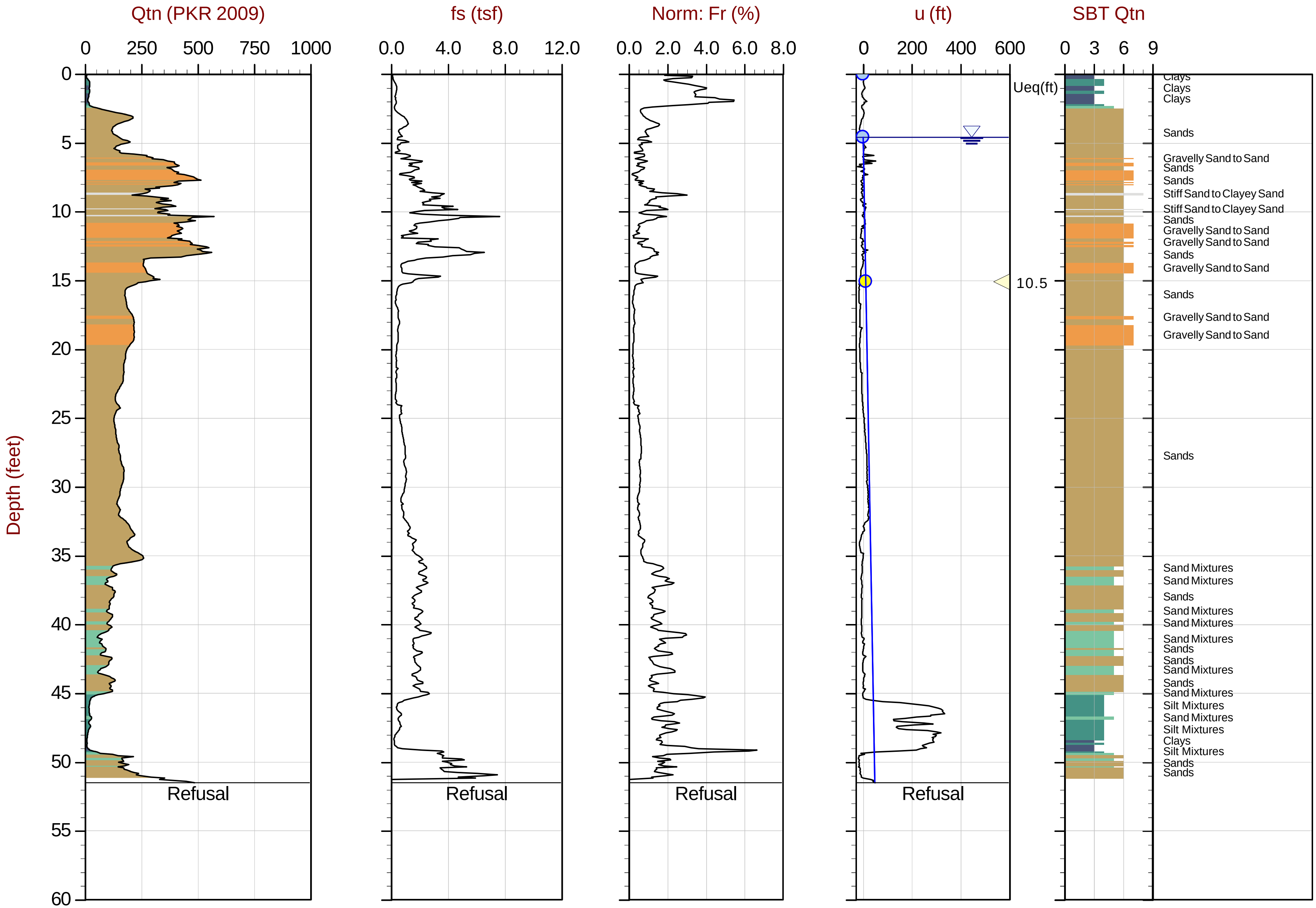
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 09:37
Site: Arlington Parcel

Sounding: CPT-02
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.700 m / 51.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18795 Long: -122.14851

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

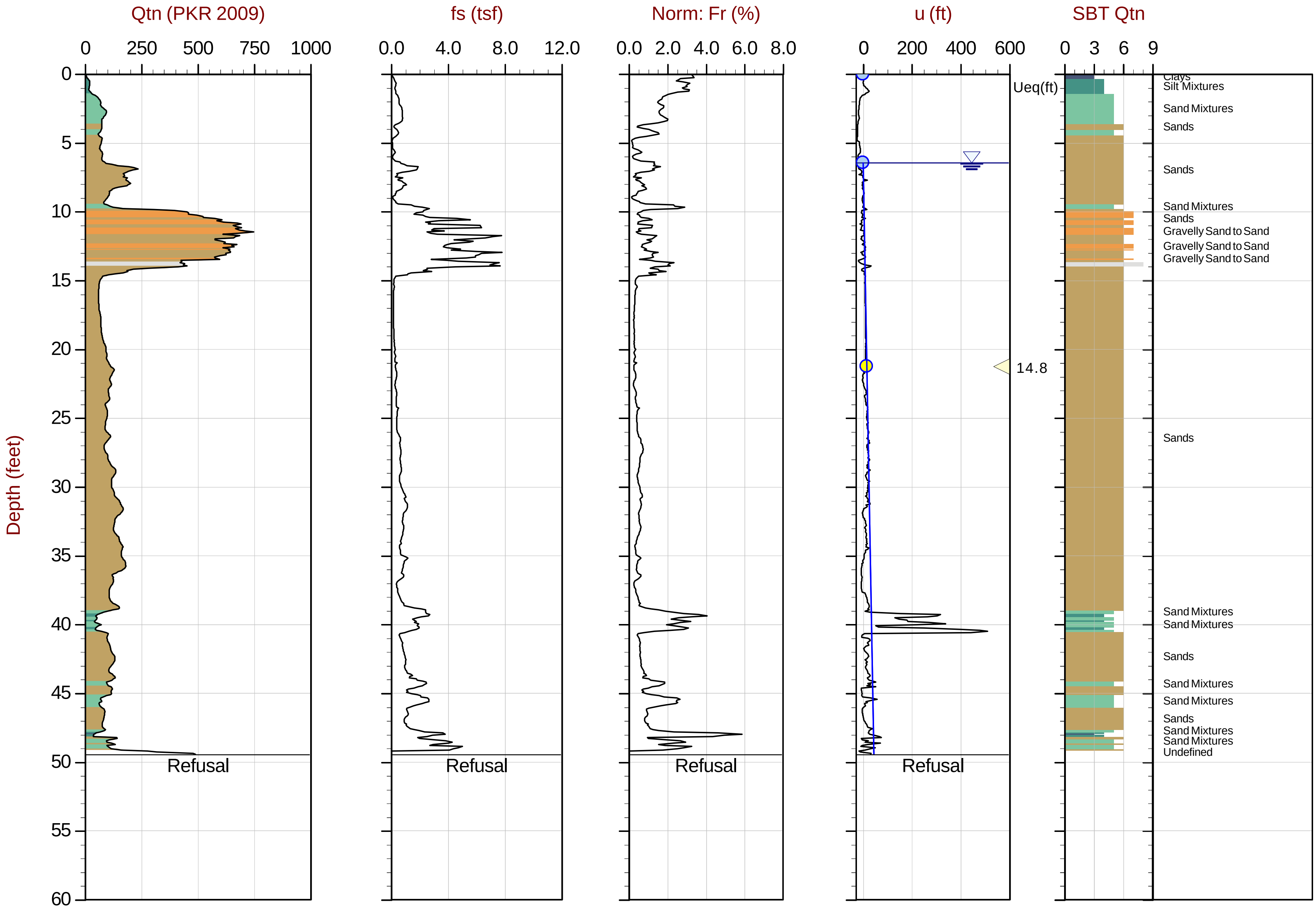
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 10:46
Site: Arlington Parcel

Sounding: CPT-03
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18714 Long: -122.14897

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

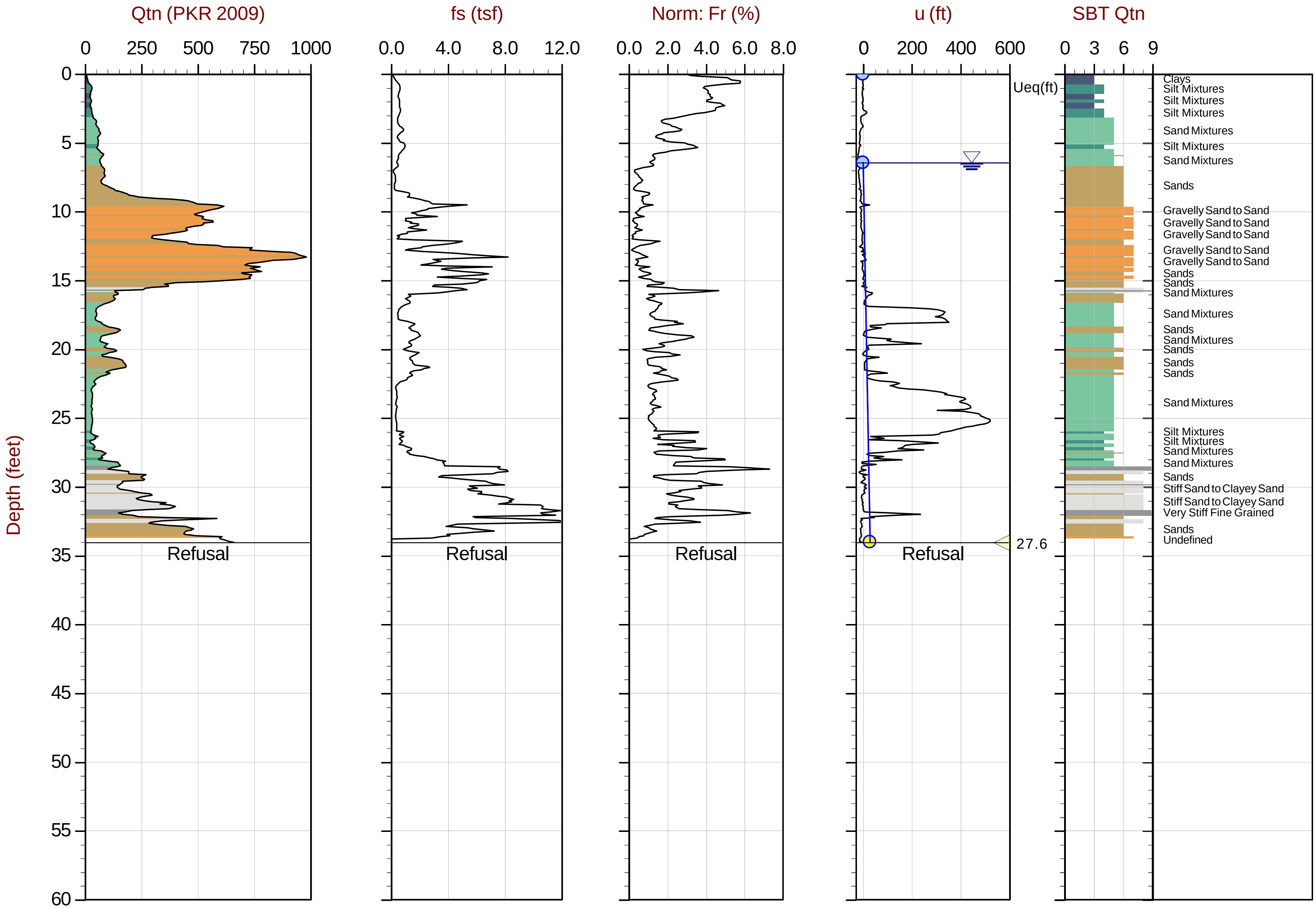
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 12:03
Site: Arlington Parcel

Sounding: CPT-04
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 10.375 m / 34.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18680 Long: -122.15056

