Geotechnical Engineering Services

Northsound Logistics Center
Arlington, Washington

for
Rockefeller Group

November 21, 2024

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1.0 Introduction

This report presents the results of our geotechnical engineering services for the Northsound Logistics Center project located in Arlington, Snohomish County, Washington. The site includes one parcel (Snohomish Couty Parcel Number 31052800101700). The project site is shown relative to surrounding physical features in the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

The purpose of this study is to review existing geotechnical information and to complete subsurface explorations at the project site as a basis for providing geotechnical engineering recommendations for design and construction. Our specific scope of services includes:

- Reviewing previous explorations completed in the vicinity of the site;
- Completing three (3) cone penetration tests (CPT) and eight (8) borings/wells to characterize the subsurface conditions at the site;
- Evaluating the potential for geologic hazards, including fault rupture, liquefaction and lateral spreading;
- Performing analyses for seismic design, building foundations and floor slab support;
- Performing an evaluation of infiltration feasibility and provide preliminary infiltration rates; and
- Preparing this report.

2.0 Project Description

Based on our review of a conceptual plan[1] for the site, we understand that site development will encompass approximately 8.43 acres and will include an industrial building with potential office space occupying a total of 153,500 square feet (sf). We understand that the building will be constructed on a fill pad above existing site grade. Based on our experience with similar buildings, we anticipate that the fundamental period of vibration will be less than 0.5 seconds for evaluating seismic considerations. Site improvements will also include north loading dock areas, car parking stalls, stormwater management facilities and associated hardscape and landscape areas. Site grades will be raised to support the structure. Based on the conceptual civil plans prepared by Barghausen dated November 12, 2024, we understand that the finished floor for the building will be Elevation 123 feet (NAVD 88). Final site grades will generally range from Elevation 123 to Elevation 120 feet.

Based on our experience, we anticipate typical column and floor loading for structures of this type, on the order of 120-to-150-kip column loads and 250 to 300 pounds per square foot floor loads.

[1] HPA Architecture. "Conceptual Site Plan, 51st Ave. NE., Marysville, WA" dated 22 March 2024.

3.0 Field Explorations and Laboratory Testing

3.1 BORINGS AND MONITORING WELLS

Subsurface conditions at the site were evaluated by drilling eight borings, B-1 through B-8, to depths between 11.5 and 21.5 feet below existing ground surface (bgs). Two of the borings, B-7 and B-8, were completed with a groundwater monitoring well and instrumented with an automated pressure transducer. The approximate location of the borings and monitoring wells are shown in Figure 2. A detailed description of the field exploration program, log of explorations and monitoring well construction details are presented in Appendix A.

3.2 LABORATORY TESTING

Soil samples were obtained during drilling and taken to GeoEngineers, Inc's (GeoEngineers') laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content and grain size characteristics (sieve analysis). A description of the laboratory testing and the test results are presented in Appendix B.

3.3 CONE PENETRATION TESTING

GeoEngineers advanced three CPTs (CPT-1 to CPT-3) to depths between 40 to 100 feet at the site. Within one of the CPTs, shear wave velocity was measured (CPT-2). The locations of the CPT explorations are shown in Figure 2. The results of the CPTs are included in Appendix C.

4.0 Site Conditions

4.1 GEOLOGY

The site is located in southern Arlington in the nearly level, heavily farmed valley between the Port Susan Inlet and the foothills of the Cascade Mountains. Surface geology along the I-5 corridor and the Marysville valley was mapped by Minard (1985); an excerpt of this mapping is presented in Figure 3. Geologic units deposited during the Quaternary Vashon stade of the Vashon glaciation include recessional outwash deposits (Qvrm), glacial till (Qvt), advance outwash (Qva) and older units including Holocene alluvial deposits (Qoal). The glacial till, advance outwash and older alluvium are mapped east of the site along the valley wall, and the recessional outwash deposits are mapped across the improvement areas within the valley. The recessional outwash deposits, better known locally as the Marysville Sand Member, typically consist of stratified outwash sand with some gravel, and some areas of silt and clay. The sediments were deposited by meltwater from the stagnating and receding Vashon glacier and are typically medium dense/stiff.

Geologic descriptions of the area are consistent with the subsurface conditions observed in our borings and cone penetration tests. Additional details of the subsurface exploration program are described below.

4.2 SURFACE CONDITIONS

The site is bounded by Longhouse Trail Lane NE and The Outpost Apartments on the north, 51st Avenue NE on the east, Emerald Springs RV Park on the south, and the future 47th Avenue NE on the west. The site is relatively flat, with existing site grades ranging from 119 to 117 feet (North American Vertical Datum of 1988, NAVD 88). The site has historically been used for agricultural purposes and is undeveloped. Overgrown grass and shrubs were present on the site during the time of site investigation, particularly on the east side of the property.

4.3 SUBSURFACE CONDITIONS

Subsurface soil and groundwater conditions were evaluated by completing three CPTs (CPT-1 through CPT-3) and eight boring/monitoring wells (B-1 through B-8). Based on the explorations and our review of previous explorations in the vicinity of the site, subsurface conditions generally consist of a reworked agricultural layer (topsoil) overlying recessional outwash deposits (Qvrm). The recessional outwash deposits are generally composed of medium dense sands that become dense at depths between 5 and 20 feet bgs. At a depth of approximately 35 feet, the recessional outwash deposits are generally medium dense and have increased silt content.

Groundwater was encountered in the borings at depths between 5.8 to 6.7 feet bgs at the time of drilling. Groundwater levels observed at the time of drilling can be inaccurate due to the limited time the drilled hole is left open and insufficient time for groundwater levels to stabilize. Groundwater was measured within the two monitoring wells on June 28, 2024, and again during the rainy season on November 15, 2024. Groundwater was observed at depths between 1.4 and 3.8 feet bgs, which correspond to Elevations 116.4 to 114.4 feet. Pore pressure dissipation testing (PPDT) performed at CPT-1 through CPT-3 indicated groundwater was present at a depths from 2.9 to 3.4 feet bgs. Groundwater levels are summarized in Table 1.

TABLE 1. GROUNDWATER MONITORING DATA

WELL ID	GROUND SURFACE ELEVATION[1] (FEET)	TOP OF WELL CASING ELEVATION[1] (FEET)	WELL SCREEN ELEVATION RANGE[1] (FEET)	DEPTH TO GROUNDWATER[2] (FEET)	GROUNDWATER ELEVATION[1] (FEET)	DATE OF GROUNDWATER MEASUREMENT
B-7	118.5	118.2	113.5 to 103.5	3,8	114.4	6/28/2024
				2.0	116.2	11/15/2024
B-8	118.0	117.7	113 to 103	3.0	114.7	6/28/2024
				1.3	116.4	11/15/2024

Notes:

[1] Elevations refer to NAVD 88 datum.

[2] Groundwater depths are measured from top of casing.

[3] Reference elevations surveyed in the field.

Based on our measurements at the site and seasonal groundwater readings from nearby, we recommend a design groundwater level of 2 feet below existing site grades. We will continue to monitor groundwater throughout the rainy season and will prepare updated recommendations as necessary. Groundwater conditions should be expected to fluctuate as a function of season, precipitation and other factors.

5.0 Conclusions and Recommendations

A summary of primary geotechnical considerations for the proposed improvements is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site soils are susceptible to seismically induced liquefaction. We estimate that liquefaction settlement could be in the range of 1 to 2 inches during the design earthquake.
- We estimated the shear wave velocity of the upper 30 meters by evaluating the shear wave velocity measurements from CPT-2. The results of these measurements indicate a ‘Stiff Soil’ site (Site Class D). However, as noted above, we conclude there is potential for liquefaction, resulting in a Site Class F designation. As discussed in a following section, seismic design parameters for Site Class D are appropriate if the fundamental period of vibration for the planned structure is less than 0.5 seconds.
- Shallow foundations can be constructed on a pad of structural fill as recommended in this report. An allowable soil bearing pressure of 3,000 pounds per square foot (psf) may be used where a minimum 2-foot thickness of structural fill underlies the footing.
- We estimate post-construction static settlement of foundation, supported as recommended above, of less than 1 inch. Foundations will need to consider the effects of liquefaction for design, which will include Site Class F and corresponding ASCE 7-16 Chapter 12 requirements, total liquefaction settlement on the order of 1 to 2 inches with differential settlement of up to 1 inch, and reduced bearing capacity during seismic conditions. The slab should be expected to settle and crack following a major earthquake.
- We recommend that all new fill placed within the building footprint be compacted to 95 percent of the maximum dry density (MDD) (ASTM International [ASTM] D1557). Following clearing and grubbing of the topsoil layer, the exposed recessional outwash deposits should be compacted to a firm and unyielding condition. Areas of loose, saturated soils may require additional excavation and replacement with structural fill depending on the subgrade conditions and planned fill height in the area. The geotechnical engineer should observe and evaluate the suitability of the subgrade soils prior to placing structural fill.
- The on-site surficial sands are moisture sensitive and will likely not be re-usable as structural fill. Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including debris, shrubs, trees and associated roots or stumps. Graded areas should be stripped of organic materials and topsoil prior to placing structural fill.

Detailed geotechnical recommendations for foundation support and other aspects of project development are presented in the following sections.

5.1 GEOLOGIC HAZARD EVALUATION

5.1.1 2021 IBC Design Parameters

Shear wave velocity (Vs) testing was performed in conjunction with the piezocone penetration test at CPT-2 in order to collect internal shear wave velocities. The results from CPT-2 indicate a V_{S30} (time-averaged shear wave velocity in the upper 30 meters) of 839 feet per second, and a ‘Stiff Soil’ site (Site Class D).

However, as described under the "Liquefaction" subsection, the site has a potential for liquefaction, which results in a Site Class F designation.

The 2021 IBC references the 2016 version of *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16). The medium dense granular soils below the water have moderate liquefaction potential, and therefore, the site is designated as Site Class F. However, if the proposed building will have a fundamental period of vibration equal to or less than 0.5 seconds, site response analysis is not required, and the site parameters can be determined based on Site Class D. Please note that the Site Class F designation and associated requirements of ASCE 7-16 Chapter 12 still apply. The values presented below assume that the proposed building will have a fundamental period of vibration equal to or less than 0.5 seconds.

We recommend the use of the following 2021 IBC parameters for short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1) and seismic coefficients (F_A and F_V) presented in Table 2 for the project site.

TABLE 2. 2021 IBC PARAMETERS

2021 IBC PARAMETER[1]	RECOMMENDED VALUE
Site Class	D
Mapped MCE_R spectral response acceleration at short period, S_S (g)	1.063
Mapped MCE_R spectral response acceleration at 1-second period, S_1 (g)	0.379
Seismic Coefficient, F_A	1.075
Seismic Coefficient, F_V	1.921[2]
MCE_R spectral response acceleration at short period adjusted or site class effects, S_{MS} (g)	1.143
MCE_R spectral response acceleration at 1-second period adjusted or site class effects, S_{M1} (g)	0.728
Design spectral response acceleration at short period adjusted or site class effects, S_{DS} (g)	0.762
Design spectral response acceleration at 1-second period adjusted or site class effects, S_{D1} (g)	0.485
Site-Modified Peak Ground Acceleration (g)	0.518

Notes:

1. Parameters developed based on latitude 48.146061 and longitude -122.164392 using the ASCE 7 Hazards online tool (https://asce7hazardtool.online/)
2. These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

5.1.2 Seismic Hazards

We evaluated the site for seismic hazards, including surface fault rupture, liquefaction and lateral spreading. These hazards are discussed in the subsections below.

5.1.3 Fault Rupture

Based on USGS maps of active faults in the Puget Sound region, the site is located more than 10 miles from the Darrington Devils Mountain Fault Zone and more than 10 miles from the South Whidbey Island Fault Zone. Because of the thickness of Quaternary sediments below the site, which are commonly more than 1,000 feet thick in the region, and lack of fault displacement evidence in the project area, the potential for surface fault rupture is considered low.

5.1.4 Liquefaction

Liquefaction refers to the condition when vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table. Liquefaction usually results in ground settlement and loss of bearing capacity, resulting in settlement of structures that are supported on foundations that are constructed within or above the liquefied soils.

The results of our analyses indicate that the medium dense recessional outwash deposits below the groundwater table have a high potential for liquefaction during the design earthquake event.

The evaluation of liquefaction potential is a complex procedure and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stress and the design ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic stress ratio (CSR), which is the ratio of the cyclic shear stress induced by an earthquake, to the initial effective overburden stress, to the cyclic resistance ratio (CRR), which is the soils resistance to liquefaction. The factor of safety against liquefaction is determined by dividing the CRR by the CSR. Liquefaction hazards, including settlement and related effects, were evaluated when the factor of safety against liquefaction was calculated as less than 1.0. We evaluated the liquefaction triggering potential (Youd et al. 2001; Idriss and Boulanger 2014) and liquefaction-induced settlement (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992; Idriss and Boulanger 2014) for soil conditions at the CPT exploration locations we completed at the site. We applied the depth weighting scheme proposed by Cetin et al. (2009) to the results of the liquefaction-induced settlement analysis.

Our liquefaction analyses indicate that the site could experience liquefaction-induced ground settlement of the potentially liquefiable zones on the order of 1 to 2 inches during an MCE seismic event. The magnitude of liquefaction-induced ground settlement will vary as a function of the characteristics of the earthquake (earthquake magnitude, location, duration and intensity) and the soil and groundwater conditions.

5.1.5 Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soils are displaced relative to adjacent blocks. Lateral spreading also occurs as blocks of surface soils are displaced toward a nearby slope or free-face by movement of the underlying liquefied soil. The risk of lateral spreading is a function of the soil conditions, distance away from the free-face and the condition of the slope. Based on the site topography, in our opinion the risk of damage to the building from lateral spreading is low and therefore does not need to be considered in the foundation design.

5.2 FOUNDATION SUPPORT

5.2.1 Shallow Foundations

We recommend that shallow foundations be founded on a minimum 2-foot-thick pad of properly compacted structural fill. The zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill or a minimum of 3 feet where possible. We recommend that the existing agricultural topsoil be removed and that the exposed surface (recessional outwash deposits) be recompacted to a dense condition. Additional site preparation and subgrade preparation recommendations are provided in the Earthwork section of this report. A representative from our firm should observe the subgrade of new footing construction prior to placement of structural fill.

Exterior footings should be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below slab subgrade. Continuous wall footings should have minimum widths of 16 inches, and column footings should have a minimum width of 24 inches. All loose or disturbed soil must be removed prior to placing the structural fill. Footings bearing on a minimum of 2 feet of structural fill may be designed using an allowable soil bearing pressure of 3,000 psf for dead plus long-term live loads . The allowable soil bearing pressures are net values. The weight of the footing and any backfill over the footing may be neglected in determining the applied bearing pressure. This value includes a factor of safety of 3 and may be increased up to one-third for short-term live loads such as wind or seismic.

We estimate post-construction static settlement of footings, designed and supported as recommended above, of less than 1 inch. As noted previously, we recommend that the zone of structural fill extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill where possible.

We estimate liquefaction-induced settlements in the range of 1 to 2 inches for the design earthquake. Differential settlement between columns is estimated to be half the total settlement, or on the order of 1 inch. As discussed previously, we also anticipate that there may be reduced bearing capacity due to liquefaction. We recommend that the structural engineer evaluate the impact of the estimated liquefaction induced settlement on the integrity of the building structural components.

5.2.2 Lateral Resistance

The soil resistance available to resist lateral loads is a function of the frictional resistance which can develop on the base of footings and floor slabs, and the passive resistance which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings and floor slabs founded on structural fill placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces. Passive pressures may be computed using an equivalent fluid density of 300 pcf if all soil extending out from the face of the foundation element for a distance at least equal to two and one-half times the depth of the element consists of structural fill compacted to at least 95 percent of MDD (ASTM D1557). The passive pressure should be reduced to 160 pcf if any foundation elements are located below the groundwater. The above coefficient of friction and passive equivalent fluid density values include a factor of safety of about 1.5.

5.3 FLOOR SLAB SUPPORT

All slab subgrade areas should be stripped and proofrolled or otherwise evaluated as recommended in the “Earthwork” section of the report, before placing any fill. We recommend that the upper 4 inches of fill placed to form the building pad consist of capillary break material, 1-inch minus clean crushed gravel with negligible sand and silt, WSDOT 9-03.1(4)C. Grading No. 67. If fill placement and slab construction occurs during extended periods of dry weather, the imported fill placed below the upper 4 inches of slab subgrade may contain an increased percentage of fines, provided the fill can be compacted to the structural fill criteria as recommended in this report.

For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 75 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended above. This value assumes the slabs are bearing directly on structural fill and will require evaluation during construction.

If water vapor migration through the slabs is objectionable, the capillary break gravel layer should be covered with heavy plastic sheeting at least 10-mil thick to act as a vapor retarder. This will be desirable where the slabs are in occupied spaces or will be surfaced with tile or will be carpeted. It may also be prudent to apply a sealer to the slab to further retard the migration of moisture through the floor. The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction. Additional water proofing measures that may be needed should be evaluated during design.

We recommend that additional underslab drainage measures be evaluated during final design to intercept groundwater on site, if necessary, based on the final grading and development plans.

5.4 SITE EARTHWORK AND UTILITY CONSTRUCTION

Utility construction in these soils will involve typical construction practices. Dewatering will be necessary for excavations that extend below groundwater level. Because of the risk of liquefaction settlement during a design earthquake, it may be prudent to provide positive, restrained joint connections on below grade piping systems. The soils are moisture sensitive and re-use of these soils as structural fill may be difficult, especially if work takes place during the wet winter months. Temporary dewatering consisting of conventional sumps and pumps, or vacuum dewatering wells may be needed for excavations for deep utilities or vaults.

The following sections provide general recommendations for site earthwork.

5.4.1 On-Site Soils and Wet Weather Considerations

The on-site soils are moisture sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, operation of equipment on these soils will be difficult, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. The contractor will need to take precautions to protect the subgrade during periods of wet weather.

The wet weather season in western Washington generally begins in October and continues through May; however, periods of wet weather may occur during any month of the year. The optimum earthwork period for these types of soils is typically June through September. If wet weather earthwork is unavoidable, we recommend that:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area;
- Erosion control techniques should be implemented to prevent sediment from leaving the site;

- Earthwork activities should not take place during periods of heavy precipitation;
- Slopes with exposed soils should be covered with plastic sheeting;
- The contractor should take necessary measures to prevent on-site soils and soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent that these soils become wet or unstable; and
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.

5.4.2 Clearing and Site Preparation

Construction of the planned development will require stripping of the organic topsoil and debris prior to placing structural fill for the building pad. Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including debris, shrubs, trees and associated stumps and roots. Graded areas should be stripped of organic materials and topsoil.

The stripped organic soils can be stockpiled and used later for landscaping purposes. If used in landscaping areas, we recommend they be placed in a layer less than 1-foot thick, should not be placed on slopes greater than 3H:1V and should be track-rolled to a uniformly compacted condition. Materials that cannot be used for landscaping should be removed from the project site.

5.4.3 Subgrade Preparation

Following site clearing and prior to placing new fill, subgrade areas should be proofrolled during dry weather to locate any soft or pumping soils. Prior to proofrolling, all unsuitable soils should be removed from below building footprints and new hardscape areas. Proofrolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather or if wet subgrade areas are present, the exposed subgrade areas should be probed to determine the extent of soft soils. If soft or pumping soils are observed, they should be removed and replaced with structural fill.

After proofrolling, subgrade areas should be recompacted to a firm and unyielding condition, if possible. The degree of compaction that can be achieved will depend on subgrade conditions and when construction is performed. If the work is performed during dry weather conditions, we recommend that all subgrade areas be recompacted to at least 95 percent of the MDD in accordance with the ASTM D1557 test procedure (modified Proctor). If the work is performed during wet weather conditions, it may not be possible to recompact the subgrade to 95 percent of the MDD. In this case, we recommend that the subgrade be compacted to the extent possible without causing undue weaving or pumping of the subgrade soils.

Subgrade disturbance or deterioration could occur if the subgrade is wet and cannot be dried. If the subgrade deteriorates during proofrolling or compaction, it may become necessary to modify the proofrolling or compaction criteria or methods.

5.4.3.1 SUBGRADE PROTECTION

Surficial site soils contain significant fines content (silt/clay) and will be highly sensitive and susceptible to moisture and equipment loads. The contractor should take necessary measures to prevent site subgrade soils from becoming disturbed or unstable. Construction traffic during the wet season should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance.

5.4.4 Structural Fill

Materials placed to support foundations and floor slabs and placed below parking areas, driveways and sidewalks should be placed as structural fill. Structural fill material quality varies depending upon its use as described below:

- Structural fill placed to support foundations, floor slabs, or driveway, parking and sidewalk areas should meet the requirements of gravel borrow, Washington State Department of Transportation (WSDOT) gravel borrow, WSDOT Standard Specification 9-03.14(1), with the added restriction that the fines content (material passing the US No. 200 sieve) not exceed 5 percent during wet weather. Common borrow may also be suitable provided construction occurs during the dry season (typically June through September) and the imported soil is near the optimum moisture content required for compaction.
- We recommend that structural fill placed within the drainage zone of below-grade walls consist of WSDOT Gravel Backfill for Walls, WSDOT Standard Specification 9-03.12(2). Gravel Backfill for Drains, WSDOT Standard Specification 9-03.12(4) should be used for footing drains.
- Structural fill placed as capillary break material below the floor slab should meet the requirements of WSDOT Standard Specification 9-03.1(4)C, grading No. 57 (1-inch minus crushed rock).

5.4.5 Fill Placement and Compaction Criteria

Structural fill must be mechanically compacted to a firm, non-yielding condition. Structural fill must be placed in loose lifts not exceeding 12 inches in thickness. Each lift must be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill must be compacted to the following criteria:

- Structural fill placed to support foundations, floor slabs, or driveway, parking and sidewalk areas should be compacted to at least 95 percent of the MDD per ASTM D1557.
- Structural fill placed to backfill utility trenches should be compacted to between 90 and 92 percent of the MDD per ASTM D1557, except for the upper 2 feet that should be compacted to at least 95 percent of MDD.

5.4.6 Temporary Slopes

For planning purposes, temporary unsupported cuts more than 4 feet high may be inclined at 1½H:1V or flatter. Flatter slopes will be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs.

The above guidelines assume that surface loads such as traffic, construction equipment, stockpiles or building supplies will be kept away from the top of the cut slopes a sufficient distance so that the stability of the excavation is not affected. We recommend that this distance be at least 5 feet from the top of the cut for temporary cuts made at 1½H:1V or flatter.

Temporary cut slopes should be planned such that they do not encroach on a 1H:1V influence line projected down from the edges of nearby or planned foundation elements. If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions. Slopes experiencing problems can be flattened, regraded to add intermediate slope benches or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

5.4.7 Temporary Shoring

The installation of the new underground utilities may require some type of shoring system to support the temporary excavations and maintain the integrity of the surrounding undisturbed soils, and to protect the personnel working within the excavations.

Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. The following paragraphs present recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project site.

The soils within the project area can be retained using conventional trench shoring systems such as trench boxes. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, surcharge loads from traffic, construction equipment and temporary stockpiles adjacent to the excavation, etc. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic or fixed braces.

Temporary trench shoring using internal bracing can be designed using active soil pressures. We recommend that temporary shoring be designed using a lateral pressure equal to an equivalent fluid density of 35 pcf, for conditions with horizontal backfill adjacent to the excavation. If the ground within 5 feet of the excavation rises at an inclination of 2H:1V or steeper, the shoring should be designed using an equivalent fluid density of 60 pcf. For adjacent slopes flatter than 2H:1V, soil pressures can be interpolated between this range of values. Additional shoring parameters will be provided on a case-by-case basis as the design progresses.

These lateral soil pressures do not include traffic or construction surcharges, which should be added separately, if appropriate. Shoring should be designed for a traffic influence equal to a uniform lateral pressure of 100 psf acting over a depth of 10 feet from the ground surface. More conservative pressure values should be used if the designer deems them appropriate. These soil pressure recommendations are predicated upon the construction being essentially dewatered; therefore, hydrostatic pressures are not included.

5.4.7.1 DEWATERING CONSIDERATIONS

Shallow groundwater is present at the site. We recommend dewatering design be evaluated on a case-by-case basis, depending on the depth, length and location of the utility. The outwash soils are highly permeable and will yield high groundwater inflow. As such, an active dewatering system consisting of wellpoints or individual wells will likely be necessary depending on the excavation depth and configuration.

5.5 BELOW-GRADE WALLS AND RETAINING WALLS

We understand that below-grade/dock-high retaining walls are planned for the structure. The following recommendations should be used in design of below-grade walls that are intended to act as retaining walls and for other retaining structures that are planned to accommodate grade transitions.

Lateral earth pressures for design of below-grade walls and retaining structures should be evaluated using an equivalent fluid density of 35 pounds per cubic foot (pcf). This assumes that the walls will not be restrained against rotation when backfill is placed. If the walls will be restrained from rotation, we recommend using an equivalent fluid density of 55 pcf. Walls are assumed to be restrained if top movement during backfilling is less than H/1000, where H is the wall height. These lateral soil pressures assume that the ground surface behind the wall is horizontal. The lateral soil pressures do not include the effects of surcharges such as floor loads, traffic loads or other surface loading. Surcharge effects should be considered as appropriate. Seismic earth pressures should also be considered in design using a rectangular distribution of 8H in psf, where H is the wall height.

In settlement-sensitive areas (e.g., beneath on-grade slabs), the upper 2 feet of backfill for subgrade walls should be compacted to at least 95 percent of the MDD determined in accordance with ASTM D1557. At other locations and below a depth of 2 feet, wall backfill should be compacted to between 90 and 92 percent of ASTM D1557. Measures should be taken to prevent overcompaction of the backfill behind the wall. This can be achieved by placing the zone of backfill located within 5 feet of the wall in lifts not exceeding 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor.

The recommended equivalent fluid density assumes a free-draining condition behind the wall. Wall drainage material may consist of Gravel Backfill for Drains per WSDOT Standard Specification Section 9-03.12(4) surrounded with a nonwoven geotextile filter fabric such as Mirafi 140N (or approved equivalent), or imported Gravel Borrow with less than 5 percent fines may be used in conjunction with a geocomposite wall drainage layer. The zone of wall drainage material should be 2 feet wide and should extend from the base of the wall to within 2 feet of the ground surface. The wall drainage material should be covered with a geotextile separator (such as Mirafi 140N) and then 2 feet of less permeable material, such as the on-site silty sand that is properly moisture conditioned and compacted.

A 4-inch-diameter perforated drain pipe should be installed within the free-draining material at the base of each wall. We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for the wall drain pipe. The footing drain recommended above can be incorporated into the bottom of the drainage zone and used for this purpose. If gravel borrow is used against the wall in conjunction with a geocomposite wall drainage layer, then the drainage pipe at the base of the wall should be surrounded with at least 12 inches of Gravel Backfill for Drains per WSDOT Standard Specification Section 9-03.12(4) that is wrapped with a nonwoven geotextile filter fabric such as Mirafi 140N (or approved equivalent).

The pipes should be laid with minimum slopes of one-quarter percent and discharge into the storm water collection system to convey the water off site. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush mounted access boxes. Collected downspout water should be routed to appropriate discharge points in separate pipe systems.

5.6 PAVEMENT CONSIDERATIONS

5.6.1 Subgrade Preparation

Pavement subgrade areas should be stripped and proofrolled or probed to evaluate the existing subgrade surface prior to placing the new pavement section. Where the existing soils are loose or wet and cannot be compacted, it will be necessary to excavate and replace these soils. The required excavation thickness will depend on the moisture content of the subgrade soils at the time of construction and should be evaluated at that time. To avoid the cost of additional overexcavation, the pavement subgrade preparation should occur during the dry season as practical.

5.6.2 Design Section

Based on our experience with similar developments, we recommend the following minimum pavement design sections as presented in Table 3. The heavier section should be utilized throughout the site if automobile parking areas cannot be strictly designated.

TABLE 3 - RECOMMENDED DESIGN PAVEMENT SECTIONS

PAVEMENT AREA	HMA CL. ½ PG 64-22[1] (INCHES)	CRUSHED SURFACING BASE COURSE WITH LESS THAN 5 PERCENT FINES CONTENT[2] (INCHES)
Automobile Parking	2	6
Entrance Drive and Heavier Truck Traffic	3	6

Notes:

[1] Hot mix asphalt (HMA) Class ½-inch, PG 64-22 per WSDOT Standard Specification 5-04 and 9-03. Minimum 2-inch thickness recommended.

[2] Crushed Surfacing per WSDOT Standard Specification 9-03.9(3) compacted to 95 percent of the MDD determined using ASTM D1557, to contain less than 5 percent fines content and to be placed on subgrade compacted to 95 percent of MDD.

5.7 PRELIMINARY INFILTRATION ASSESSMENT

We evaluated infiltration feasibility by performing grain size analysis testing and by evaluating the initial saturated infiltration rate (K_{sat}) using the Massmann et al. 2008 relationship presented in the *2024 Stormwater Management Manual for Western Washington* (SMMWW). We estimate an uncorrected K_{sat} value of 24 inches per hour (in/hr) for the surficial recessional outwash deposits above the observed groundwater level.

The design infiltration rate is determined by applying correction factors to the estimated initial infiltration rate. The correction factors account for uncertainties in site variability and number of tests, testing method and long-term reductions in permeability due to plugging from biological activity and accumulation of fines. Table V-5.1 of the 2024 SWMMWW describes the correction factors to be used to estimate design infiltration rates.

- CF_v accounts for site variability and number of locations tested. A CF_v of 0.33 was used based on the widely spaced exploration locations.
- CF_t is determined based on the test method. The CF_t for grain size method is 0.40.
- CF_m accounts for the degree of influent control to prevent siltation and bio-buildup and is 0.9.

The total correction factor CF_T for the grain size method is 0.119. When applied to the correlated initial infiltration rates of 24 in/hr, the design infiltration rate is approximately 2.8 in/hr.

Additional testing (both grain size analyses and pilot infiltration tests) should be completed when the location of planned facilities are known. Groundwater mounding analysis will also be required during final design based on the shallow depth to groundwater. We will continue to monitor groundwater levels at the site throughout the winter to obtain the seasonal high condition. Planned site grading will also impact the infiltration capacity and may benefit infiltration design. Based on the requirements of the SWMMWW, a minimum vertical separation of 3 feet is required between the bottom of the infiltration facility and the seasonal high condition.

5.8 PERMANENT SLOPES

We recommend that permanent cut and fill slopes be constructed no steeper than 2H:1V. To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose properly compacted fill. We recommend that the finished slope faces be compacted by track walking with the equipment running perpendicular to the slope contours so that the track grouser marks help provide an erosion-resistant slope texture.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may require localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, loose straw or excelsior or straw/coconut matting, should be used to protect the slopes during periods of rainfall.

5.9 DRAINAGE CONSIDERATIONS

We recommend that pavement surfaces be sloped so that surface drainage flows away from the building. We recommend that all roof drainage be collected in tight lines for diversion into the storm drain system. A perimeter footing drain is recommended to intercept surface water runoff that may be perched on the surficial silty sand soils. All areas should be graded to avoid concentration of runoff onto fill, cut slopes, natural slopes steeper than 10 percent or other erosion-sensitive areas.

5.10 RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should evaluate the suitability of the foundation subgrades; evaluate structural backfill; observe the condition of temporary cut slopes and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix D, Report Limitations and Guidelines for Use.