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

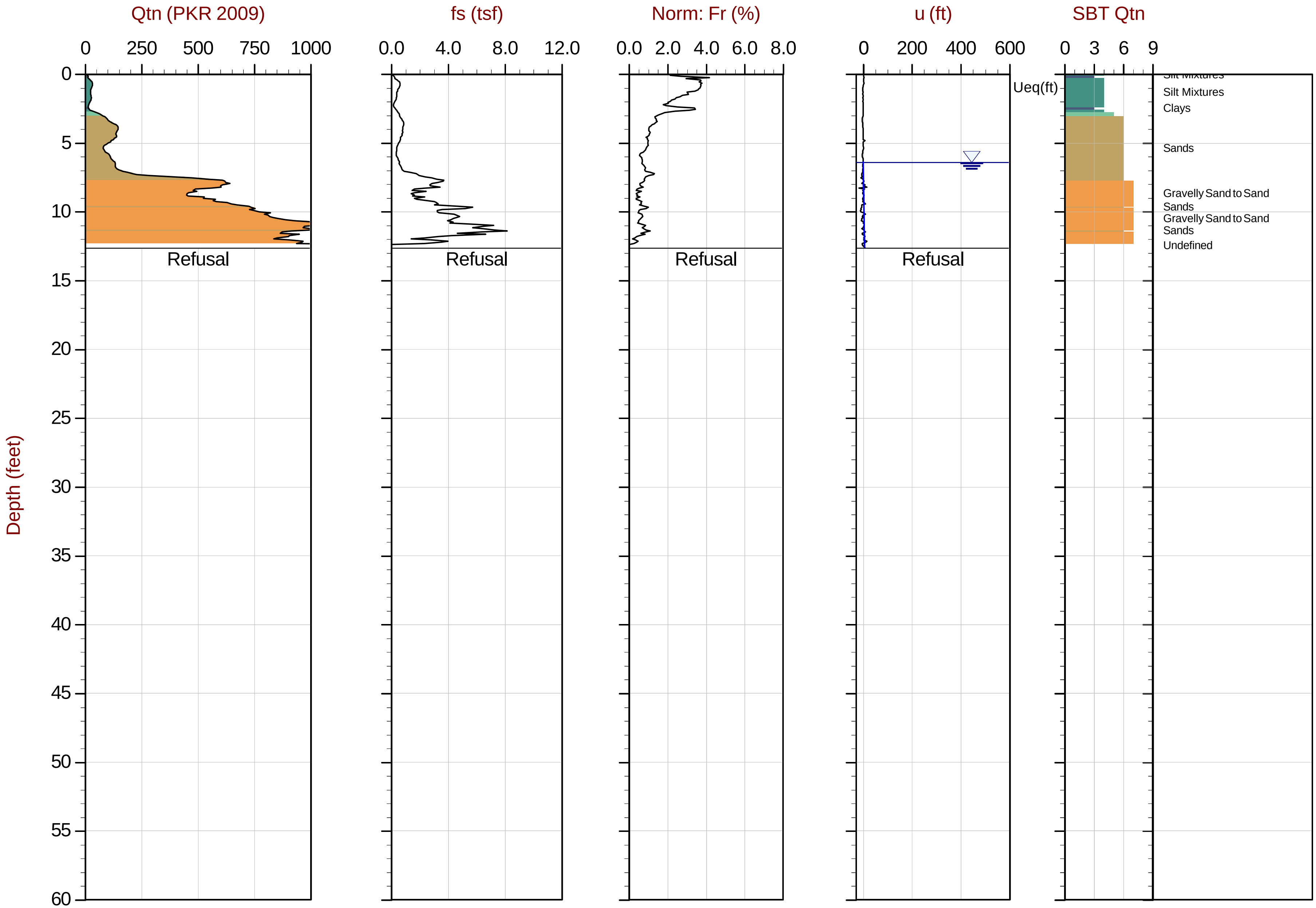
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 13:34
Site: Arlington Parcel

Sounding: CPT-04B
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 3.850 m / 12.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_CP04B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18676 Long: -122.15055

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.

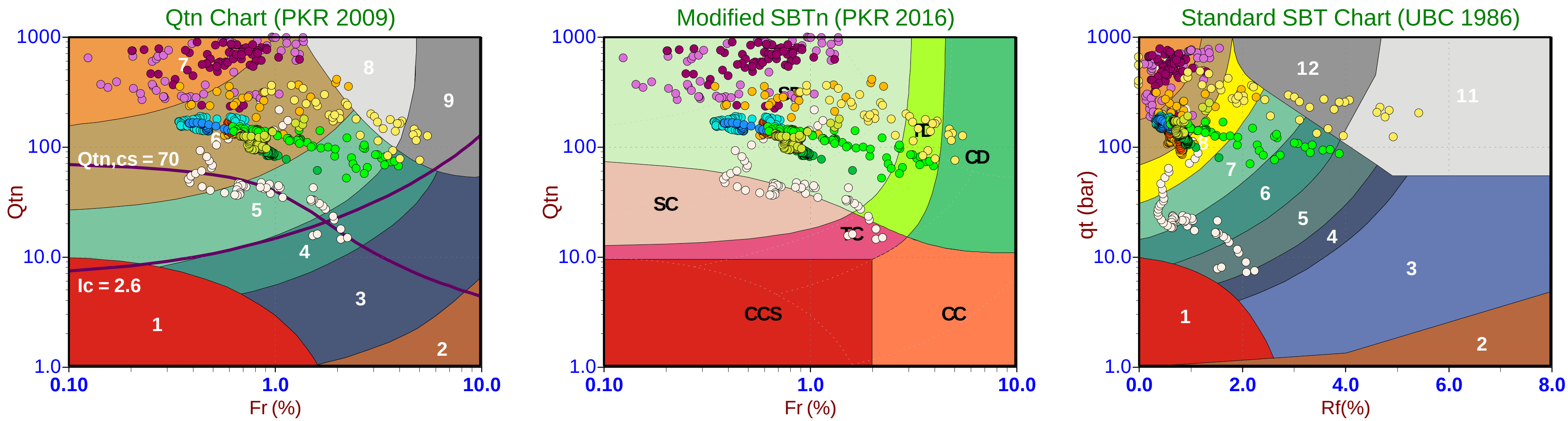
Soil Behavior Type (SBT) Scatter Plots

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel

Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Depth Ranges

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

Legend

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

Legend

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

Legend

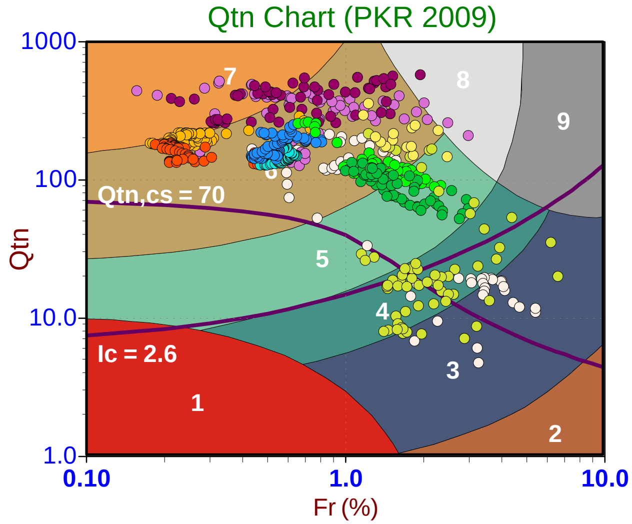
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

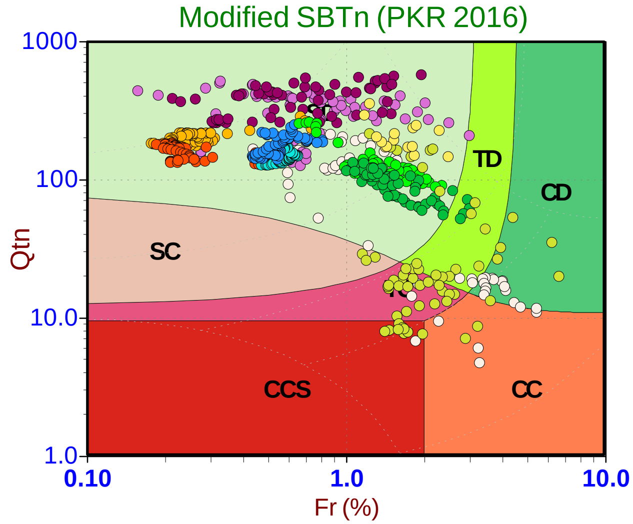
CONETEC

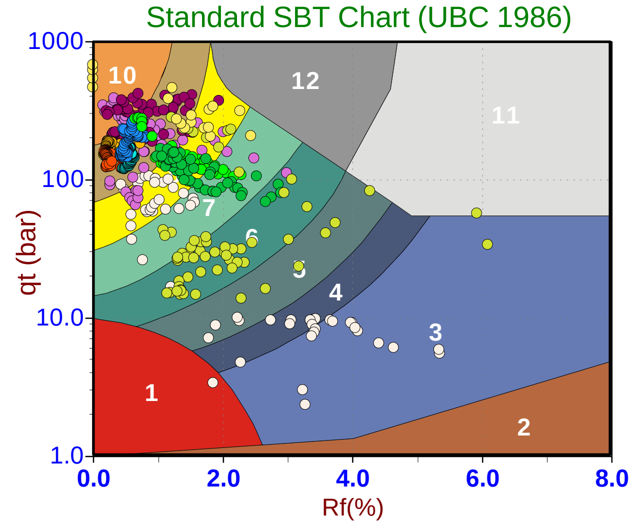
PanGEO

Job No: 24-59-27779
Date: 2024-06-28 09:37
Site: Arlington Parcel

Sounding: CPT-02
Cone: 855:T1500F15U35 Area=15 cm²







Depth Ranges

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

Legend

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

Legend

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

Legend

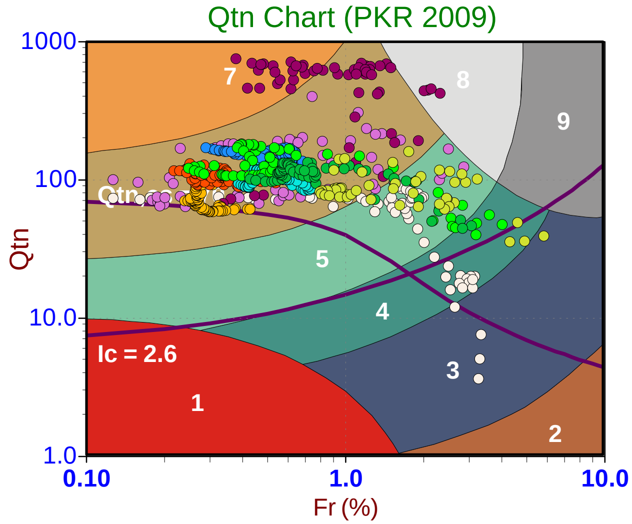
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

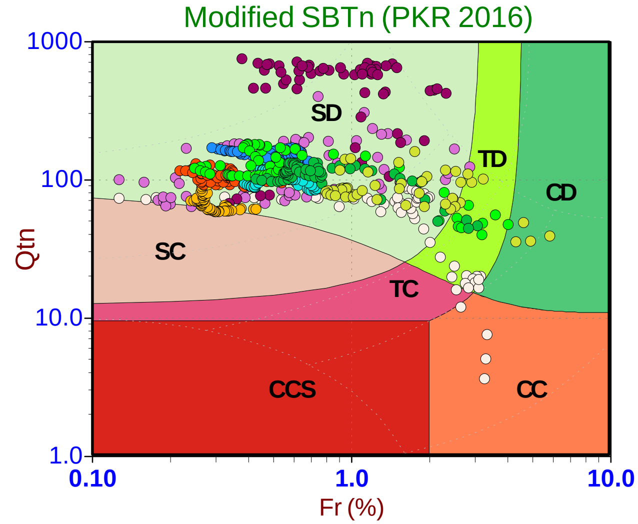
CONETEC

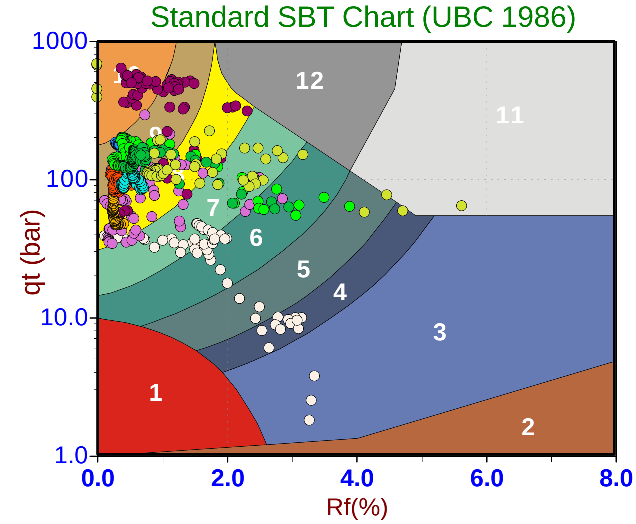
PanGEO

Job No: 24-59-27779
Date: 2024-06-28 10:46
Site: Arlington Parcel

Sounding: CPT-03
Cone: 855:T1500F15U35 Area=15 cm²







Depth Ranges

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

Legend

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

Legend

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

Legend

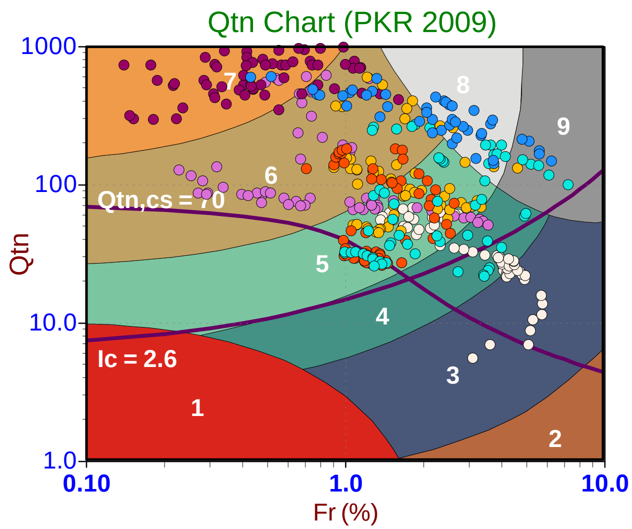
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