6.0 Limitations

We have prepared this report for the exclusive use of Rockefeller Group and members of the design team for the Northsound Logistics Center property in Arlington, Washington. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix D “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

7.0 References

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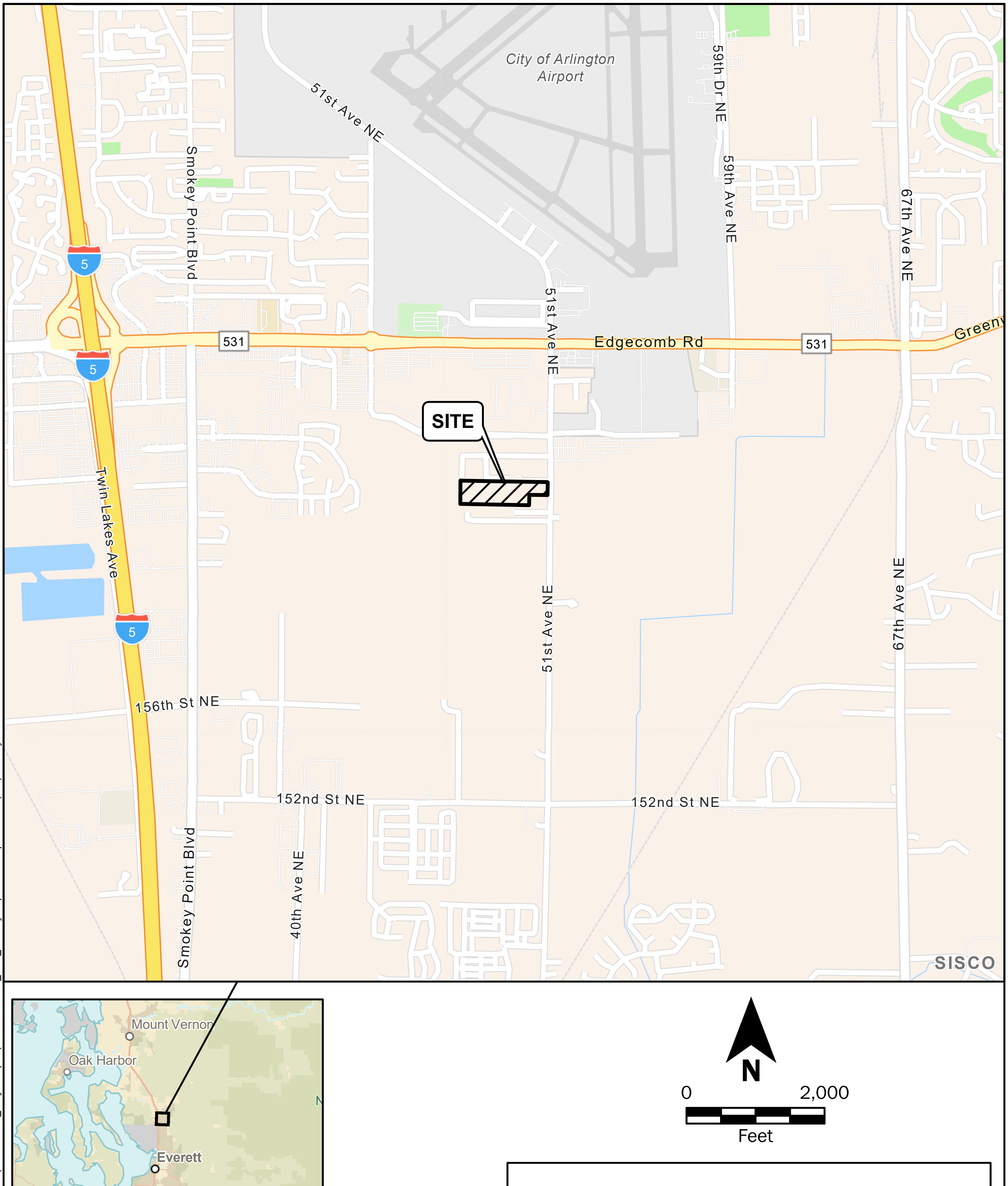
Figures



Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

Northsound Logistics Center
Arlington, Snohomish County, Washington

GeoEngineers

Figure 1

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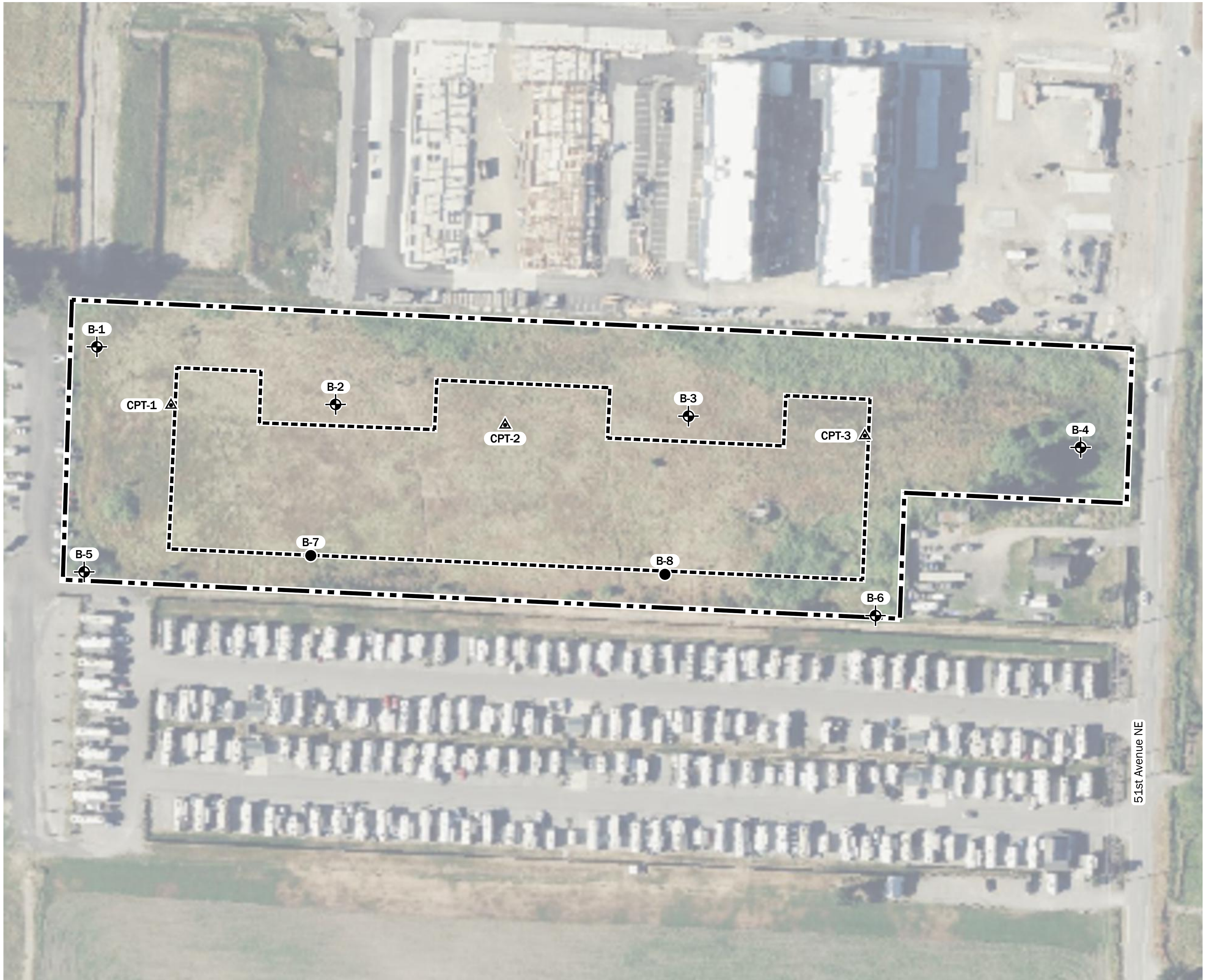

Legend

- Site Boundary
- Proposed Building
- CPT-1 Cone Penetration Test by GeoEngineers, 2024
- B-1 Boring by GeoEngineers, 2024
- B-7 Monitoring Well by GeoEngineers, 2024

Source(s):
- Aerial from Microsoft Bing
- "Conceptual Site Plan, 51st Ave. NE." by HPA Architecture, dated 22 March 2024

Coordinate System: WA State Plane, North Zone, NAD83, US Foot

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

N

0 120

Feet

Site Plan

Northsound Logistics Center
Arlington, Snohomish County, Washington

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Figure 2

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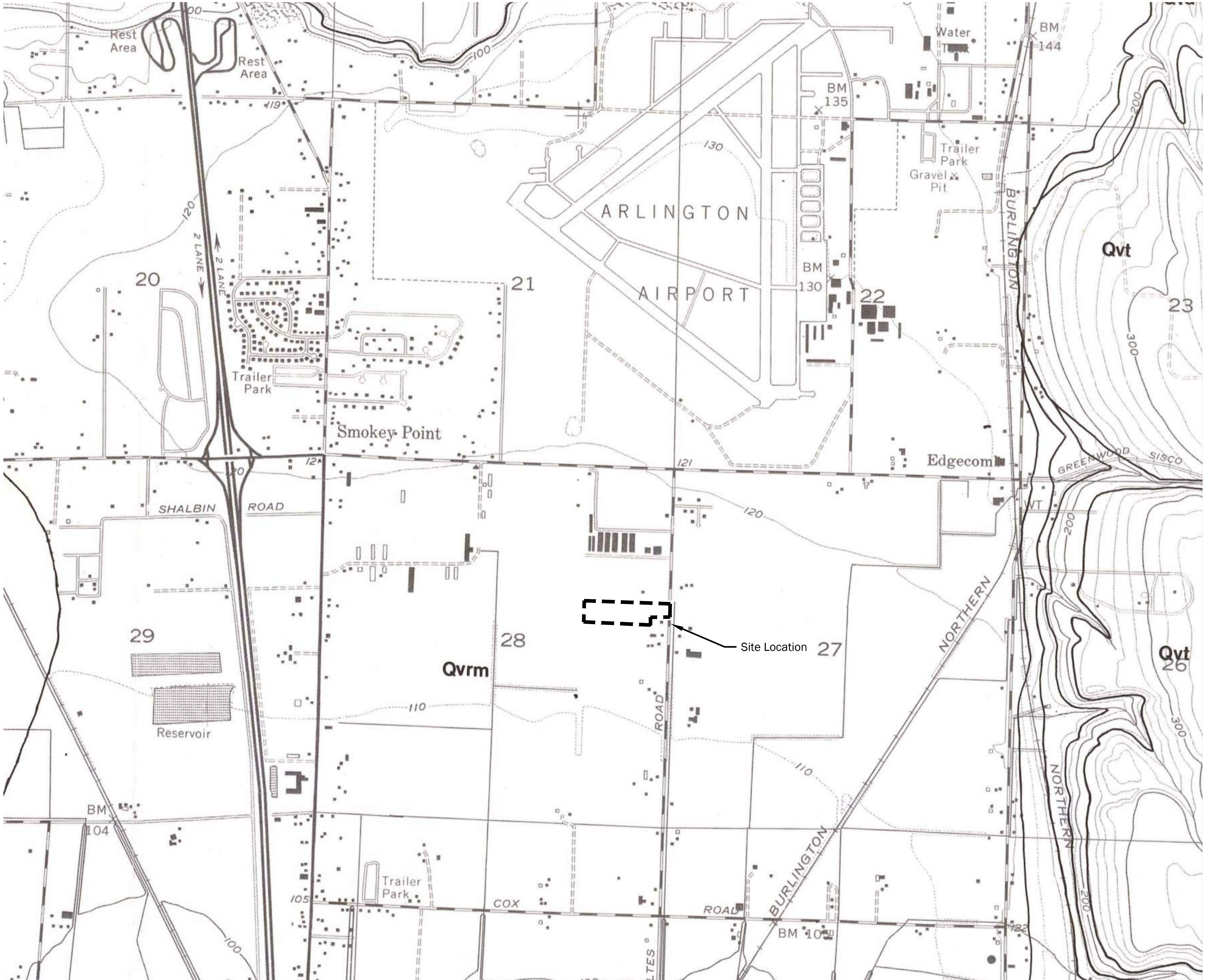

Legend

Qvrm — Marysville Sand Member - These deposits fill the broad, flat valley in the southern half of the quadrangle, consisting mostly of well-drained, stratified to massive outwash sand, some fine gravel, and some areas of silt and clay. The sediments were deposited by meltwater flowing south from the stagnating and receding Vashon glacier. Clast composition is varied and is similar to that of the undivided recessional outwash. The unit is bordered by Vashon till along the west and part of the east slopes of the valley. Till also probably underlies much of the main valley deposits. The member is at least 20 m thick and may be twice that. It is mostly farmed, but locally is used for fill. Within the member, in a pit in the south central part of the Section 10, township 31N, Range 5E, a unique diamicton can be seen that appears to be a lahar (volcanic mudflow), containing mostly dacitic rocks, probably derived from Glacier Peak (J. E. Beget, written commun., 1978)

Qvt — TILL - These deposits mantle hills, ridges, and slopes, and crop out in valley sides beneath younger recessional outwash and ablation deposits. The till (referred to locally as the Vashon till) consists of a non-sorted mixture of mud, sand, pebbles, cobbles, and boulders (diamicton), but includes some lenses of stratified material, particularly in the base. The unit is generally compact lodgement till, and often is referred to as hardpan. The hardness is largely a result of compaction by the great weight of the overriding ice, possibly 1000 m thick. The till was deposited directly by the ice as it advanced over bedrock and previously deposited sediment. Some calcite and silica cements are present. Mineralogically the till is similar to the underlying older deposits and to the younger overlying deposits because all were derived from similar sources and partly from each other. The till ranges from very sandy to quite clayey and it contains variable amounts of pebbles, cobbles, and boulders; this is, in large part, determined by constituents of the overridden materials. Internal drainage is greatly retarded by

Source(s):

- Geologic map of the Arlington West 7.5 minute quadrangle, Snohomish County, Washington" by Minard (1985), U.S. Geological Survey

Coordinate System: WA State Plane, North Zone, NAD83, US Foot

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

N

0 1500
Feet

Regional Geology Map

Northsound Logistics Center
Arlington, Snohomish County, Washington

GEOENGINEERS

Figure 3

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Appendices

Appendix A
Field Explorations

Appendix A
Field Explorations

Subsurface conditions at the site were explored on June 25 and 26, 2024, by drilling eight borings/monitoring wells (B-1 through B-8), shown in Figure 2 (Site Plan). The approximate exploration locations were established in the field by measuring distances from existing site features. Boring and monitoring well elevations were determined by assuming a project datum based on the base of the fire hydrant at 51st Avenue NE and Trailer Park Driveway. Exploration elevations should be updated to reference North American Vertical Datum of 1988 when the site survey has been completed. The explorations were completed to depths of about 11.5 to 21.5 feet using track-mounted equipment owned and operated by Holocene Drilling.

BORINGS

Disturbed soils samples were obtained during drilling using standard penetration test (SPT) methodology with the standard split-spoon sampler in the borings. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The borings were continuously monitored by an engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration. Soils encountered were classified visually in general accordance with ASTM D2488-09a the classification system described in Figure A-1. An explanation of our boring log symbols is also shown in Figure A-1.

The logs of the borings are presented in Figures A-2 through A-7 and the logs of the monitoring wells are presented in Figures A-8 and A-9. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

MONITORING WELLS

A representative of GeoEngineers, Inc. observed the installation of monitoring wells in borings B-7 and B-8. The monitoring wells were constructed using 2-inch-diameter polyvinyl chloride (PVC) casing. The depth to which the casing was installed was selected based on our understanding of subsurface soil and groundwater conditions in the project area. The lower portion of the casing is slotted to allow entry of water into the casing. Medium sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above and below the slotted portion of the casing. The monitoring well is protected with flush-mount steel monuments set in concrete. Completion details for the monitoring wells are shown on the logs presented in Appendix A.

The monitoring wells will require decommissioning by a licensed well driller prior to construction of the planned development. The decommissioning of the well will include backfilling the monitoring well and providing documentation of the decommissioning to the Washington State Department of Ecology (Ecology). The well installation log and Ecology registry information required for decommissioning and documentation are included on the boring log.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

Modified California Sampler (6-inch sleeve) or Dames & Moore

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 6/26/2024	End 6/26/2024	Total Depth (ft)	11.5	Logged By BA / Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	118.5 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) / Northing (Y)	1314869 / 421055	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Brown silty fine sand with occasional gravel and rootlets (medium dense, moist) (topsoil)			
						1						
115			7	14		2 / %F		SP-SM	Brown fine to medium sand with silt and occasional gravel and rootlets (medium dense, moist) (recessional outwash)	11	8	
	5		8	14		3		SP	Gray fine to coarse sand with occasional gravel (medium dense, wet)			
110			12	8		4 / %F			Becomes loose	21	4	Groundwater observed at approximately 6.90 feet from ground surface at time of drilling
	10		18	17		5			Becomes medium dense			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Boring B-1

GeoEngineers

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Drilled	Start 6/26/2024	End 6/26/2024	Total Depth (ft)	11.5	Logged By BA Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	119 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) Northing (Y)	1315153 420986	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Brown silty fine to medium sand with organic matter (medium dense, moist) (topsoil)			
						1						
			12	19		2 / %F		SP-SM	Gray fine to medium sand with silt and occasional rootlets (medium dense, moist) (recessional outwash)	19	9	
115												
	5		9	20		3 / SA		SP	Gray fine to coarse sand with gravel (medium dense, wet)	13	4	Groundwater observed at approximately 6.30 feet from ground surface at time of drilling
			10	12		4						
110												
	10		18	17		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Boring B-2

GeoEngineers

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Figure A-3
Sheet 1 of 1

Drilled	Start 6/26/2024	End 6/26/2024	Total Depth (ft) 11.5	Logged By BA / Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	118 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) / Northing (Y)	1315574 / 420972	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Dark brown silty fine to medium sand with occasional gravel and rootlets (medium dense, moist) (topsoil)			
						1						
115			12	18		2 / %F		SP-SM	Gray fine to coarse sand with silt and occasional gravel (medium dense, moist to wet) (recessional outwash)	14	6	
	5		12	15		3 / %F		SP	Gray fine to coarse sand with occasional gravel (medium dense, wet)	17	4	Groundwater observed at approximately 5.90 feet from ground surface at time of drilling
110			12	16		4						
	10		18	23		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Boring B-3

GEOENGINEERS

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Drilled	Start 6/26/2024	End 6/26/2024	Total Depth (ft) 11.5	Logged By BA Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	119 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) Northing (Y)	1316041 420935	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Brown silty fine to medium sand, rust stain color (medium dense, moist) (topsoil)			
						1 / %F				28	36	
			6	8		2 / %F		SP-SM	Gray silty fine to coarse sand with occasional gravel, rust stain color (loose, moist to wet) (recessional outwash)	18	8	
115												
	5		12	15		3			Becomes medium dense, wet			Groundwater observed at approximately 5.80 feet from ground surface at time of drilling
			16	14		4		SP	Gray fine to coarse sand with occasional gravel (medium dense, wet)			
110												
	10		15	12		5			Becomes fine to medium sand			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Date:11/19/24 Path:P:\20\20881002\GINT\2088100200.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring B-4

GeoEngineers

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Drilled	Start 6/25/2024	End 6/25/2024	Total Depth (ft) 11.5	Logged By BA / Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	117.5 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) / Northing (Y)	1314854 / 420787	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Dark brown silty fine sand with rootlets (medium dense, moist) (topsoil)			
						1						
115			6	19		2 %F		SP-SM	Gray fine sand with silt and occasional gravel (medium dense, moist) (recessional outwash)	20	10	
	5		7	14		3 %F				20	7	
110			12	13		4		SP	Gray fine to coarse sand with occasional gravel (medium dense, wet)			Groundwater observed at approximately 6.65 feet from ground surface at time of drilling
	10		18	28		5			Becomes fine to medium sand			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Boring B-5

GEOENGINEERS

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Figure A-6
Sheet 1 of 1

Drilled	Start 6/25/2024	End 6/25/2024	Total Depth (ft)	11.5	Logged By BA Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	117 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Easting (X) Northing (Y)	1315796 420735	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Brown silty fine to medium sand with occasional gravel with rootlets (medium dense, moist) (topsoil)			
115						1						
			9	13		2 %F		SP	Gray fine to coarse sand with occasional gravel (medium dense, moist to wet) (recessional outwash)	17	5	
	5		6	11		3			Becomes wet			
110												Groundwater observed at approximately 6.70 feet from ground surface at time of drilling
			18	18		4 SA				19	2	
	10		18	10		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Boring B-6

GEOENGINEERS

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Figure A-7
Sheet 1 of 1

Drilled	Start 6/25/2024	End 6/25/2024	Total Depth (ft) 21.5	Logged By BA	Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Surface Elevation (ft) / Vertical Datum	118.5 / NAVD88	Top of Casing Elevation (ft)	118.20
Easting (X) / Northing (Y)	1315124 / 420806	Horizontal Datum	WA State Plane North NAD83 (feet)

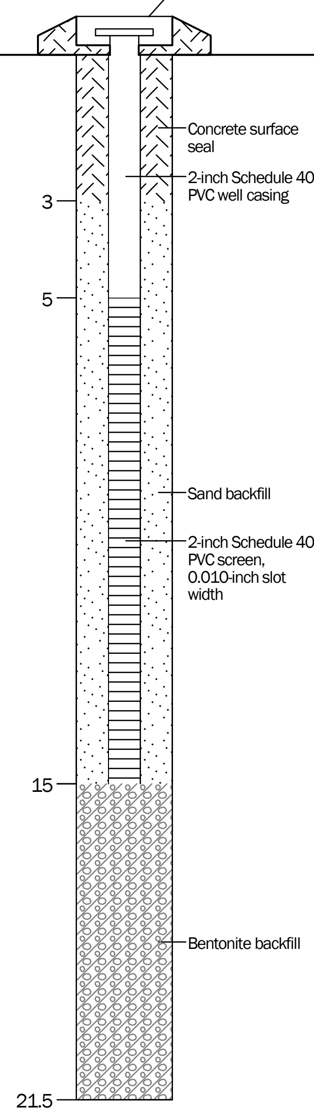
DOE Well I.D.: BQB 732
A 2-in well was installed on 6/26/2024 to a depth of 15 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
6/28/2024	3.8	114.4
11/15/2024	2.0	116.2

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	Material Description	Moisture Content (%)	Fines Content (%)
	0				ML	Brown sandy silt with organic matter and rootlets (soft, moist) (topsoil)		
				1				
115		12	15	2 %F	SM	Brown silty fine to medium sand, rust stain color (medium dense, moist) (recessional outwash)	23	12
	5	14	15	3	SP	Gray fine to coarse sand with gravel (medium dense, wet)		
110		10	14	4 SA			14	5
	10	14	16	5				
105					SP-SM	Gray fine to medium sand with silt and occasional gravel (medium dense, wet)		
	15	15	17	6 %F			18	5
100								
	20	18	45	7		Becomes dense		

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Monitoring Well B-7

GeoEngineers

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Figure A-8
Sheet 1 of 1

Drilled	Start 6/25/2024	End 6/25/2024	Total Depth (ft) 21.5	Logged By BA / Checked By RN	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Surface Elevation (ft) / Vertical Datum	118 / NAVD88	Top of Casing Elevation (ft)	117.70
Easting (X) / Northing (Y)	1315546 / 420784	Horizontal Datum	WA State Plane North NAD83 (feet)

DOE Well I.D.: BQB 733
A 2-in well was installed on 6/26/2024 to a depth of 15 ft.

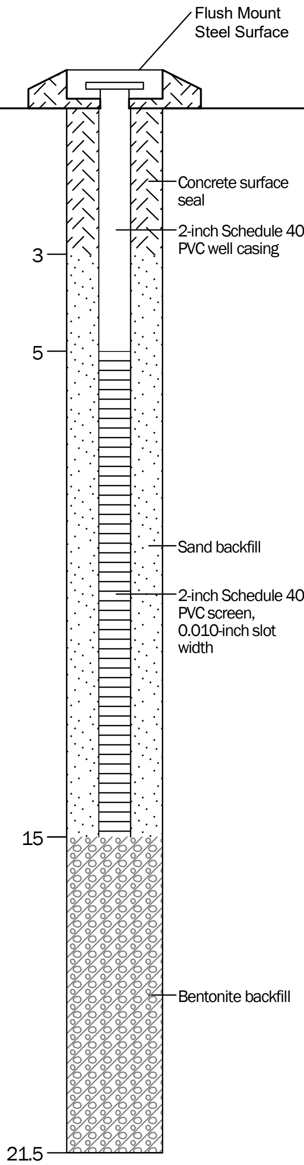
Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
6/28/2024	3.0	114.7
11/15/2024	1.3	116.4

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
118	0				SM	Brown silty fine to medium sand with rootlets (medium dense, moist) (topsoil)		
				1				
115		0*	45**	2 / SA	SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, moist) (recessional outwash)	11	12
	5	12	19	3 / %F	SP	Gray fine to coarse sand with gravel (medium dense, wet)	16	4
110		12	22	4		Becomes fine to medium sand		
	10	14	9	5				
105								
	15	18	55**	6 / %F			14	4
100					SP-SM	Gray fine to coarse sand with silt and occasional gravel (medium dense, wet)		
	20	18	18	7 / %F			18	9

WELL LOG



Note: *California sampler driven to collect sample
**Blowcount overstated

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on field measurements. Vertical approximated based on Assumed project datum with benchmark of fire hydrant base at 51st Ave NE and Trailer Park Driveway.

Log of Monitoring Well B-8

GeoEngineers

Project: Northsound Logistics Center
Project Location: Arlington, Snohomish County, Washington
Project Number: 20881-002-00

Appendix B
Laboratory Testing

Appendix B
Laboratory Testing

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, sieve analyses, hydrometer analysis and percent fines. The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

MOISTURE CONTENT TESTING

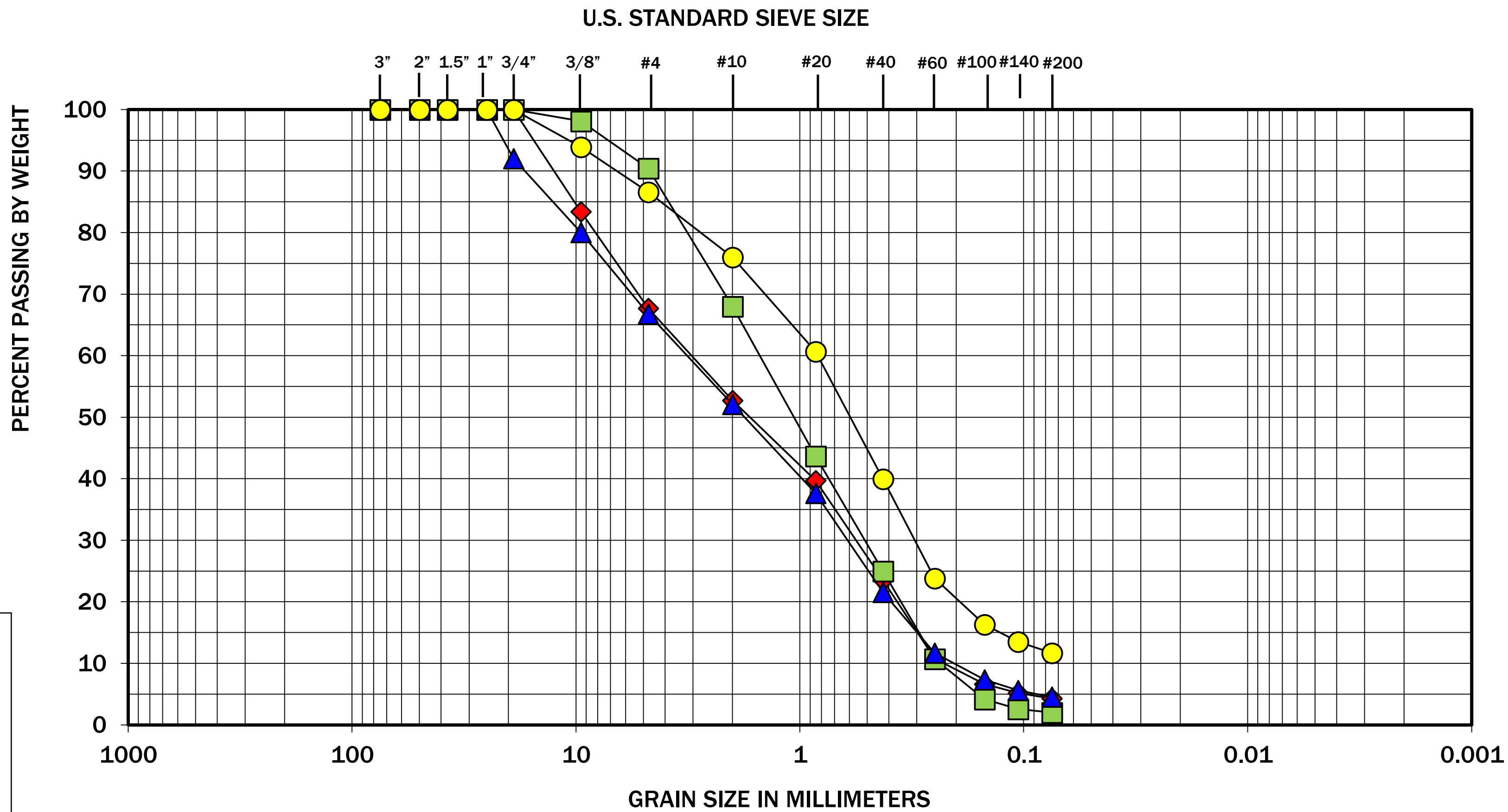
Moisture content tests were completed in general accordance with ASTM D2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs at the depths at which the samples were obtained.

SIEVE ANALYSES

Sieve analyses were performed on selected samples in general accordance with ASTM D422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS) and are presented in Figure B-1.

PERCENT FINES TEST

Percent fines (particles passing the No. 200 sieve) were completed on soil samples using ASTM D1140. The wet sieve method was used to determine the percentage of soil particles larger than the U.S. No. 200 sieve opening. The results of the percent fines tests are presented on the boring logs.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-2	5	13	Fine to coarse sand with gravel (SP)
■	B-6	7.5	19	Fine to coarse sand with occasional gravel (SP)
▲	B-7	7.5	14	Fine to coarse sand with gravel (SP)
●	B-8	2.5	11	Fine to medium sand with silt and occasional gravel (SP-SM)

AASHTO ACCREDITED

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM D6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

Sieve Analysis Results

Northsound Logistics Center

Arlington, Snohomish County, Washington

GEOENGINEERS

Figure B-1

Appendix C
Results of Cone Penetration Testing

PRESENTATION OF SITE INVESTIGATION RESULTS

51st Ave NE, Arlington, WA

Prepared for:

GeoEngineers

ConeTec Job No:	**24-59-27877**
Project Start Date:	2024-07-26
Project End Date:	2024-07-26
Release Date:	2024-07-29

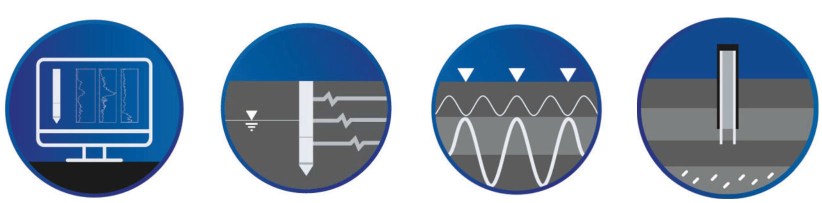
Report Prepared by:

ConeTec, Inc.