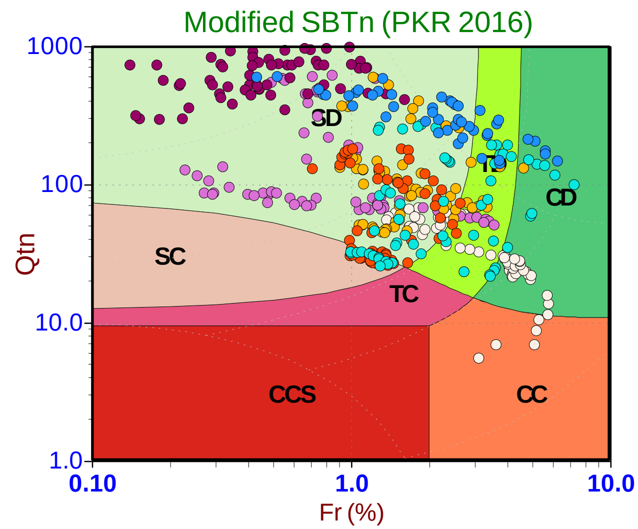
CONETEC

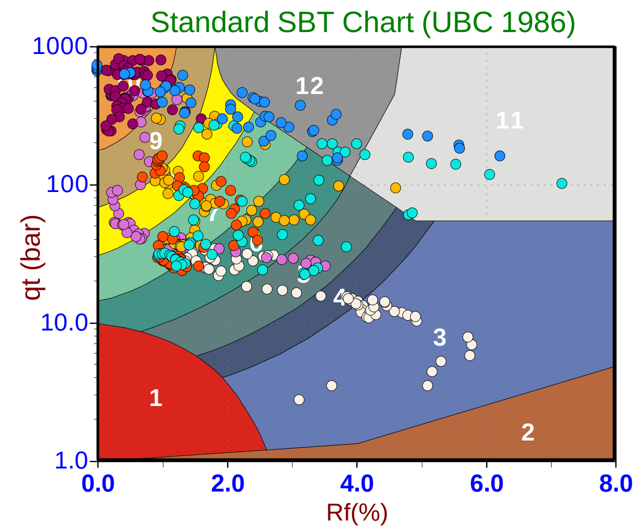
PanGEO

Job No: 24-59-27779
Date: 2024-06-28 12:03
Site: Arlington Parcel

Sounding: CPT-04
Cone: 855:T1500F15U35 Area=15 cm²







Depth Ranges

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

Legend

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

Legend

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

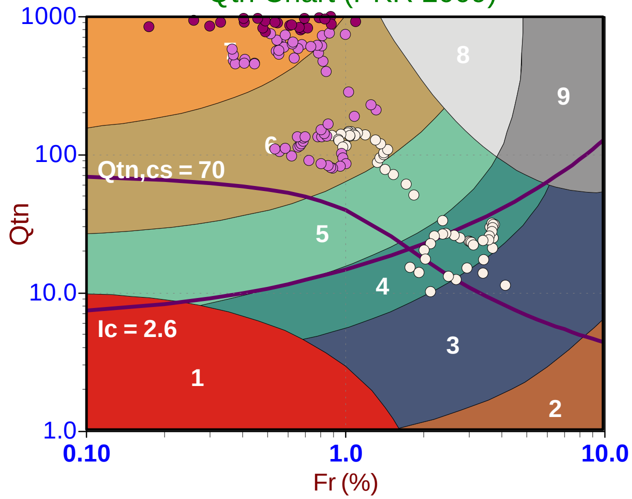
Legend

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

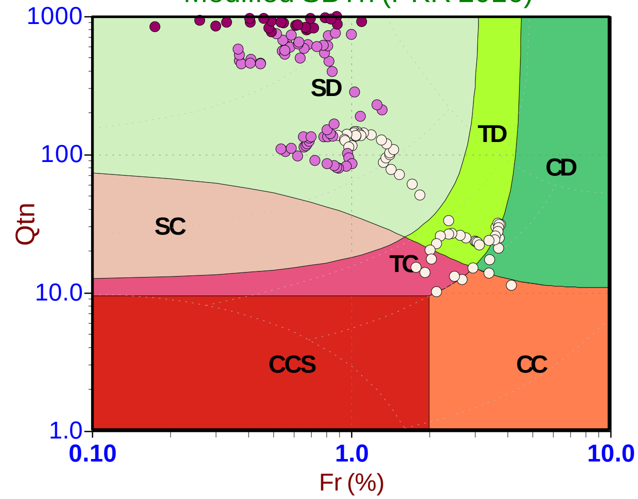
CONETEC

PanGEO	Job No: 24-59-27779	Sounding: CPT-04B
	Date: 2024-06-28 13:34	Cone: 855:T1500F15U35 Area=15 cm²
	Site: Arlington Parcel	

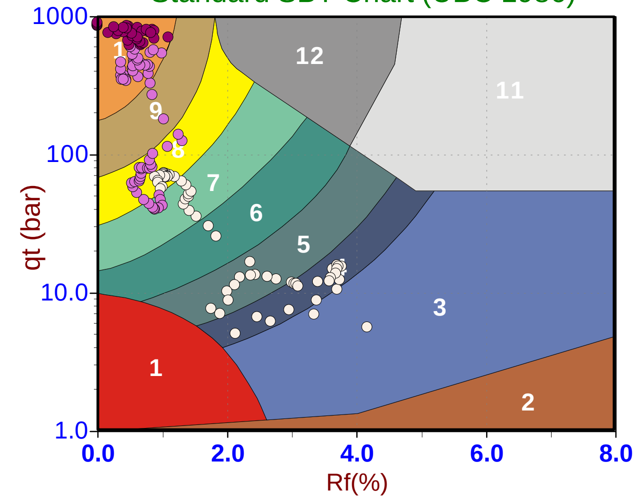
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

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

Legend

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

Legend

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

Legend

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

Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots

Job No:	24-59-27779
Client:	PanGEO
Project:	Arlington Parcel
Start Date:	2024-06-28
End Date:	2024-06-28

CPTu PORE PRESSURE DISSIPATION SUMMARY

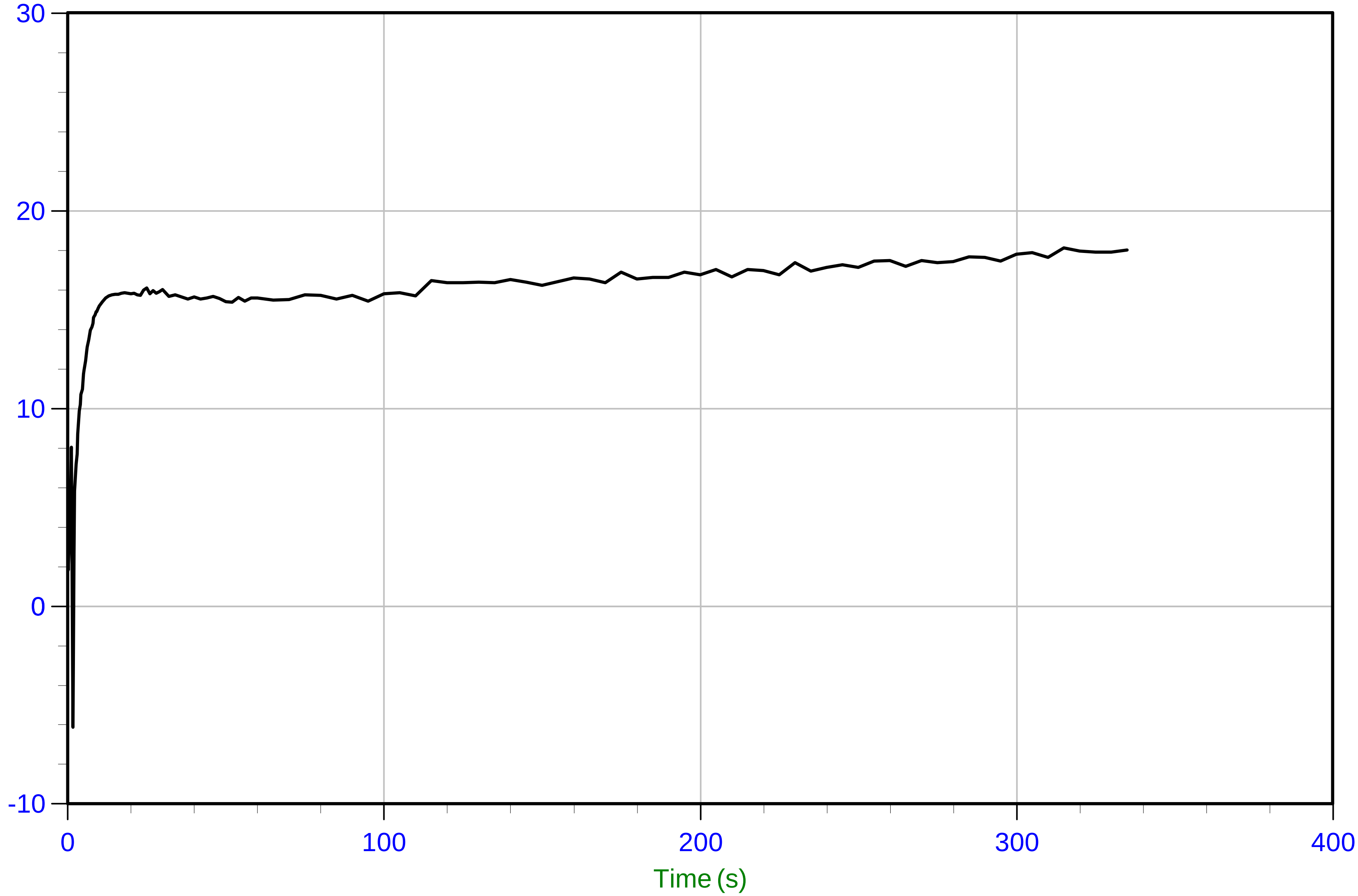
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Refer to Notation Number
CPT-01	24-59-27779_SP01	15	335	24.44	18.0	6.4	
CPT-02	24-59-27779_CP02	15	600	15.09	10.5	4.6	
CPT-03	24-59-27779_CP03	15	740	21.24	14.8	6.4	
CPT-04	24-59-27779_CP04	15	630	34.04	27.6	6.4	
Totals			38 min				

ConeTec

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel

Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



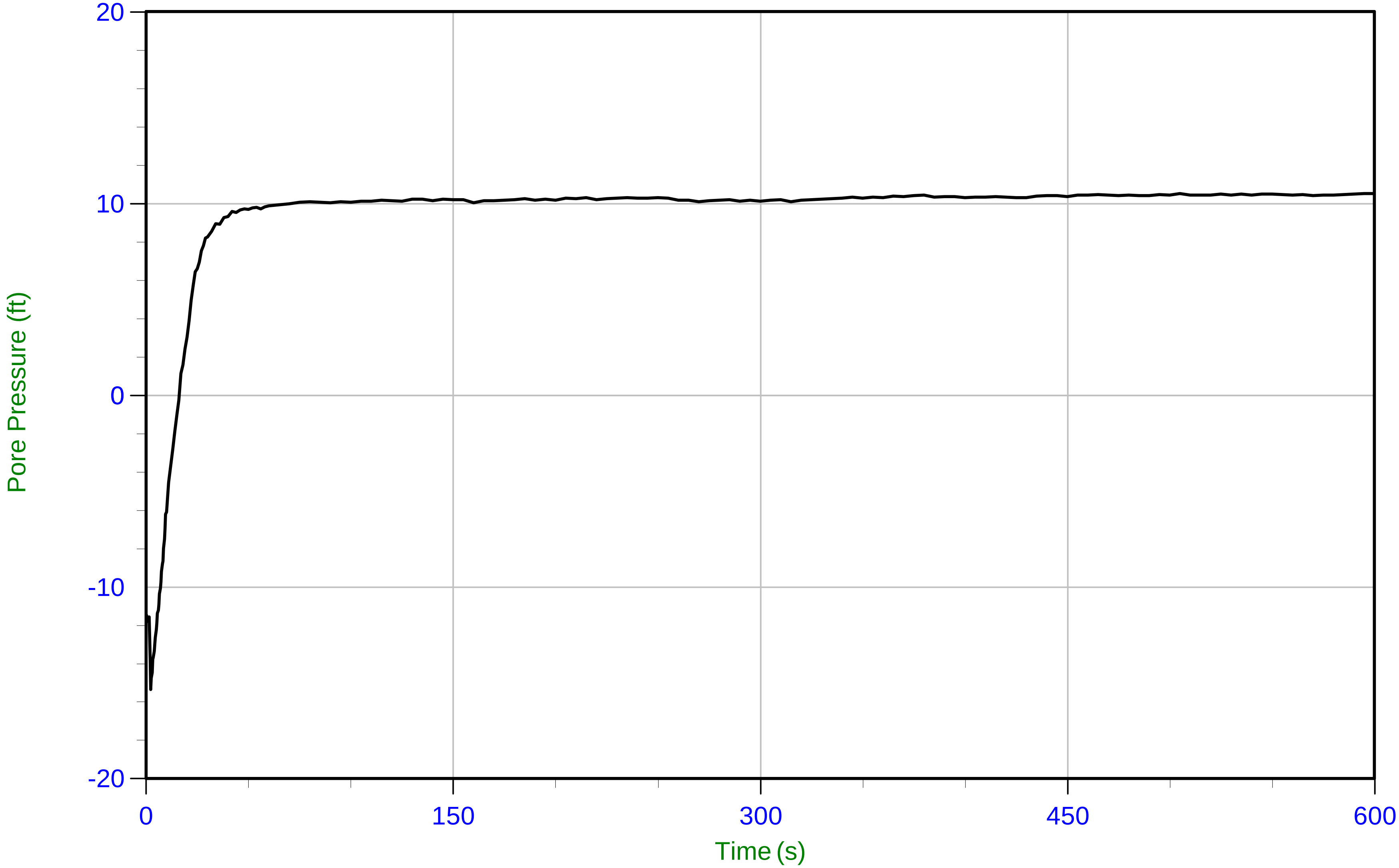
Trace Summary:

Filename: 24-59-27779_SP01.PPF2
Depth: 7.450 m / 24.442 ft
Duration: 335.0 s

u Min: -6.1 ft
u Max: 18.1 ft
u Final: 18.0 ft

WT: 2.0 m / 6.4 ft
Ueq: 18.0 ft

PanGEO	Job No: 24-59-27779 Date: 2024-06-28 09:37 Site: Arlington Parcel	Sounding: CPT-02 Cone: 855:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27779_CP02.PPF2
Depth: 4.600 m / 15.092 ft
Duration: 600.0 s

u Min: -15.4 ft
u Max: 10.5 ft
u Final: 10.5 ft

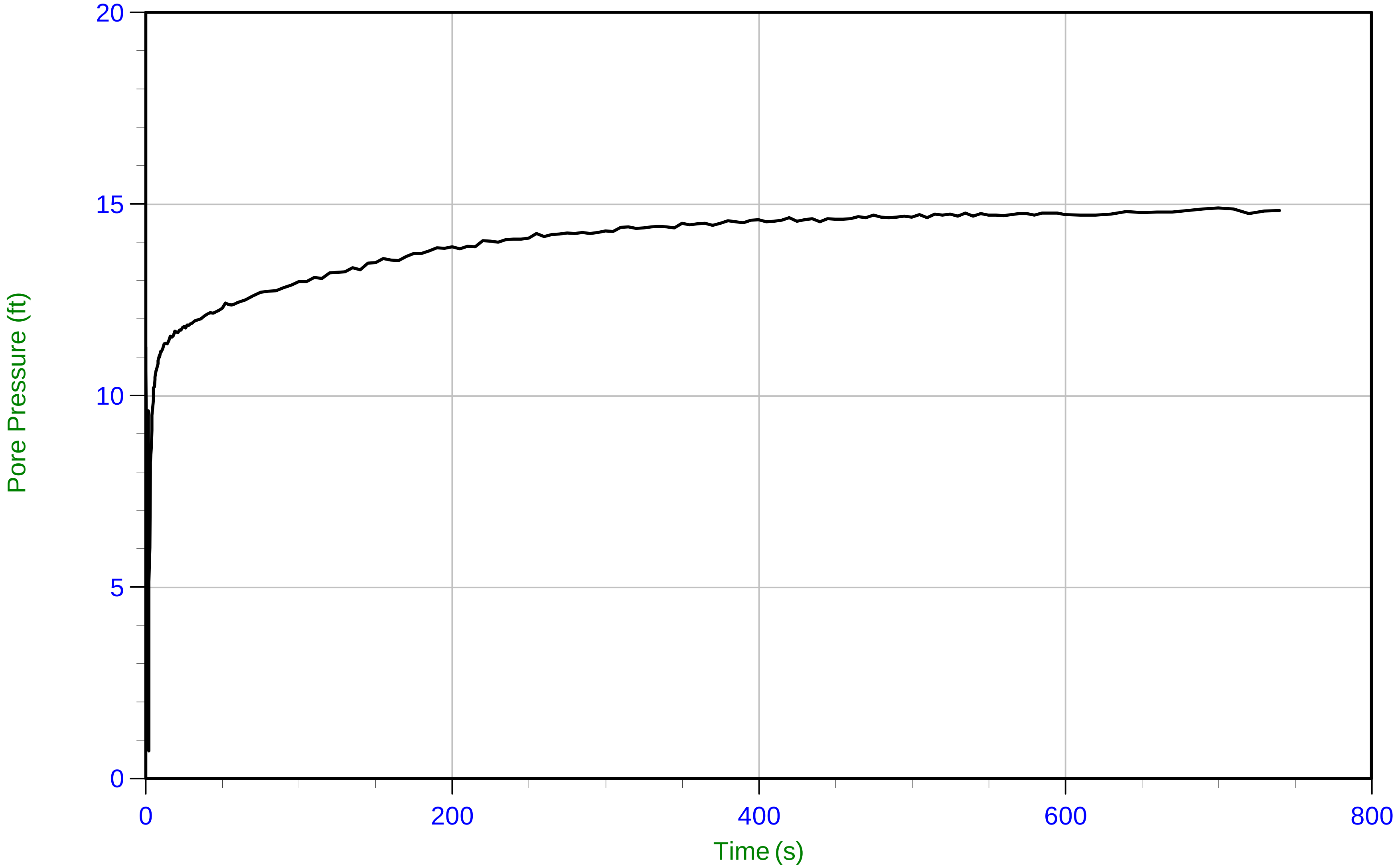
WT: 1.4 m / 4.6 ft
Ueq: 10.5 ft

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 10:46
Site: Arlington Parcel

Sounding: CPT-03
Cone: 855:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27779_CP03.PPF2
Depth: 6.475 m / 21.243 ft
Duration: 740.0 s

u Min: 0.7 ft
u Max: 14.9 ft
u Final: 14.8 ft

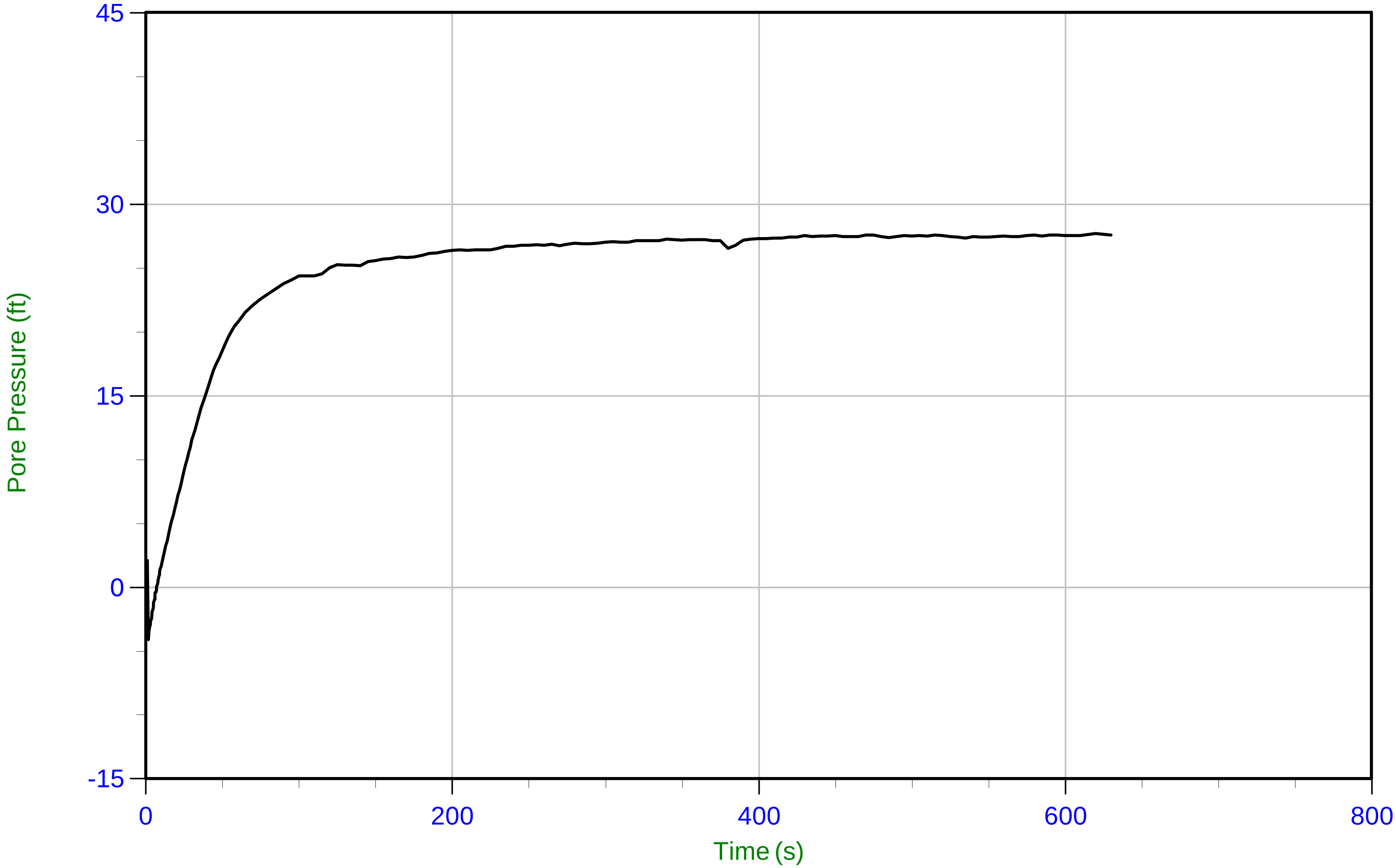
WT: 2.0 m / 6.4 ft
Ueq: 14.8 ft

CONETEC

PanGEO

Job No: 24-59-27779
Date: 2024-06-28 12:03
Site: Arlington Parcel

Sounding: CPT-04
Cone: 855:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27779_CP04.PPF2
Depth: 10.375 m / 34.038 ft
Duration: 630.0 s

u Min: -4.1 ft
u Max: 27.7 ft
u Final: 27.6 ft

WT: 2.0 m / 6.4 ft
Ueq: 27.6 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

ConeTec

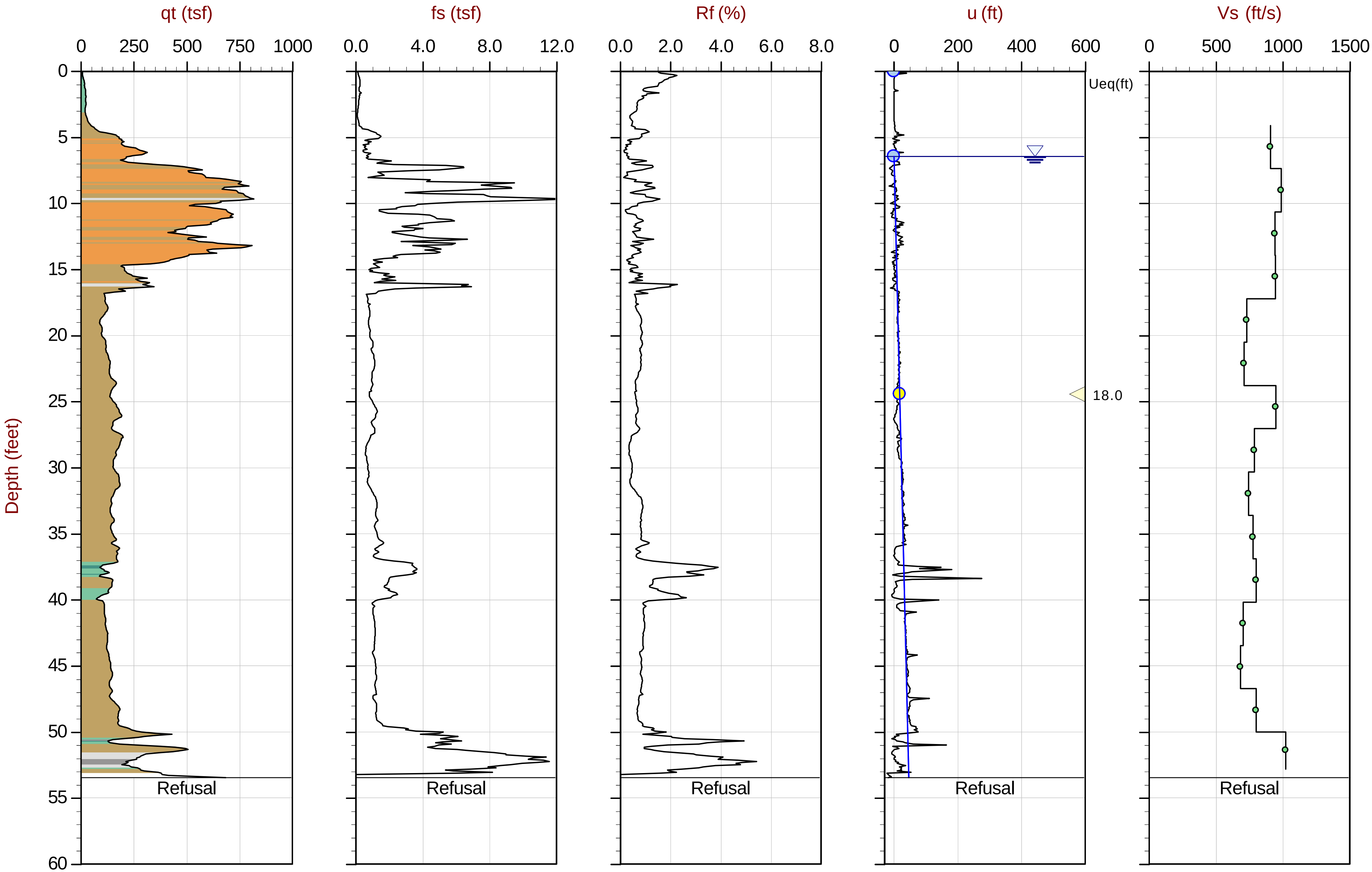
CONETEC	**Job No:**	24-59-27779
	Client:	PanGEO
	Project:	Arlington Parcel
	Sounding ID:	CPT-01
	Date:	2024-06-28
	Seismic Source:	Beam
	Seismic Offset (ft):	1.74
	Source Depth (ft):	0.00
	Geophone Offset (ft):	0.66