1237 S Director Street, Seattle, WA 98108

Tel: (253) 397-4861

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



CONETEC

ABOUT THIS REPORT

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

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	GeoEngineers
Project	51st Ave NE, Arlington, WA
ConeTec Project Number	24-59-27877
Test Types	CPTu/SCPTu
Additional Comments	None

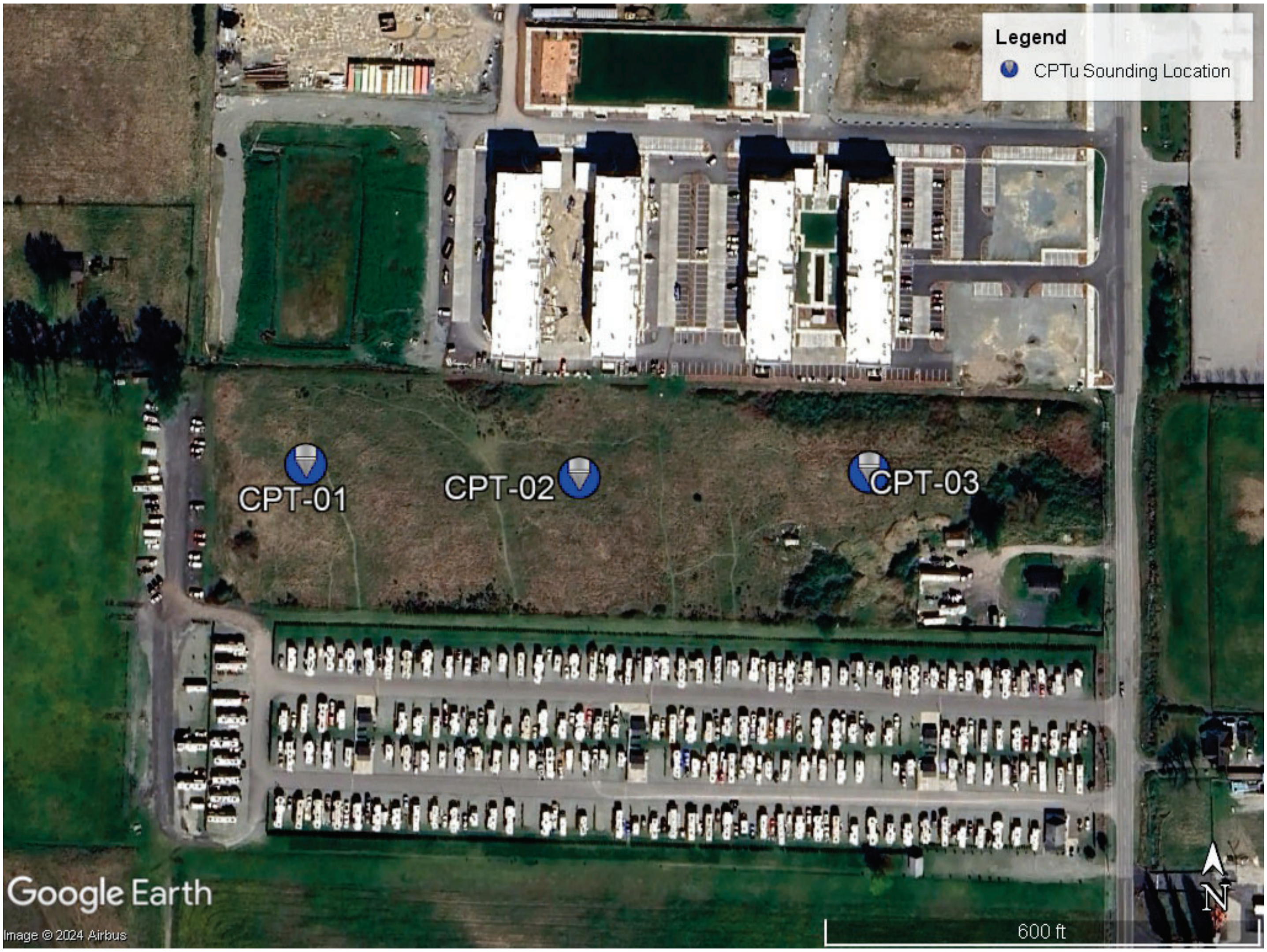
Contents

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**

ConeTec

SITE MAP



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

ConeTec Job Number: 24-59-27877

Client: GeoEngineers

Project: 51st Ave NE, Arlington, WA

Date: 2024-07-29

LIMITATIONS

3rd Party Disclaimer

The “Report” refers to this report titled: 51st Ave NE, Arlington, WA

The Report was prepared by ConeTec for: GeoEngineers

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: GeoEngineers

The “Report” refers to this report titled: 51st Ave NE, Arlington, WA

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 and Jesse Martinez

PROJECT INFORMATION

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

Coordinates		
Test Type	**Collection Method**	**EPSG Number**
CPTu	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**
- **Normalized CPTu Plots**
- **Advanced CPTu Plots with Ic, Su(Nkt), Phi, and N1(60)Ic**
- **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:	24-59-27877
Client:	GeoEngineers
Project:	51st Ave NE, Arlington, WA
Start Date:	2024-06-26
End Date:	2024-06-26

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	24-59-27877_CP01	2024-06-26	855:T1500F15U35	15	3.4	41.42		48.14635	-122.16648	
CPT-02	24-59-27877_SP02	2024-06-26	855:T1500F15U35	15	3.1	103.02	32	48.14630	-122.16490	
CPT-03	24-59-27877_CP03	2024-06-26	855:T1500F15U35	15	2.9	41.17		48.14632	-122.16323	
Totals	3 Soundings					185.61 ft	32			

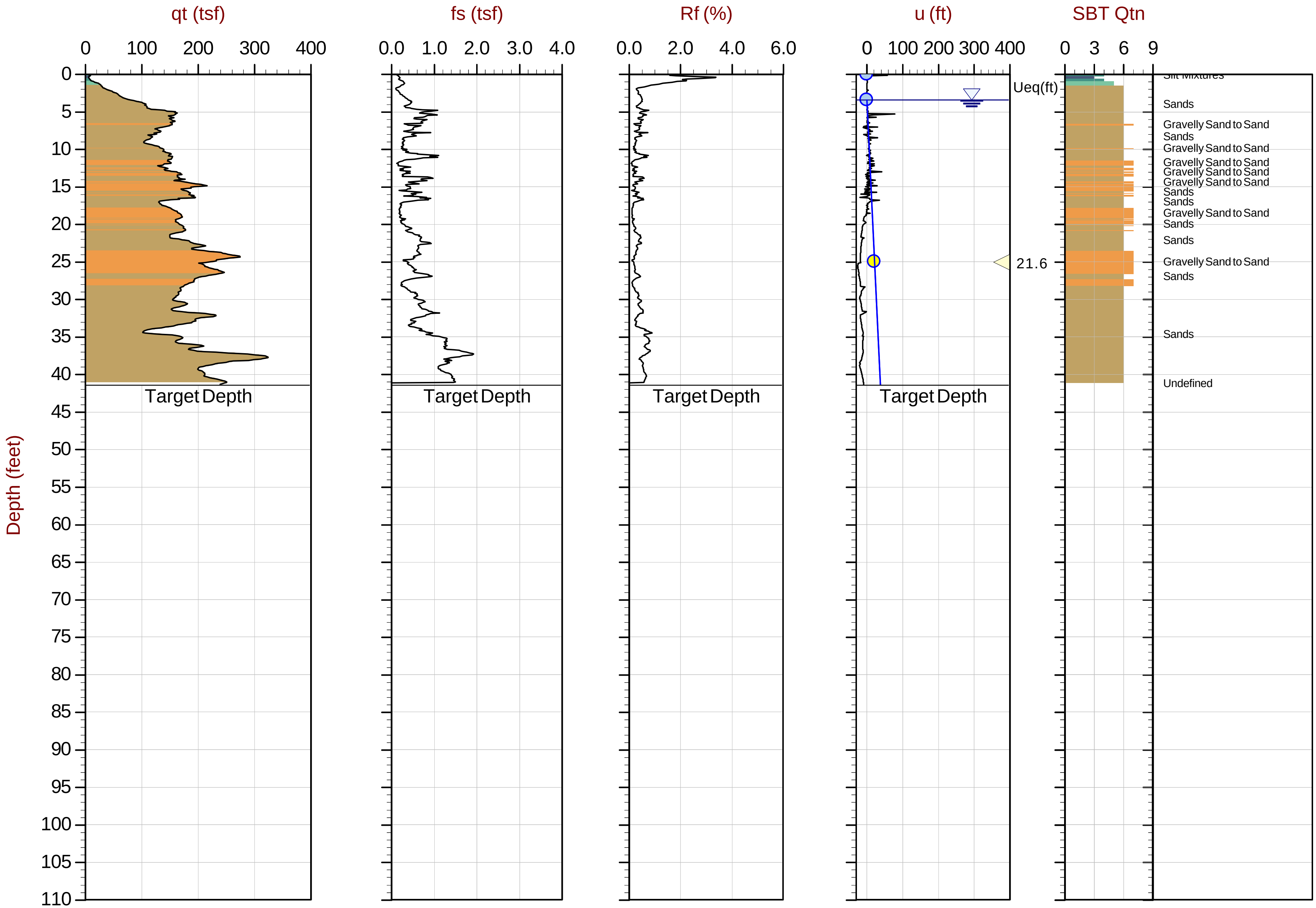
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:06
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 12.625 m / 41.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14635 Long: -122.16648

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

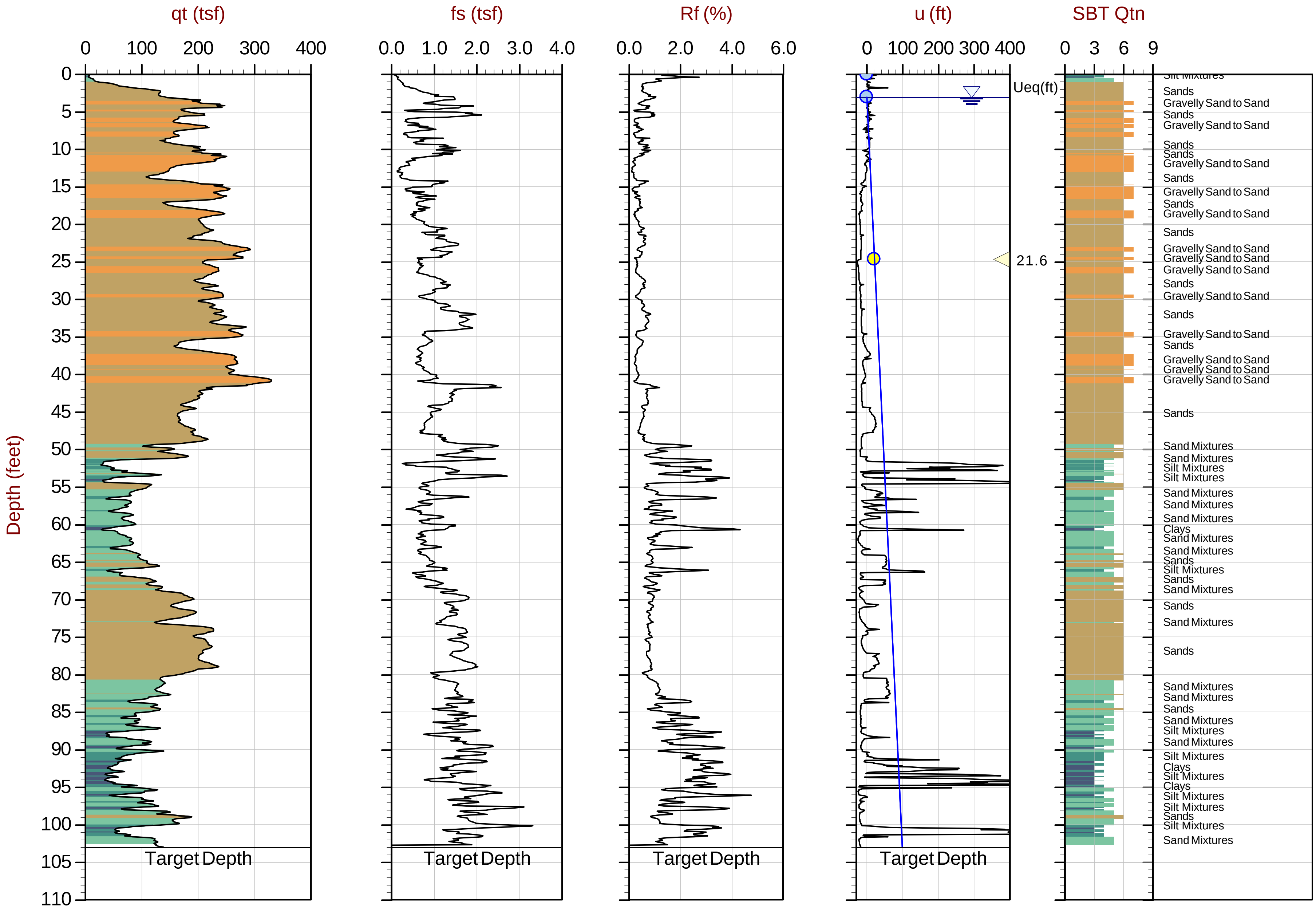
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 31.400 m / 103.02 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14630 Long: -122.16490

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

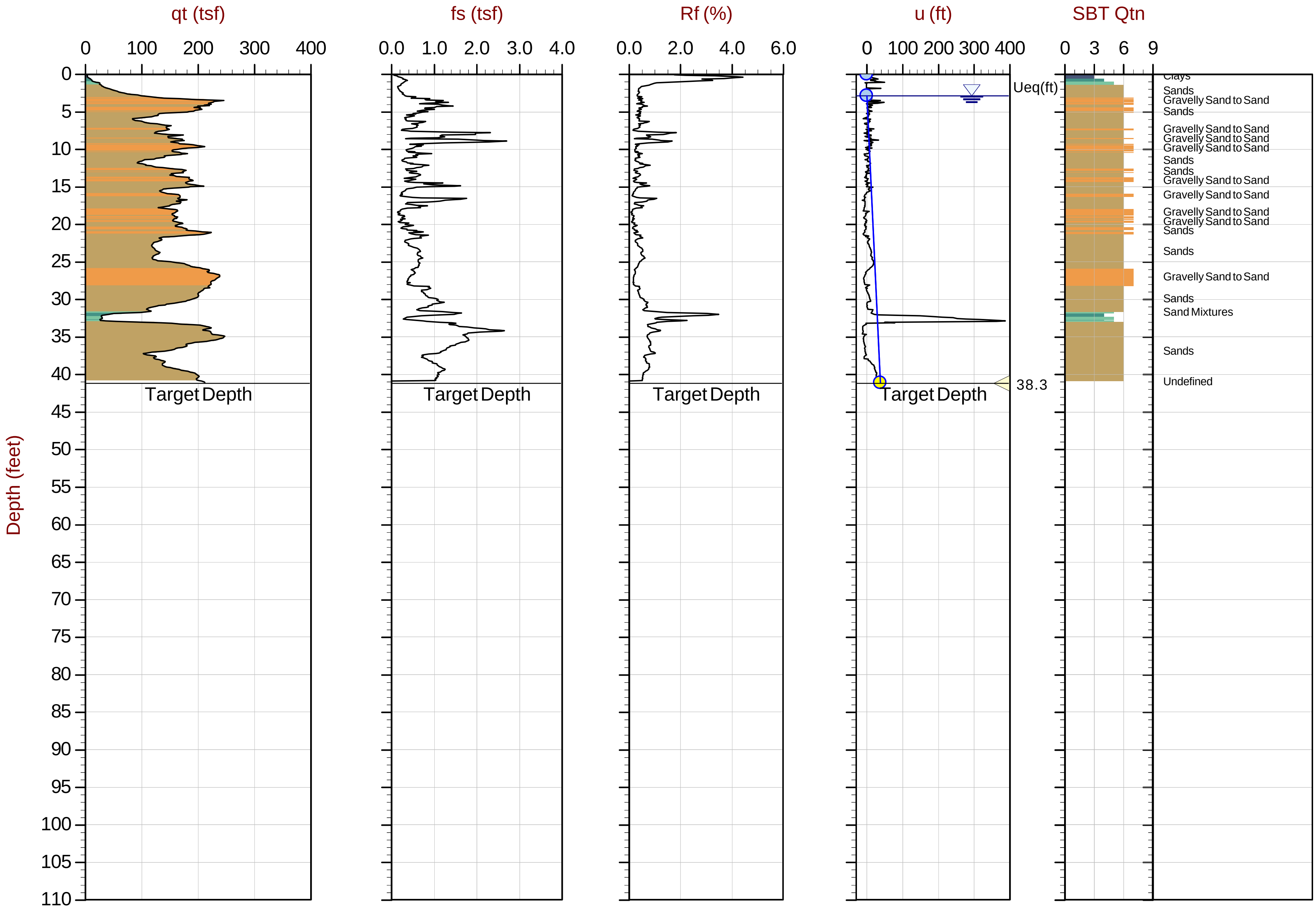
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 10:43
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 12.550 m / 41.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14632 Long: -122.16323

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

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

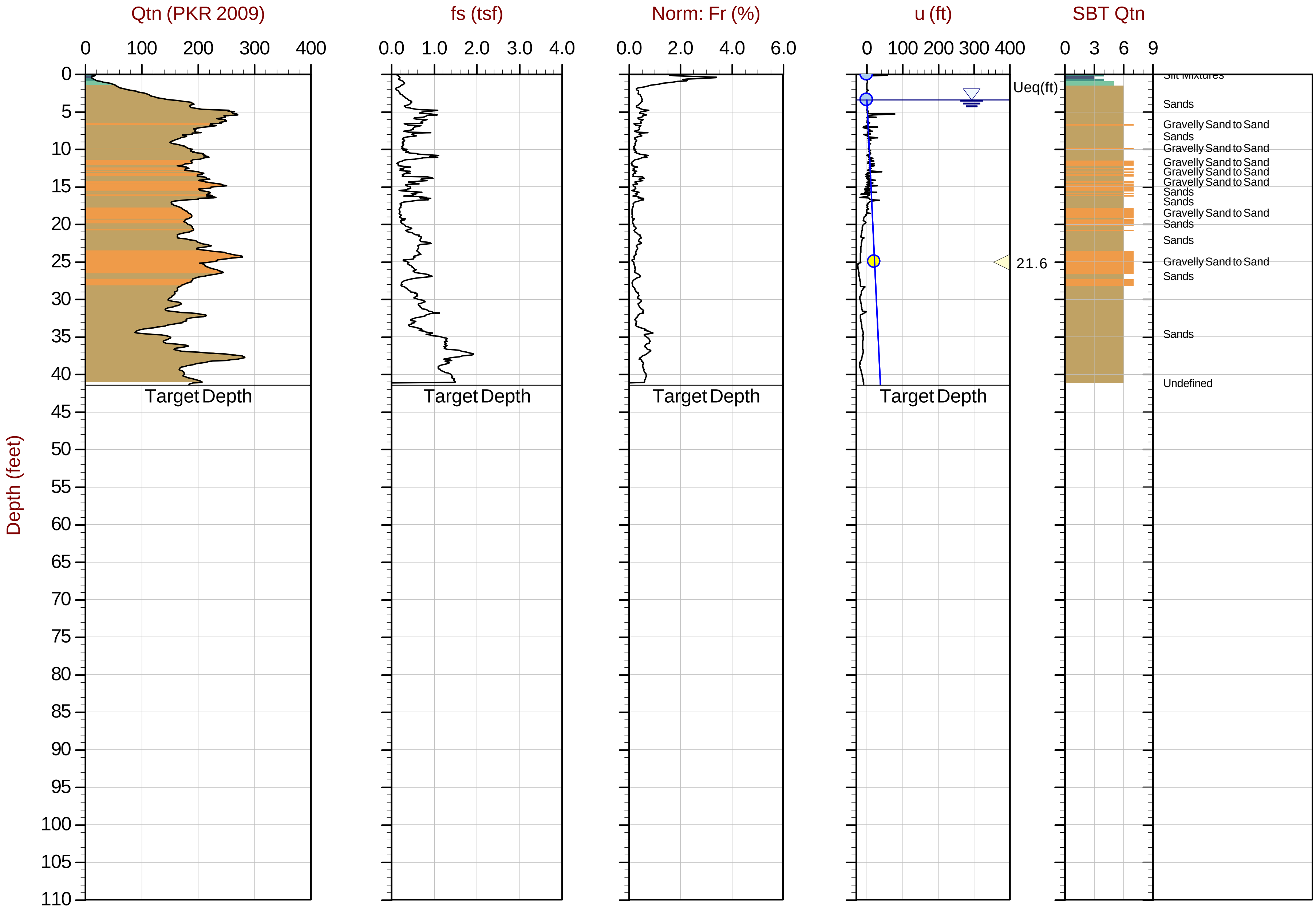
Normalized Cone Penetration Test Plots

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:06
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 12.625 m / 41.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14635 Long: -122.16648

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

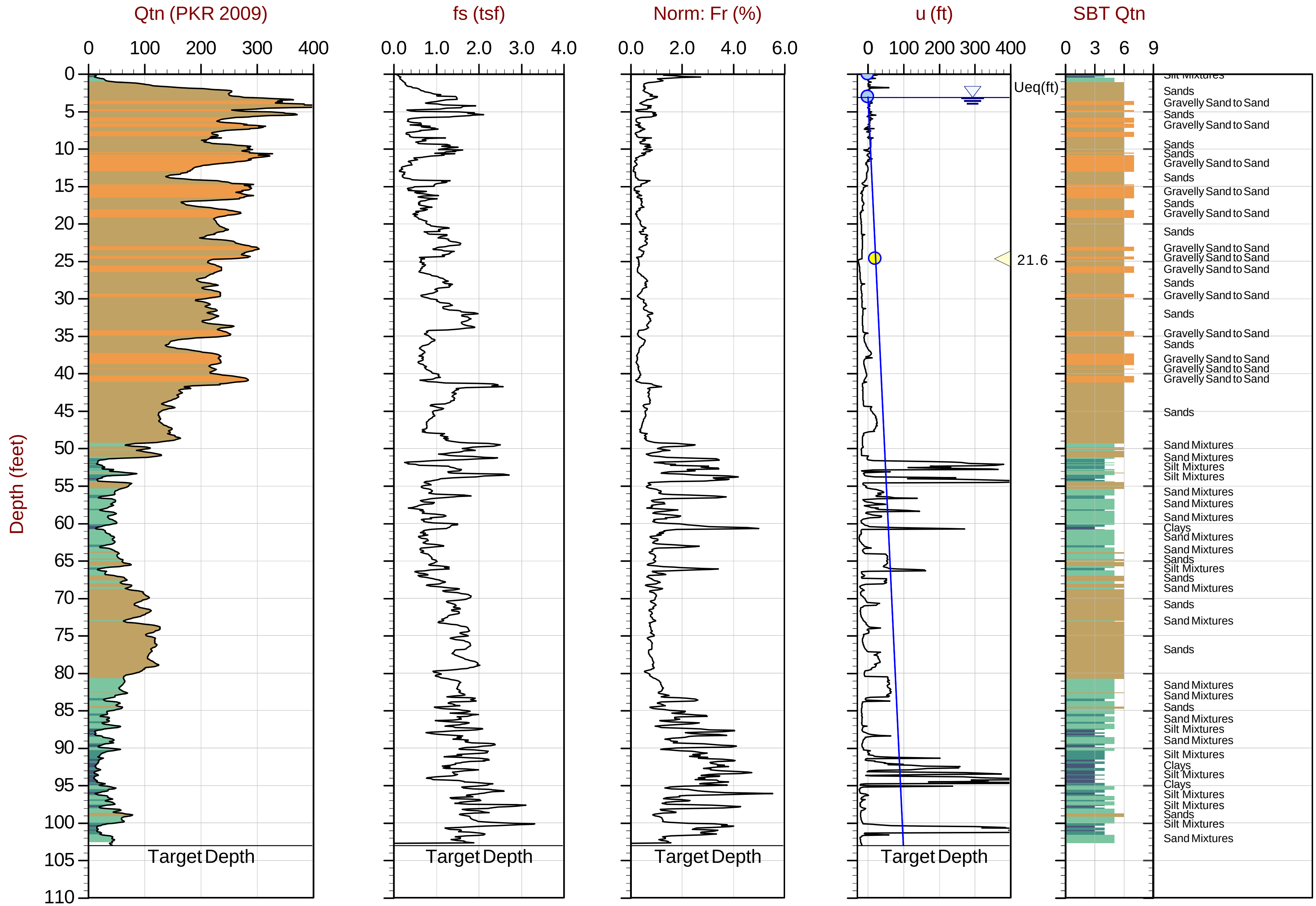
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 31.400 m / 103.02 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14630 Long: -122.16490

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

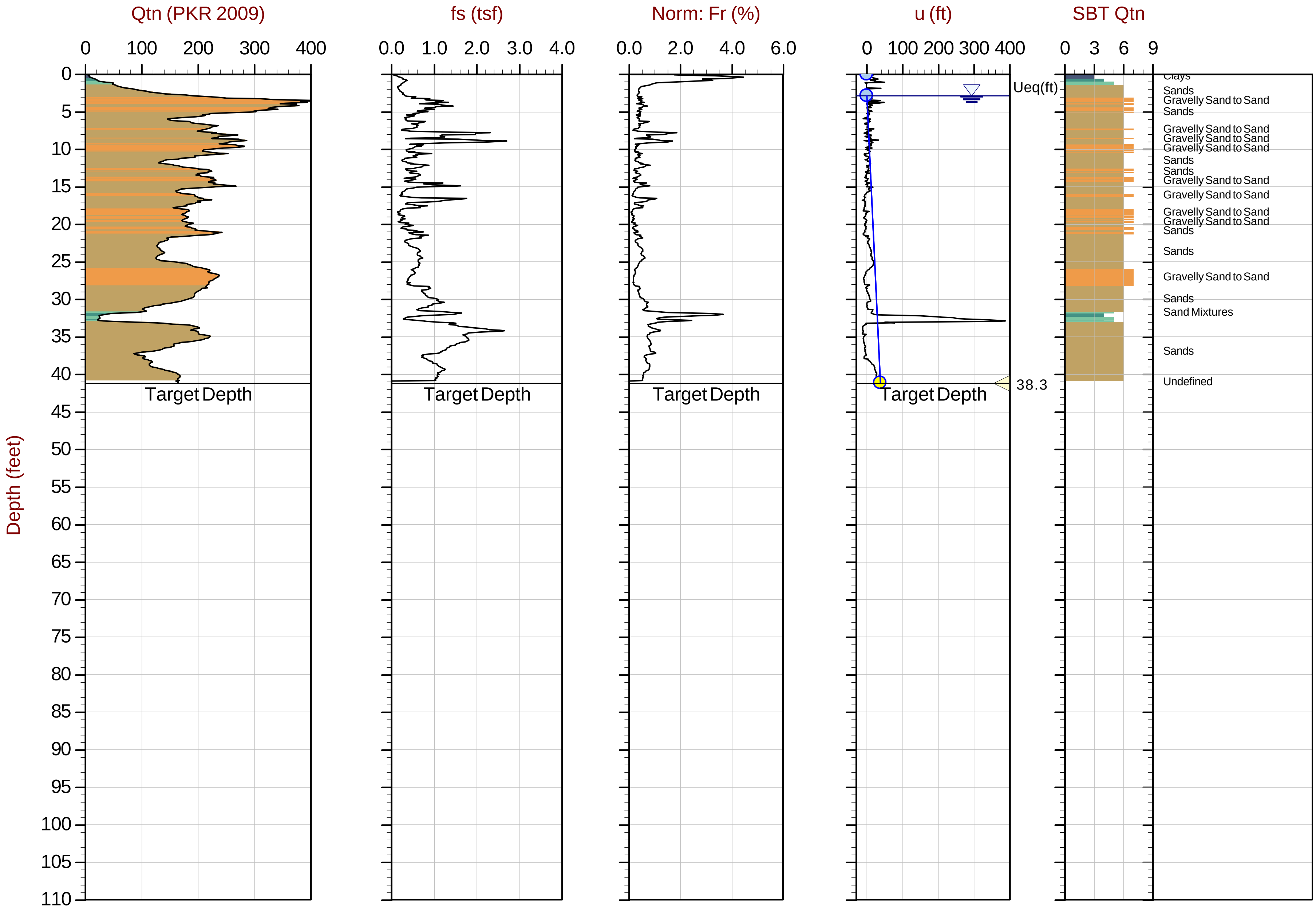
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 10:43
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 12.550 m / 41.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14632 Long: -122.16323

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.

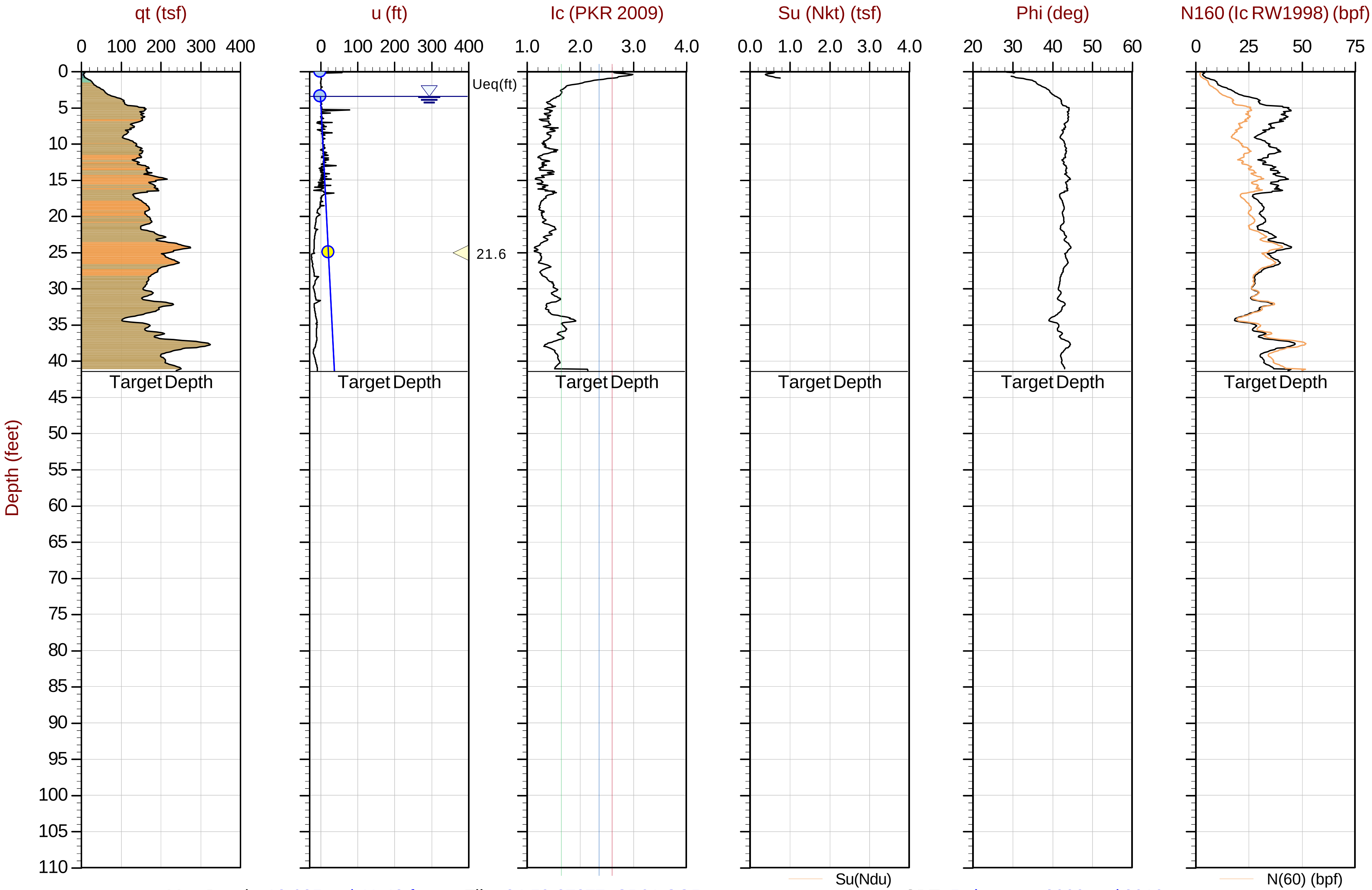
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

ConeTec

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:06
Site: 51st Ave NE, Arlington, WA
Sounding: CPT-01
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 12.625 m / 41.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14635 Long: -122.16648

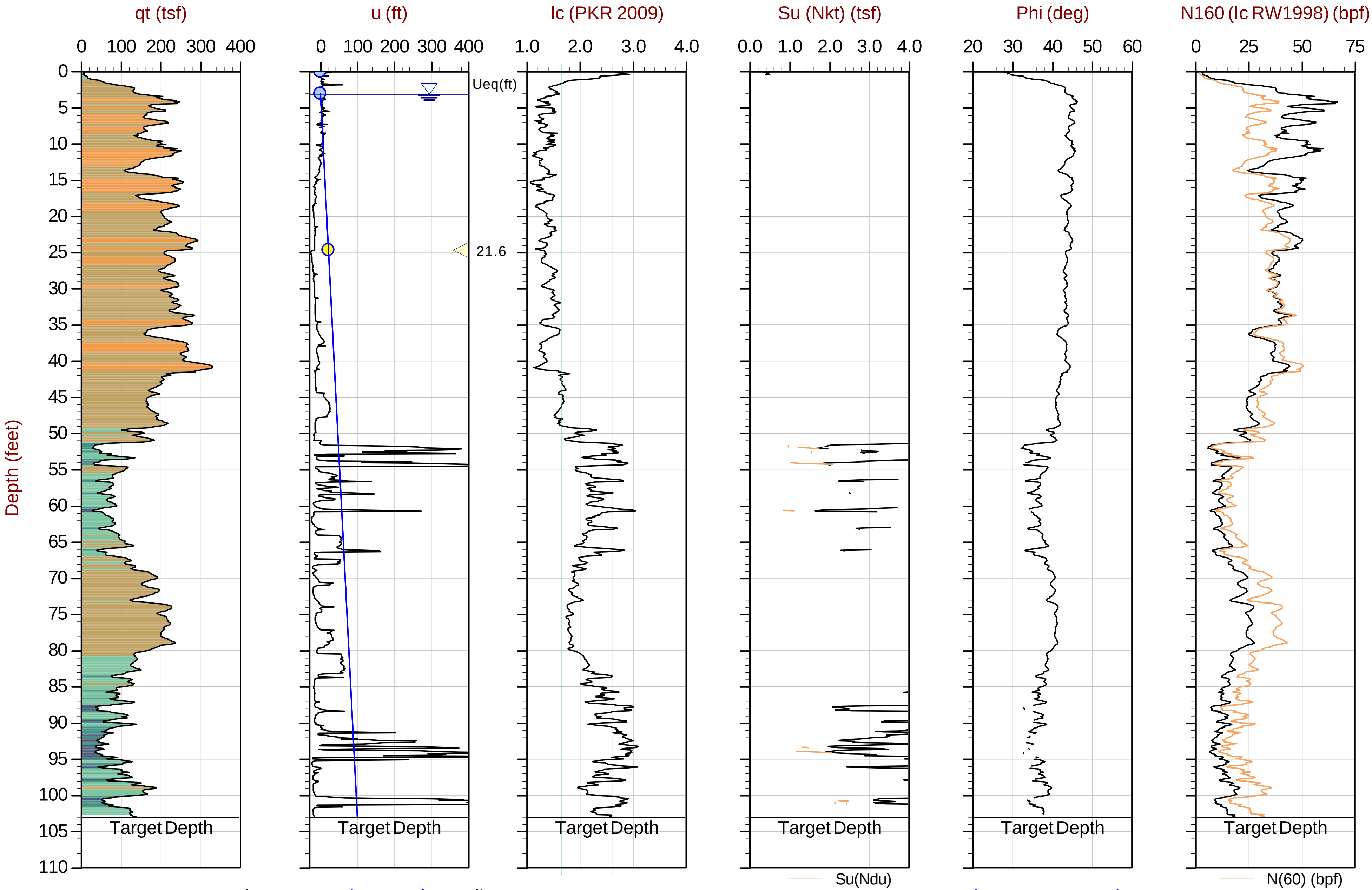
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA
Sounding: CPT-02
Cone: 855:T1500F15U35 Area=15 cm²