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)
4.76	4.10	4.45			
8.04	7.38	7.58	3.13	3.44	910
11.32	10.66	10.80	3.22	3.25	990
14.60	13.94	14.05	3.25	3.44	943
17.88	17.22	17.31	3.26	3.45	947
21.16	20.51	20.58	3.27	4.47	732
24.44	23.79	23.85	3.27	4.59	712
27.72	27.07	27.12	3.27	3.45	950
31.00	30.35	30.40	3.27	4.15	790
34.28	33.63	33.67	3.28	4.40	744
37.57	36.91	36.95	3.28	4.21	778
40.85	40.19	40.23	3.28	4.08	803
44.13	43.47	43.51	3.28	4.66	704
47.41	46.75	46.78	3.28	4.78	685
50.69	50.03	50.06	3.28	4.08	803
53.48	52.82	52.85	2.79	2.73	1023

SCPTu Test Plots

Job No: 24-59-27779
Date: 2024-06-28 08:14
Site: Arlington Parcel
Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 16.300 m / 53.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27779_SP01.COR
Unit Wt: SBTQtn (PKR2009)

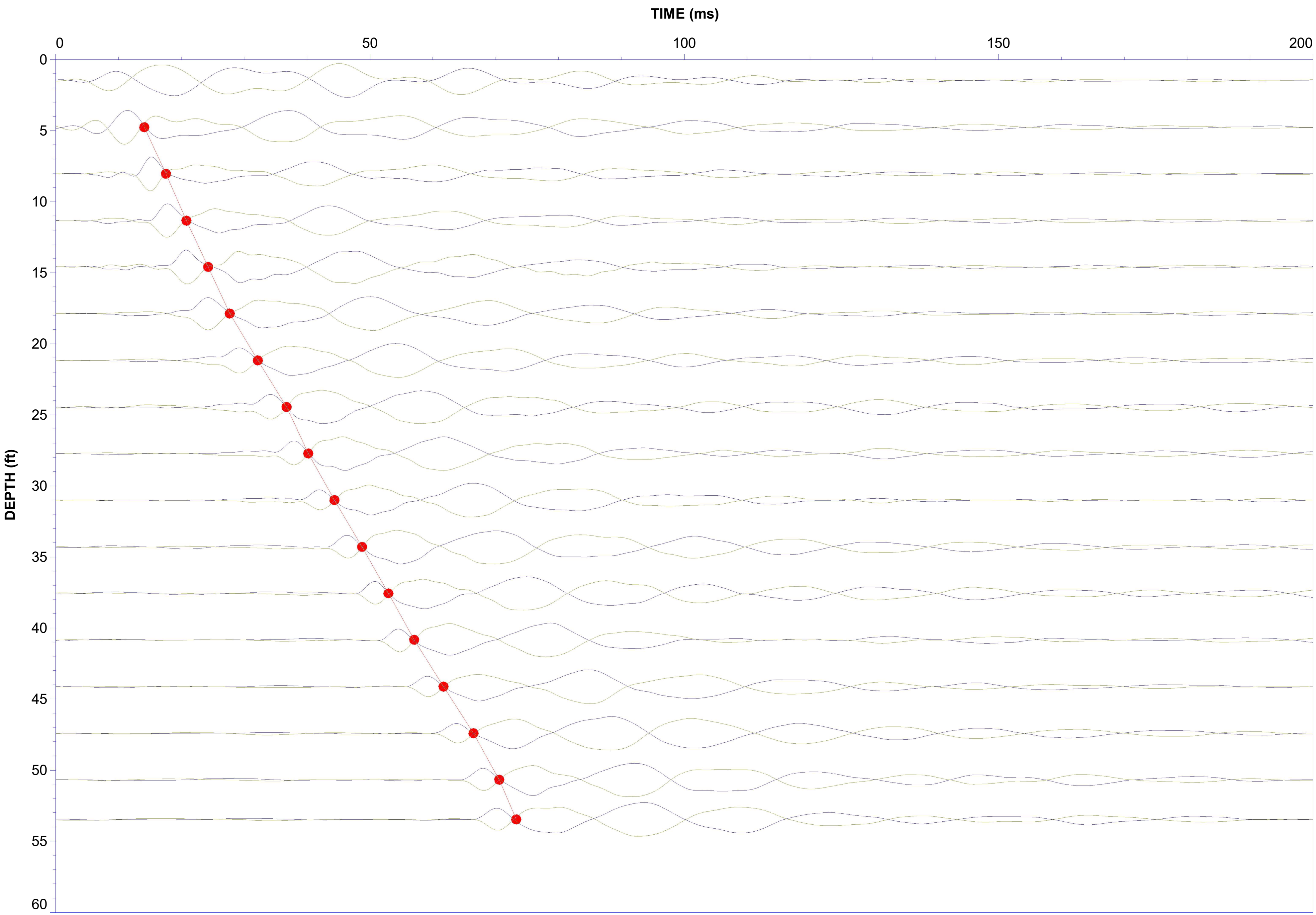
SBT: Robertson, 2009 and 2010
Coords: Lat: 48.18761 Long: -122.14984

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.

SCPTu Velocity Wave Traces

Job No: 24-59-27779 | Client: PanGEO | Project: Arlington Parcel | Filter: BP 0-300 Hz | Sounding: CPT-01 | Analysis: Shear Wave
Date: 2024-06-28 | Cone: 855:T1500F15U35



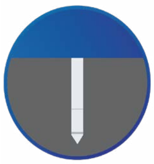
SUPPORTING DOCUMENTS AND MATERIALS

The documents and materials listed below are included in the report:

- **Methodology Statements**
- **Cone Penetration Digital File Formats**
- **Description of Methods for Calculated CPTu Geotechnical Parameters**
- **Calibration Records**

Methodology Statements

METHODOLOGY STATEMENTS



CONE PENETRATION TEST (CPTu) - eSeries

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

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

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

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

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

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

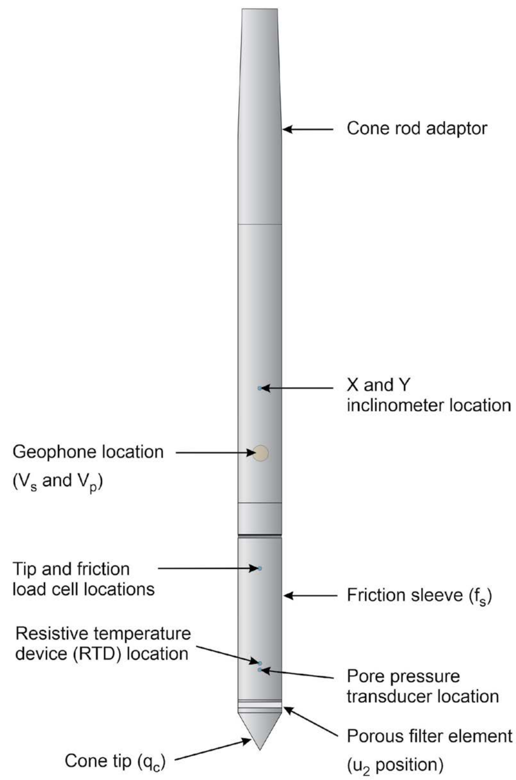


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

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

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

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

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

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

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

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

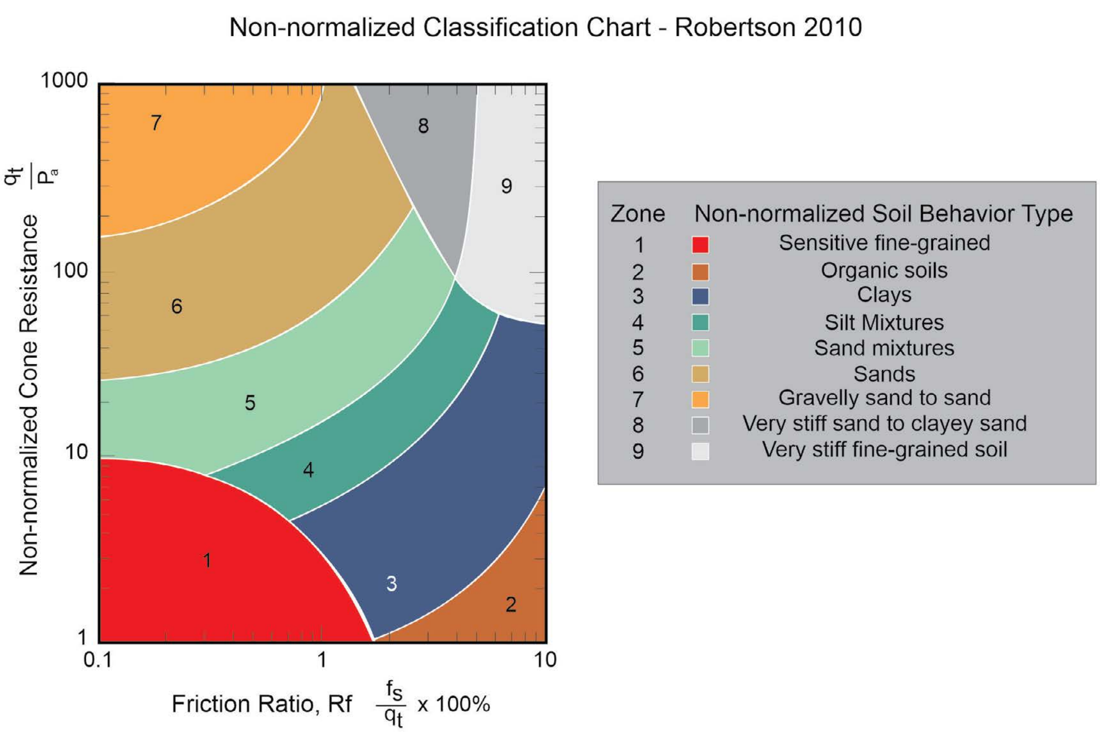


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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

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

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

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

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

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

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

REFERENCES

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

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

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

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

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

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

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

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

PORE PRESSURE DISSIPATION TEST

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

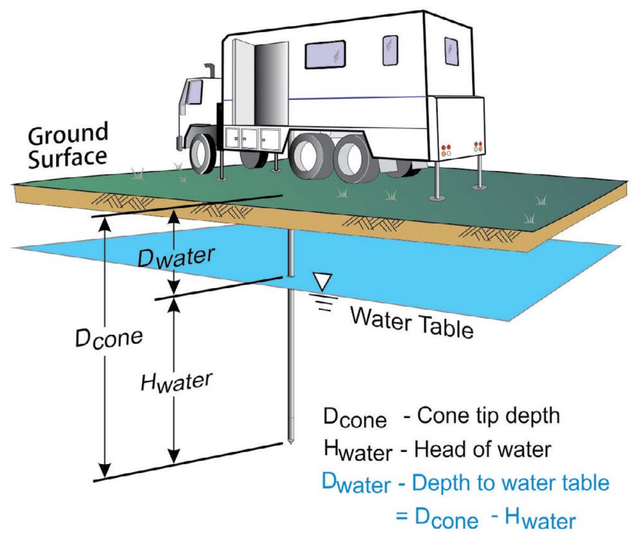


Figure PPD-1. Pore pressure dissipation test setup

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

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

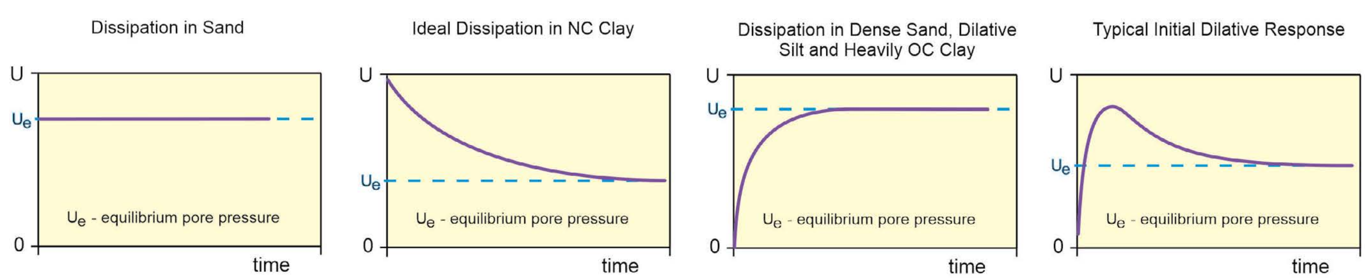


Figure PPD-2. Pore pressure dissipation curve examples

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

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

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.