Max Depth: 31.400 m / 103.02 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14630 Long: -122.16490

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

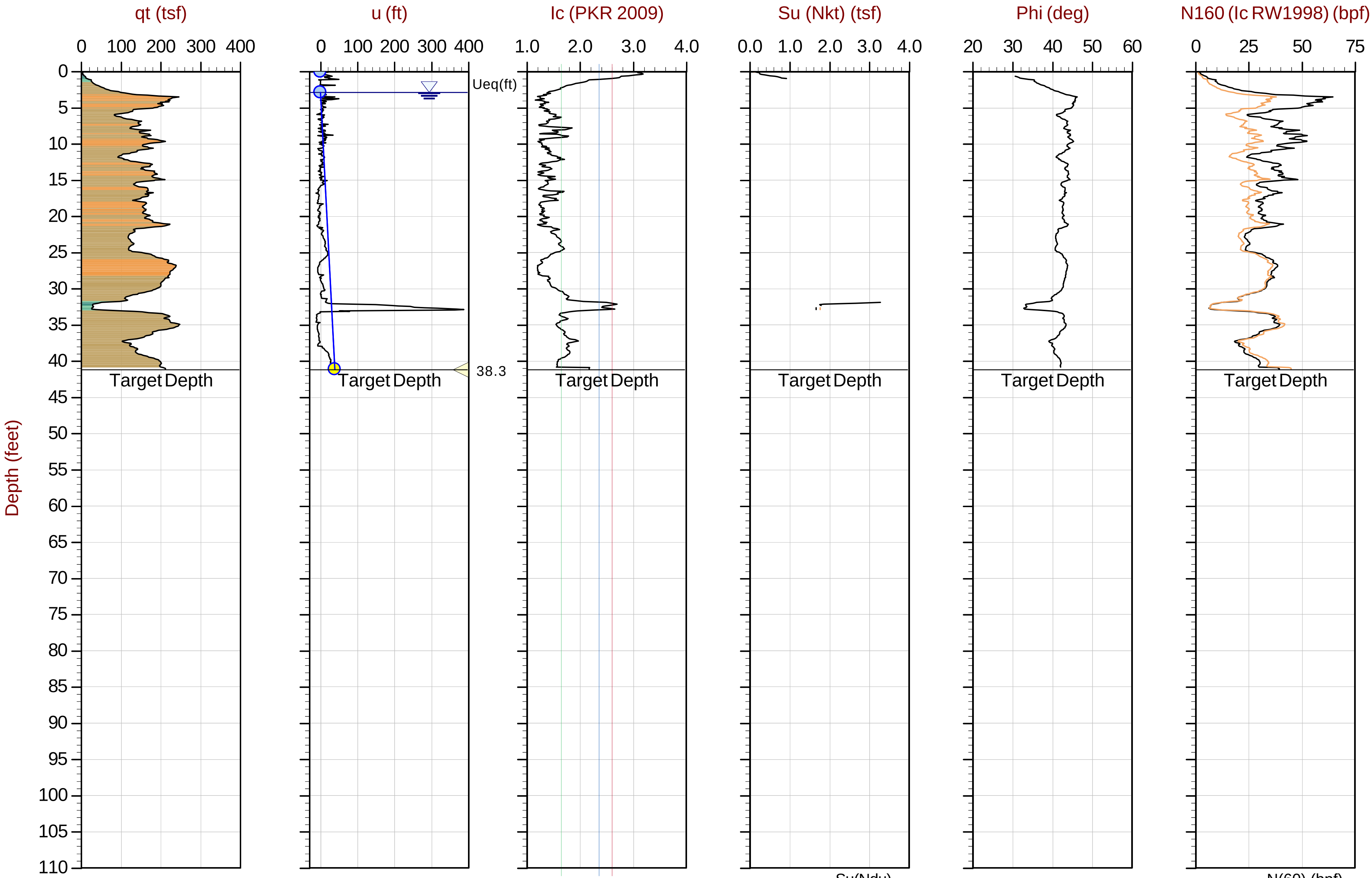
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 10:43
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 12.550 m / 41.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14632 Long: -122.16323

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

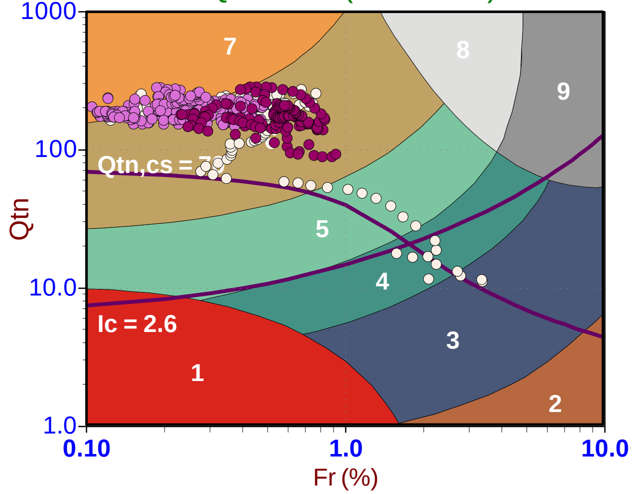
CONETEC

GeoEngineers

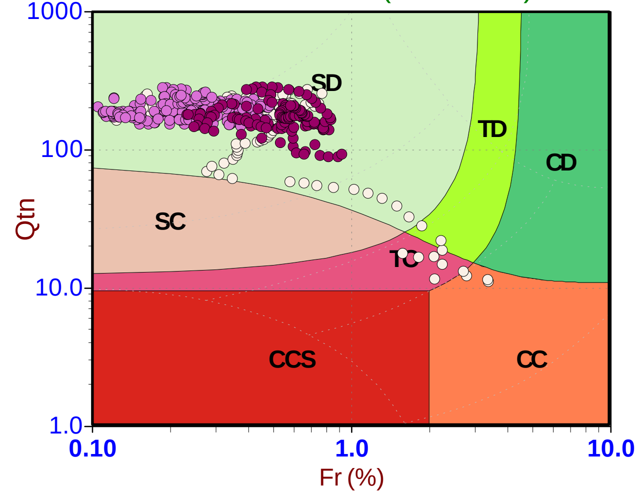
Job No: 24-59-27877
Date: 2024-06-26 08:06
Site: 51st Ave NE, Arlington, WA

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

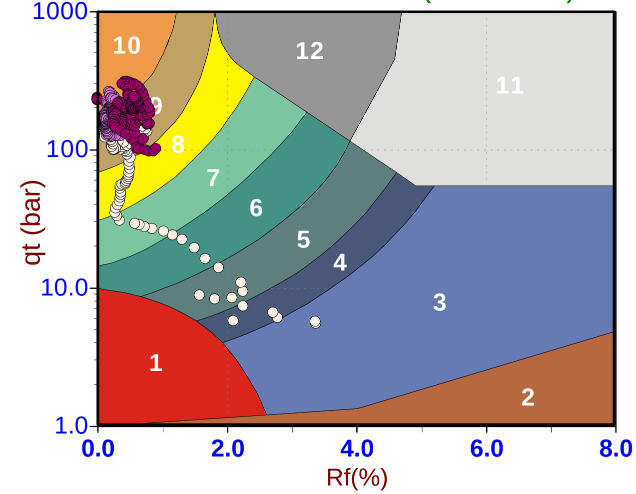
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 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)

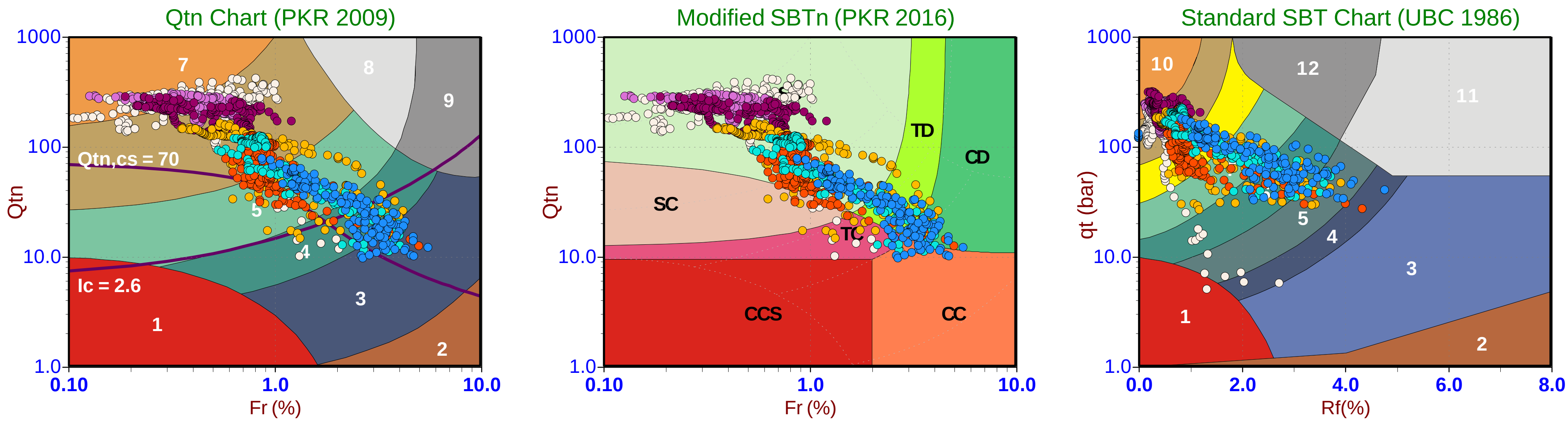
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA

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



Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 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)

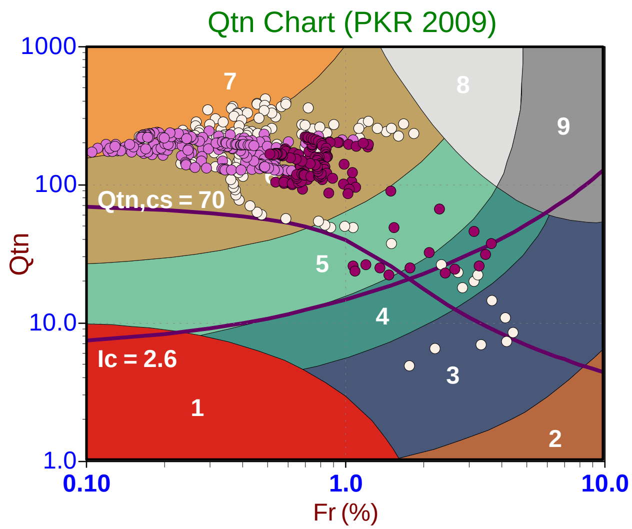
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

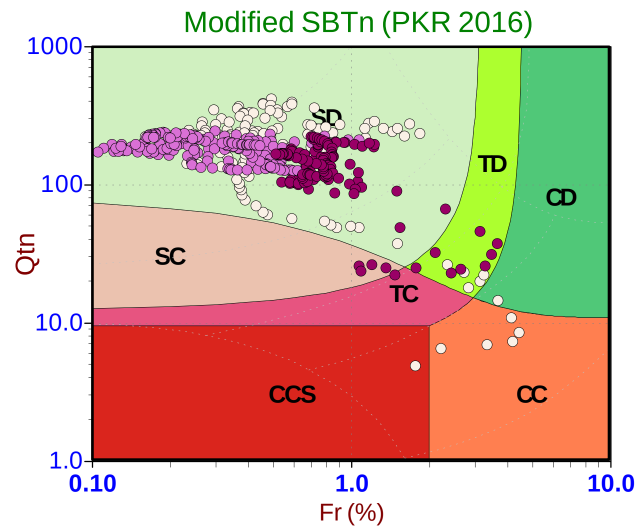
CONETEC

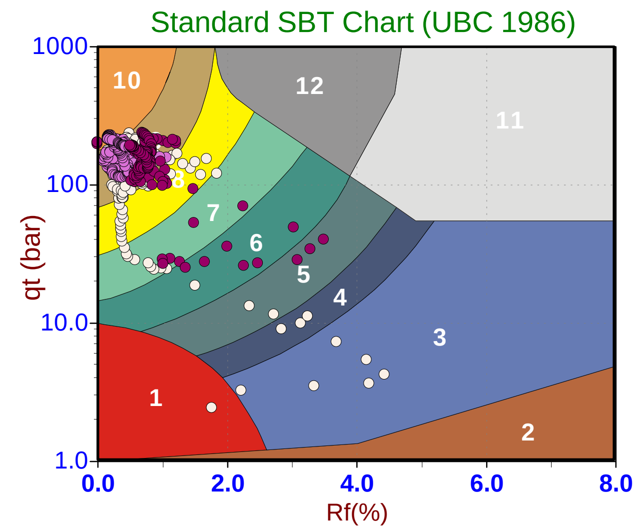
GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 10:43
Site: 51st Ave NE, Arlington, WA

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







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 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: 24-59-27877
Client: GeoEngineers
Project: 51st Ave NE, Arlington, WA
Start Date: 2024-06-26
End Date: 2024-06-26

CPTu PORE PRESSURE DISSIPATION SUMMARY

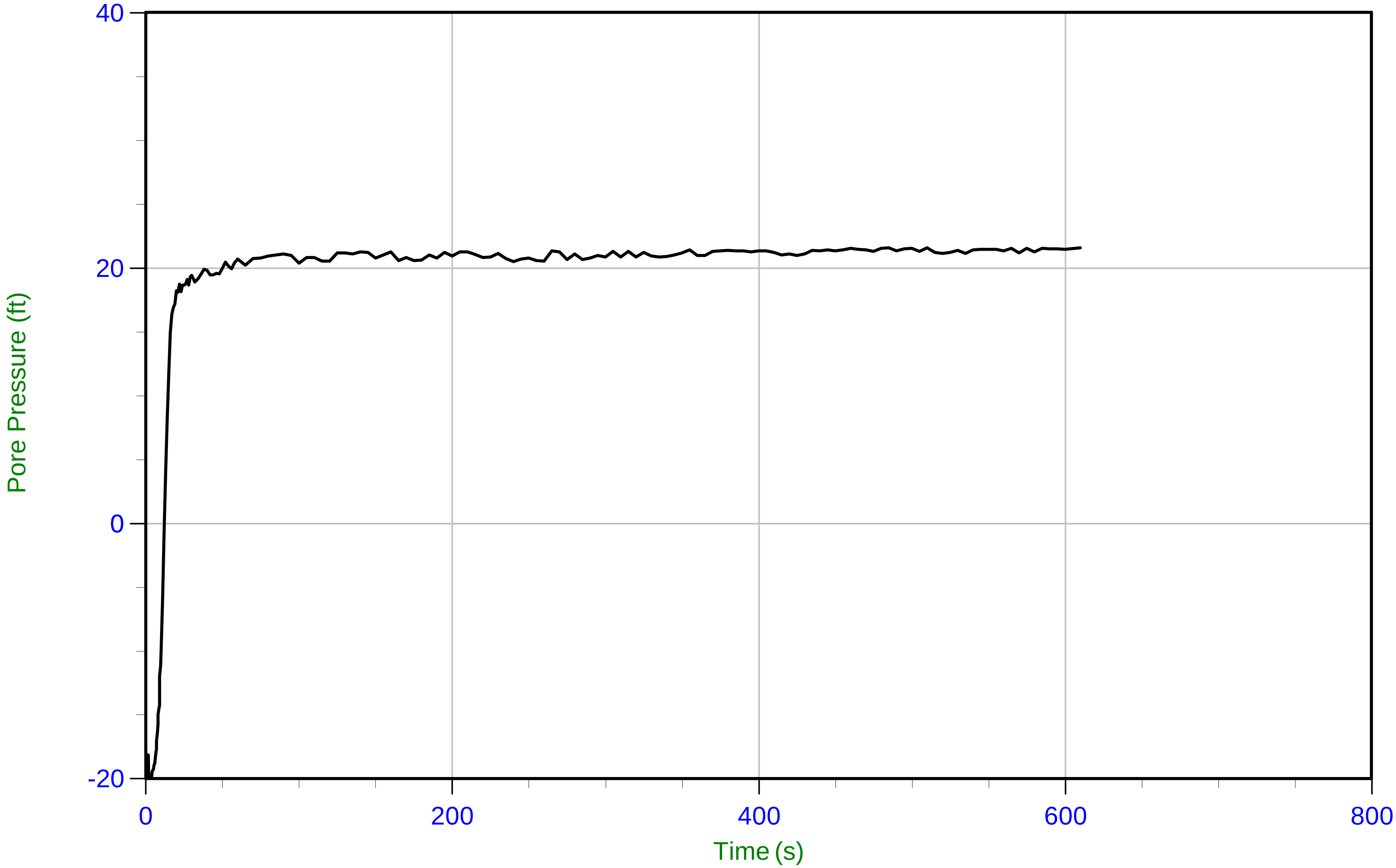
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Refer to Notation Number
CPT-01	24-59-27877_CP01	15.0	610	25.02	21.6	3.4	
CPT-02	24-59-27877_SP02	15.0	420	24.69	21.6	3.1	
CPT-03	24-59-27877_CP03	15.0	970	41.17	38.3	2.9	
Totals			33 min				