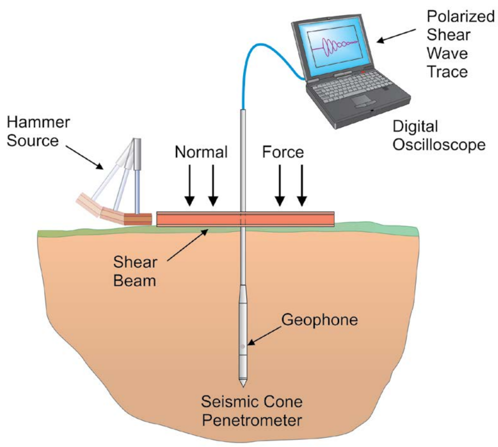


Figure SCPTu-1. Illustration of the SCPTu system

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).

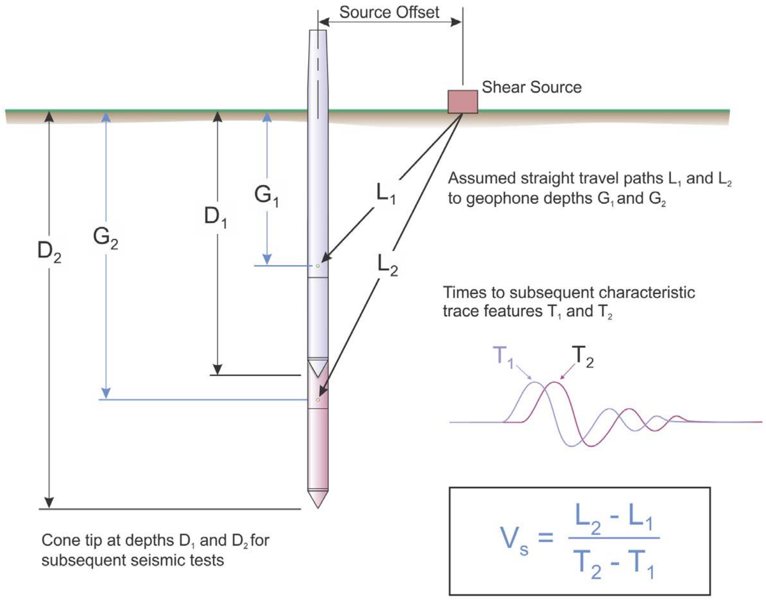


Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

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.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

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

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 ($\overline{v}_S$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

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

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 $V_{S}100$ or $V_{S}30$.

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

REFERENCES

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Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

Cone Penetration Digital File Formats

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

CONETEC

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023

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

Limitations

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

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

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

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

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

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

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

a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

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

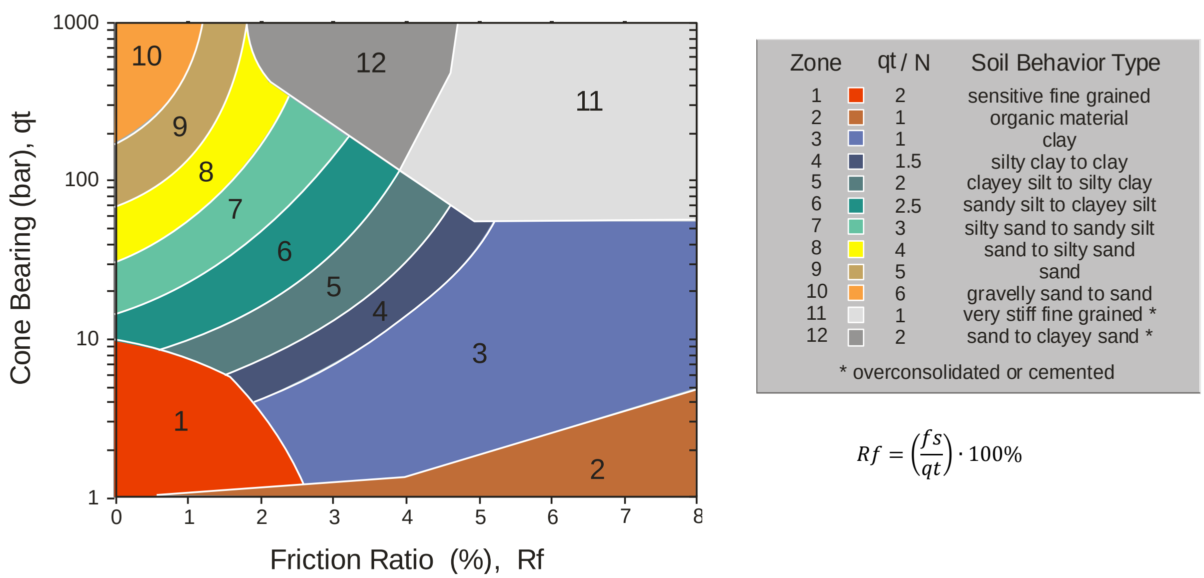
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from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

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

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

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

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



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

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

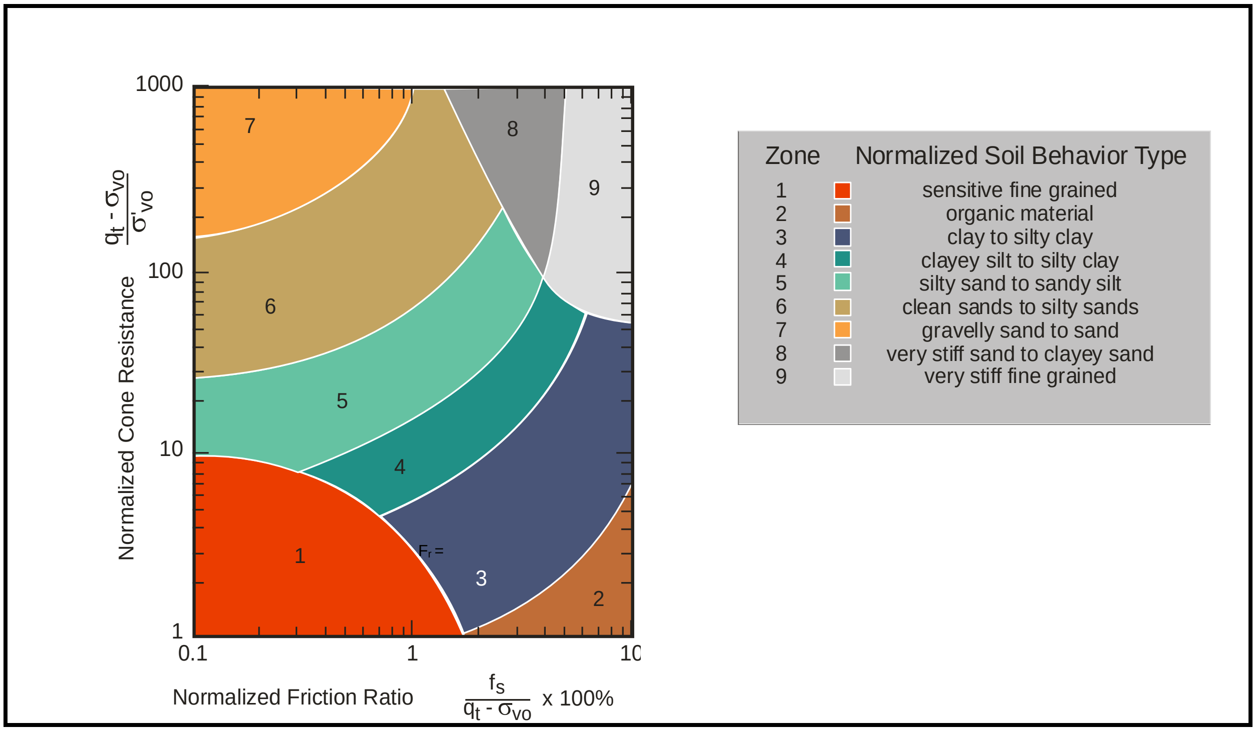


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

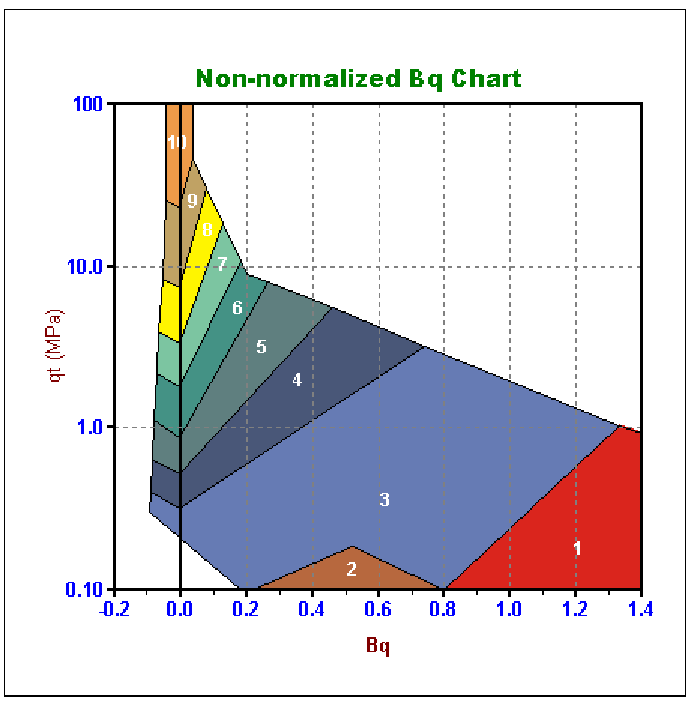


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

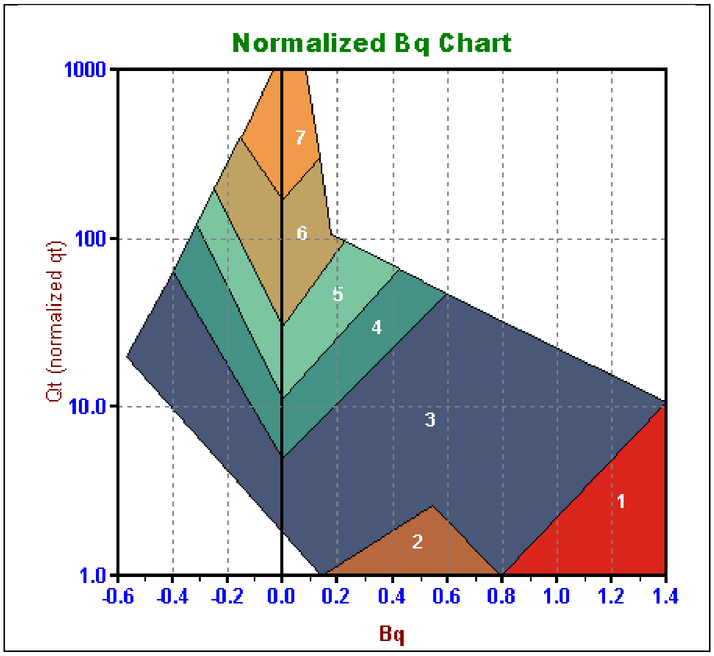


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

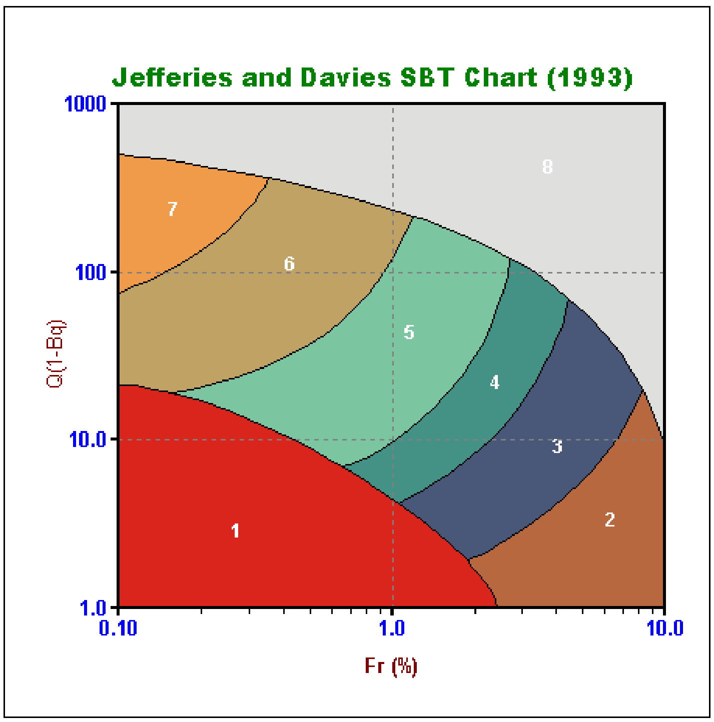


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

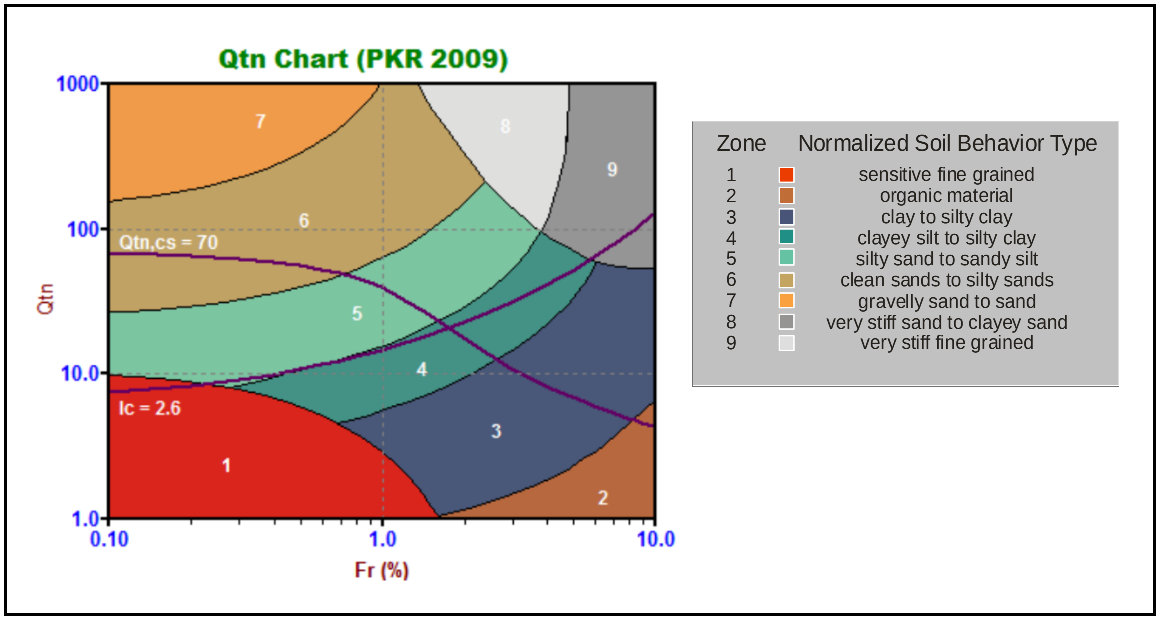


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

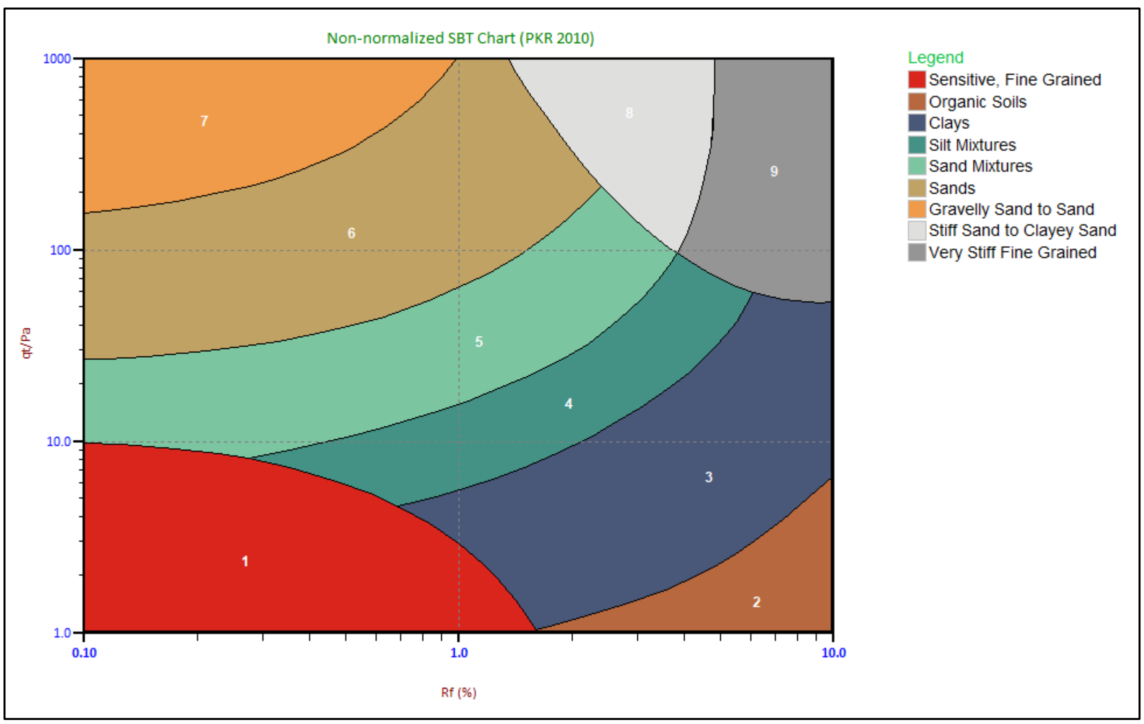


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

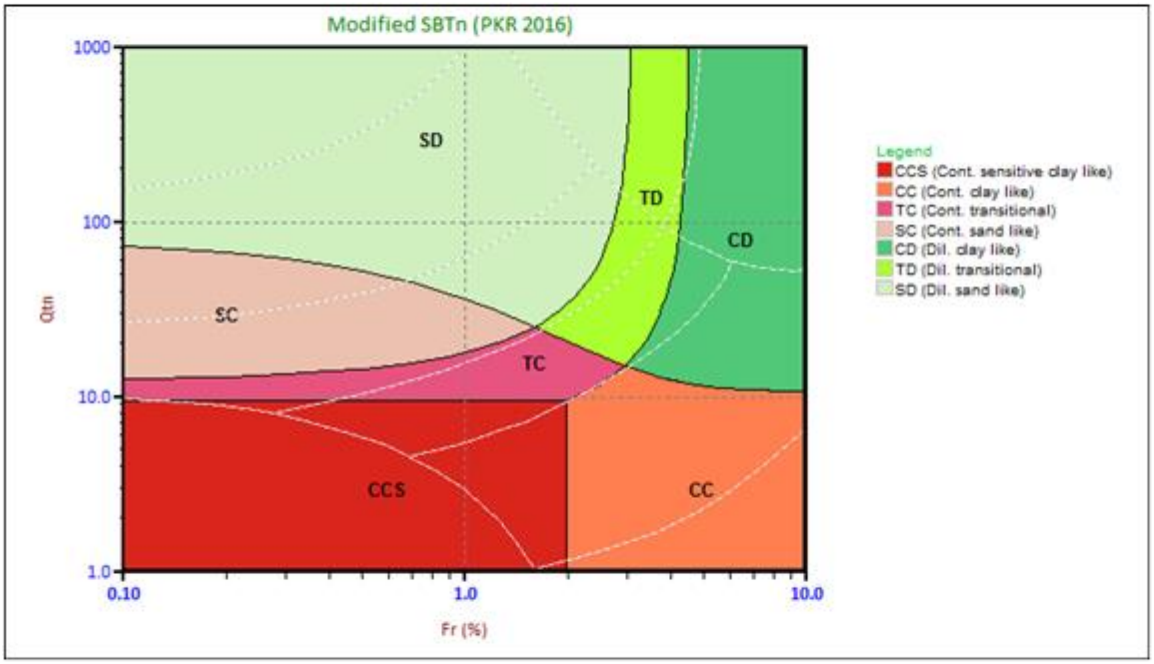


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

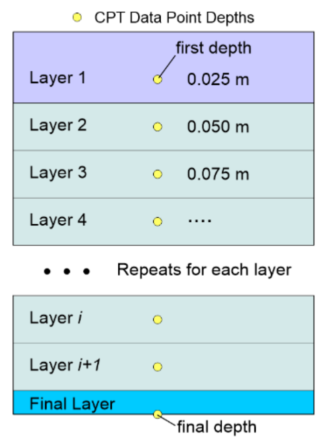
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

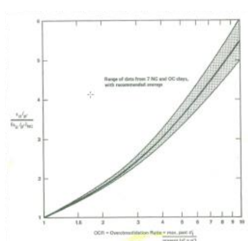
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

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

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

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

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

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

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

Table 2. References

No.	Reference
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Calibration Records

Adara Systems Ltd.
201-8327 Eastlake Drive, Burnaby, BC, Canada V5A4W2
Tel: 1-604-273-6013

CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC855	Model	A15 T1500 F15 U35
Date	2024-02-08		
Technician Performing Calibration	Richard Chen	Signature	
Calibration Approved By	Vishrut Khunt	Signature	

Lab Condition	As Found	As Left		
Lab Temperature	N/A	23˚C		
Lab Humidity	N/A	29%	Reason for Calibration	Repair

Cone Information				
Tip Stress Limit	1500	bar	Tip End Area	15 cm²
Friction Stress Limit	15	bar	Friction Surface Area	225 cm²
Pressure Limit	35	bar	RTD Location	Pressure Carrier
X-Inclinometer Limit	30	degrees	Geophone	X and Z
Y-Inclinometer Limit	30	degrees	Temperature Range	-20°C to 60°C

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	-0.001	0.503
Sleeve	bar	0.000	-0.012
Pressure	bar	0.037	1.012
X-Inclinometer	degrees	-0.675	0.000
Y-Inclinometer	degrees	1.925	0.000
Temperature	˚C	24.574	22.279

Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

Calibrated with Adara calibration procedure EC_CPTCAL-2.2

Collective uncertainty of the measurement standards conforms to a test uncertainty ratio (TUR) of 3:1 for tip and sleeve measurement and 4:1 for pressure measurement with a confidence level k=2

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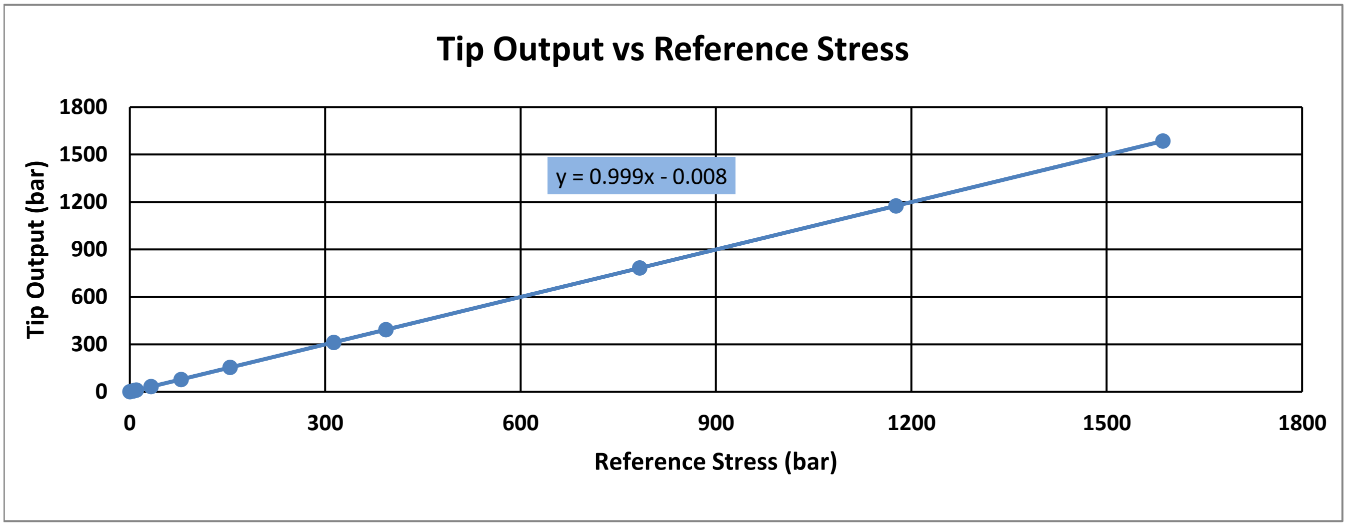
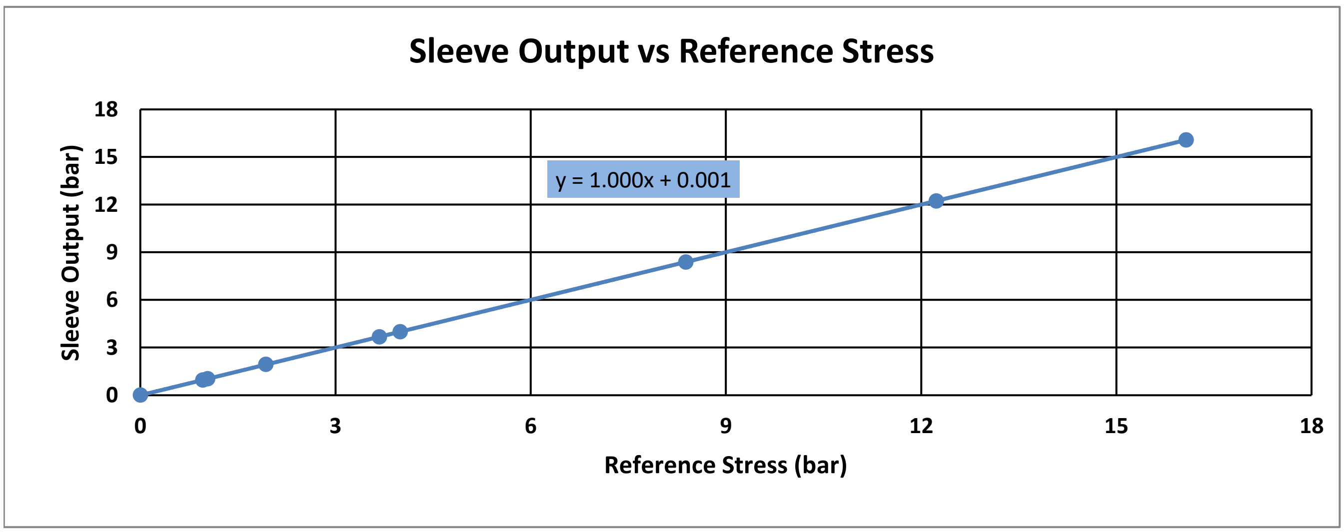
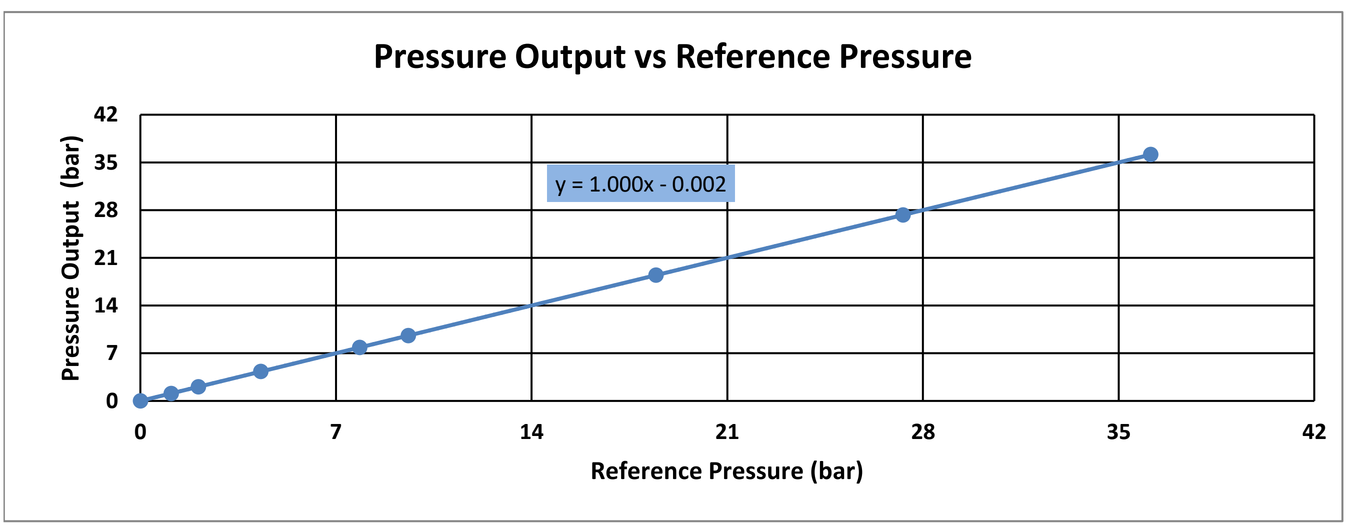
Cone Output vs Reference Stress/Pressure Plots







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Calibration Results

Tip Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.08%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.10%	PASS

Sleeve Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.05%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.14%	PASS

Pressure Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.02%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.18%	PASS

X-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.37%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.75%	PASS

Y-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.25%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.50%	PASS

Seismic Calibration					
As Found			As Left		
Trigger Delay Error	N/A	N/A	Trigger Delay Error	0.01%	PASS

Temperature Calibration			
Full Scale Error	0.18%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	4.8	22.1	42.7	˚C
Tip	-2.473	-0.197	2.769	bar
Sleeve	0.012	-0.016	-0.038	bar
Pressure	1.042	1.057	1.054	bar
Temperature	4.941	21.927	42.790	˚C

Tip Temperature Coefficient	0.138 bar/˚C	PASS
Sleeve Temperature Coefficient	-0.001 bar/˚C	PASS
Pressure Temperature Coefficient	0.000 bar/˚C	PASS

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Testing Equipment Details

Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3910	100835	2024-12-12
Tektronix Function Generator	AFG3021B	C030955	100751	2024-10-20
Thermometer	THS-222-555	D23255834	100410	2024-07-11
Thermometer	THS-222-555	D23255829	100410	2024-07-11
Thermometer	THS-222-555	D20345575	100565	2024-07-14

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Adara Error Definitions

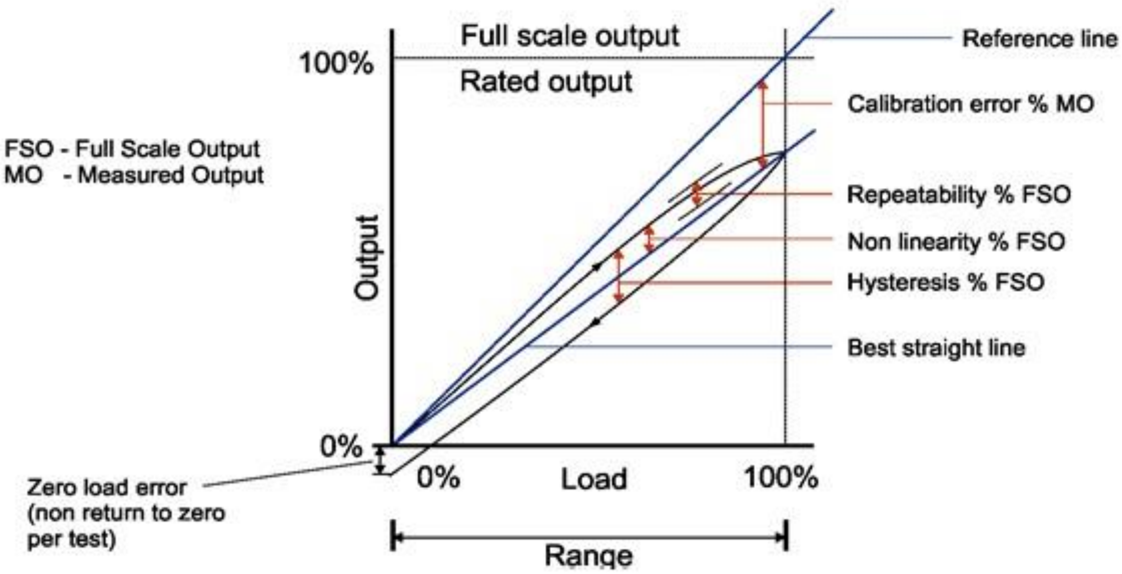

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

Actual Sensitivity	The slope of the best fit line through all data points starting at zero load.
Slope Error	The error in the best fit line compared to the ideal linear calibration in % . Slope Error = (Best Fit Slope - Ideal Slope) / Ideal Slope
Maximum Non Linearity	This value represents the maximum error (absolute value) relative to the best fit line considering each calibration point starting at loads greater than approximately 10% of FSO. The reported errors are a percent error of FSO. Adara's Pass/Fail criteria is 0.5% of FSO (ASTM is 0.5% of FSO at loads > 20% FSO).
Calibration Error	This value represents the maximum error (absolute value) in the recorded load value as compared to the actual load value for each calibration point for loads greater than approximately 10% of FSO. Adara's Pass/Fail criteria for the tip and sleeve is 0.5% of MO and 1.0% of MO for the pore pressure (ASTM for the tip and sleeve is 1.5% and 1.0% of MO respectively at loads greater than 20% of FSO)

Temperature Check Passing Criteria

Tip Temperature Coefficient	<0.200	bar/˚C
Sleeve Temperature Coefficient	<0.005	bar/˚C
Pressure Temperature Coefficient	<0.0196	bar/˚C

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ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

Non Linearity	Tip	≤ +0.5% of FSO
	Sleeve	≤ +1.0% of FSO
Calibration Error	Tip	≤ +1.5% of MO at loads > 20% FSO
	Sleeve	≤ +1.0% of MO at loads > 20% FSO

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

Non Linearity	Pore Pressure	≤ +1.0% of FSO
Calibration Error	Pore Pressure	not specified

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

Class 1	Cone Resistance	35 kPa or 5%
	Sleeve Friction	5 kPa or 10%
	Pore Pressure	10 kPa or 2%
Class 2	Cone Resistance	100 kPa or 5%
	Sleeve Friction	15 kPa or 15%
	Pore Pressure	25 kPa or 3%

Note: ISO Compliance is based on low end calibration only.

References

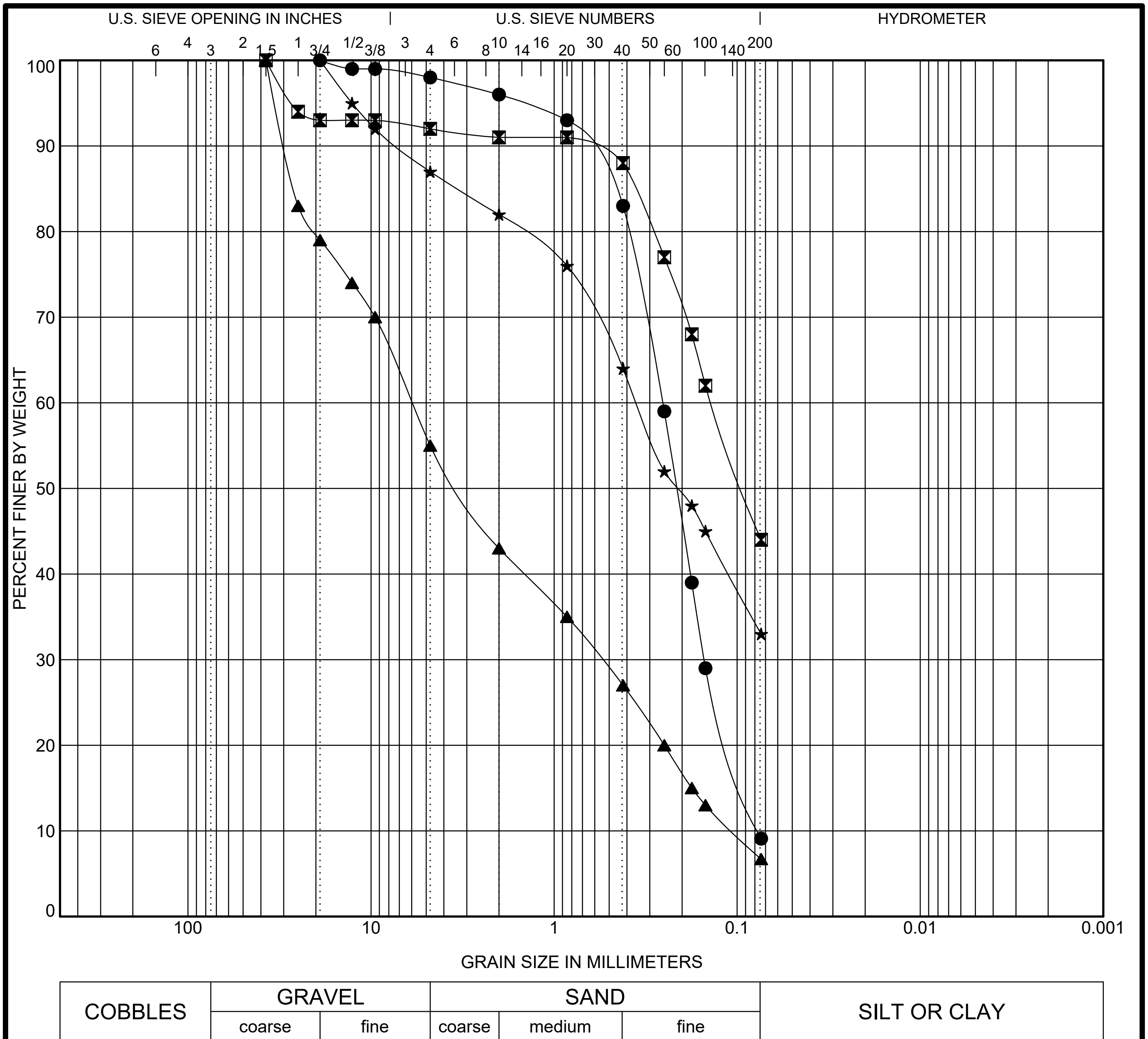
[1] ASTM D5778-20. "Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils". ASTM, West Conshohocken, PA, USA.

[2] ISO 22476-1:2012. "Geotechnical investigation and testing - Field Testing - Part 1: Electrical cone and piezocone penetration test". ISO, Geneva, Switzerland.

ASTM D7400-08. "Standard Test Methods for Downhole Seismic Testing". ASTM, West Conshohocken, PA, USA.

APPENDIX C

LABORATORY TEST RESULTS



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● PG-1	@ 2.5 ft.	**POORLY GRADED SAND with SILT(SP-SM)**				**1.18**	**3.34**
⧗ PG-2	@ 2.5 ft.	**SILTY SAND(SM)**					
▲ PG-3	@ 5.0 ft.	**POORLY GRADED SAND with SILT and GRAVEL(SP-SM)**				**0.47**	**56.05**
★ PG-4	@ 2.5 ft.	**SILTY SAND(SM)**					

Specimen Identification		D100	D90	D60	D10	%Gravel	%Sand	%Silt	%Clay
● PG-1	2.5	19	0.688	0.255	0.076	2.0	88.5	9.5	
⧗ PG-2	2.5	37.5	0.672	0.138		8.0	47.7	44.3	
▲ PG-3	5.0	37.5	29.543	5.985	0.107	45.0	48.2	6.8	
★ PG-4	2.5	19	7.2	0.353		13.0	53.8	33.2	

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GRAIN SIZE DISTRIBUTION

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Job Number: 24-207
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Figure C-1