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:06
Site: 51st Ave NE, Arlington, WA

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



Trace Summary:

Filename: 24-59-27877_CP01.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 610.0 s

u Min: -20.8 ft
u Max: 21.6 ft
u Final: 21.6 ft

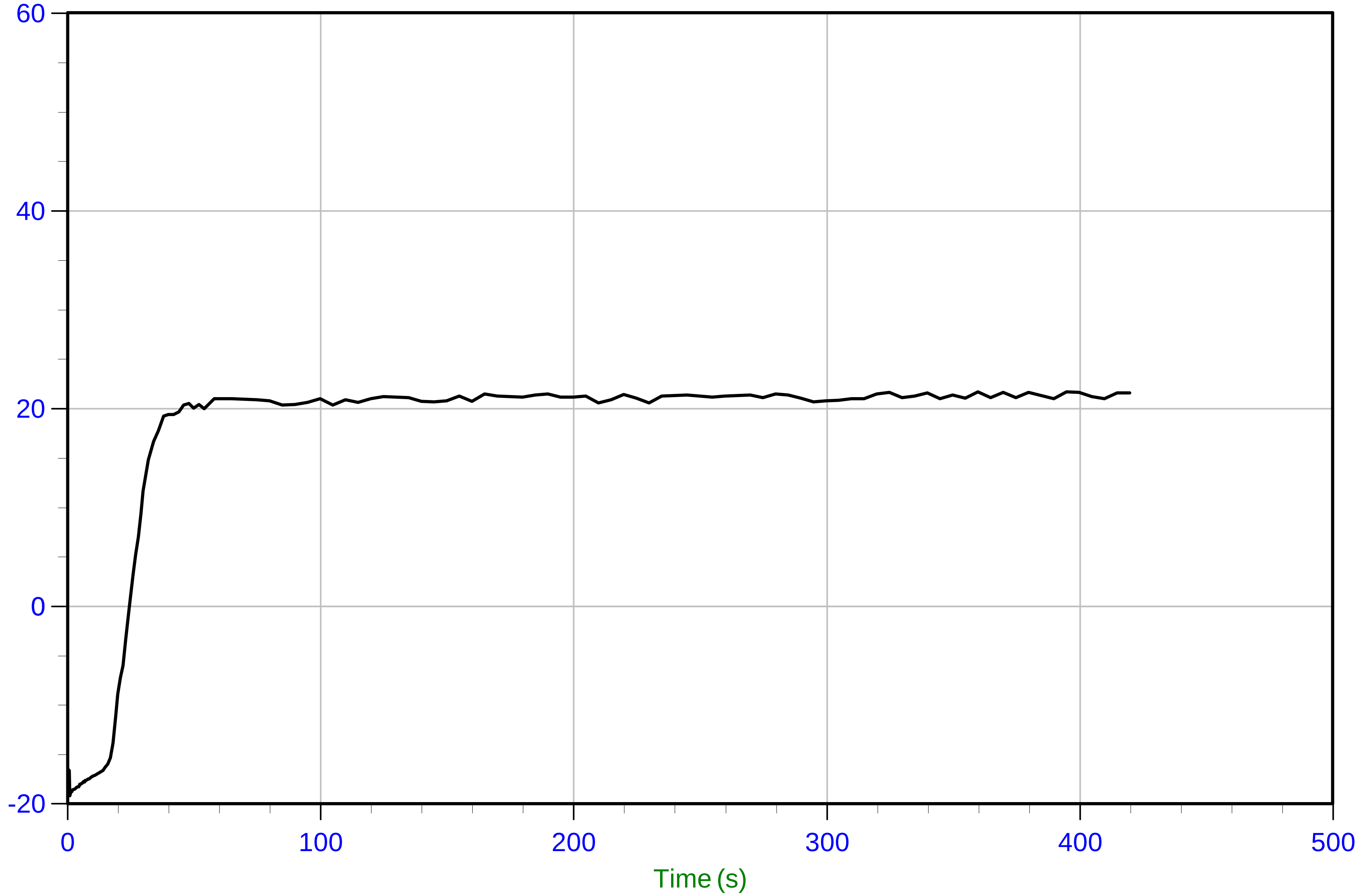
WT: 1.0 m / 3.4 ft
Ueq: 21.6 ft

CONETEC

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA

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



Trace Summary:

Filename: 24-59-27877_SP02.PPF2
Depth: 7.525 m / 24.688 ft
Duration: 420.0 s

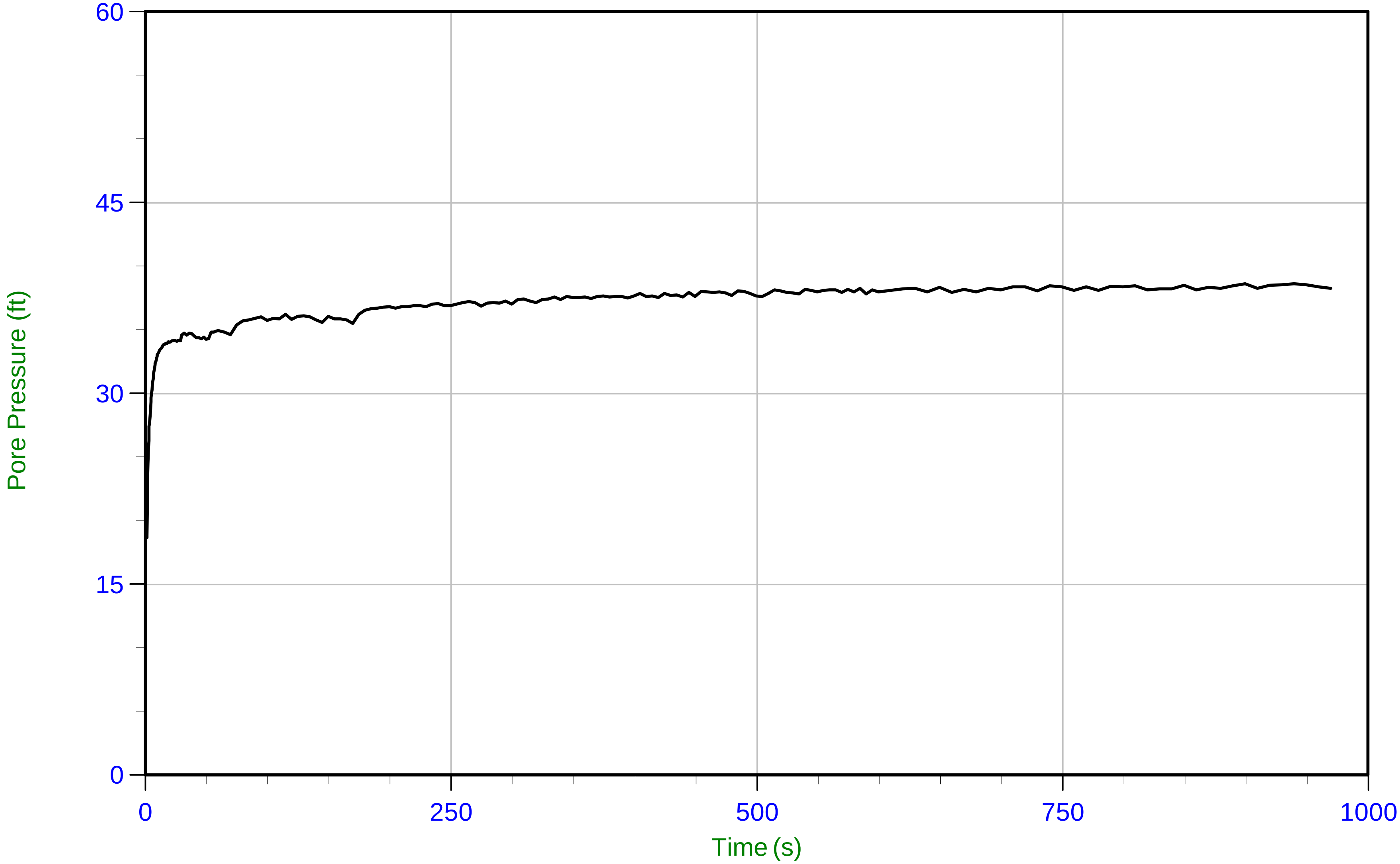
u Min: -19.2 ft
u Max: 21.7 ft
u Final: 21.6 ft

WT: 0.9 m / 3.1 ft
Ueq: 21.6 ft

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 10:43
Site: 51st Ave NE, Arlington, WA

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



Trace Summary:

Filename: 24-59-27877_CP03.PPF2
Depth: 12.550 m / 41.174 ft
Duration: 970.0 s

u Min: 18.6 ft
u Max: 38.6 ft
u Final: 38.3 ft

WT: 0.9 m / 2.9 ft
Ueq: 38.3 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

Job No: 24-59-27877
Client: GeoEngineers
Project: 51st Ave NE, Arlington WA
Sounding ID: CPT-02
Date: 2024-06-26

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

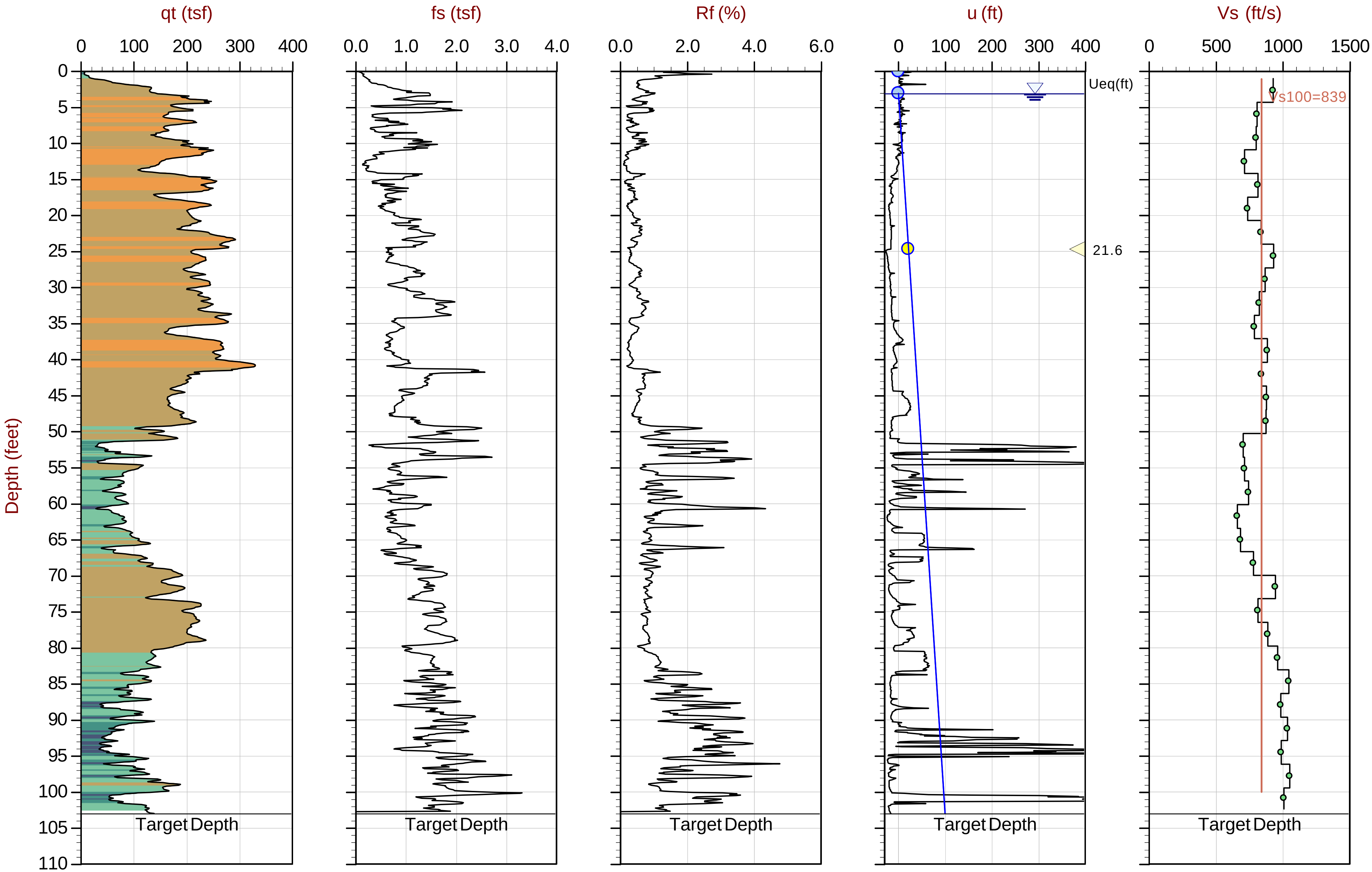
Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
1.71	1.05	2.03			
4.99	4.33	4.67	2.64	2.84	928
8.30	7.64	7.84	3.17	3.92	809
11.58	10.93	11.06	3.22	4.03	800
14.86	14.21	14.31	3.25	4.55	714
18.11	17.45	17.54	3.23	3.97	814
21.39	20.74	20.81	3.27	4.42	739
24.67	24.02	24.08	3.27	3.90	839
27.95	27.30	27.35	3.27	3.51	933
31.23	30.58	30.63	3.27	3.77	869
34.51	33.86	33.90	3.28	3.97	826
37.80	37.14	37.18	3.28	4.16	788
41.08	40.42	40.46	3.28	3.70	885
44.36	43.70	43.74	3.28	3.90	841
47.64	46.98	47.01	3.28	3.72	880
50.92	50.26	50.29	3.28	3.75	875
54.23	53.58	53.60	3.31	4.71	703
57.51	56.86	56.88	3.28	4.59	715
60.79	60.14	60.16	3.28	4.40	746
64.08	63.42	63.44	3.28	4.96	662
67.32	66.67	66.69	3.25	4.74	685
70.60	69.95	69.97	3.28	4.19	783
73.88	73.23	73.25	3.28	3.46	947
77.17	76.51	76.53	3.28	4.02	816
80.45	79.79	79.81	3.28	3.69	890
83.73	83.07	83.09	3.28	3.41	962
87.01	86.35	86.37	3.28	3.13	1047
90.29	89.63	89.65	3.28	3.33	986
93.57	92.91	92.93	3.28	3.17	1035
96.85	96.19	96.21	3.28	3.32	988
100.13	99.48	99.49	3.28	3.11	1055
103.02	102.36	102.38	2.89	2.86	1011

SCPTu Test Plots

GeoEngineers

Job No: 24-59-27877
Date: 2024-06-26 08:58
Site: 51st Ave NE, Arlington, WA

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



Max Depth: 31.400 m / 103.02 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27877_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.14630 Long: -122.16490

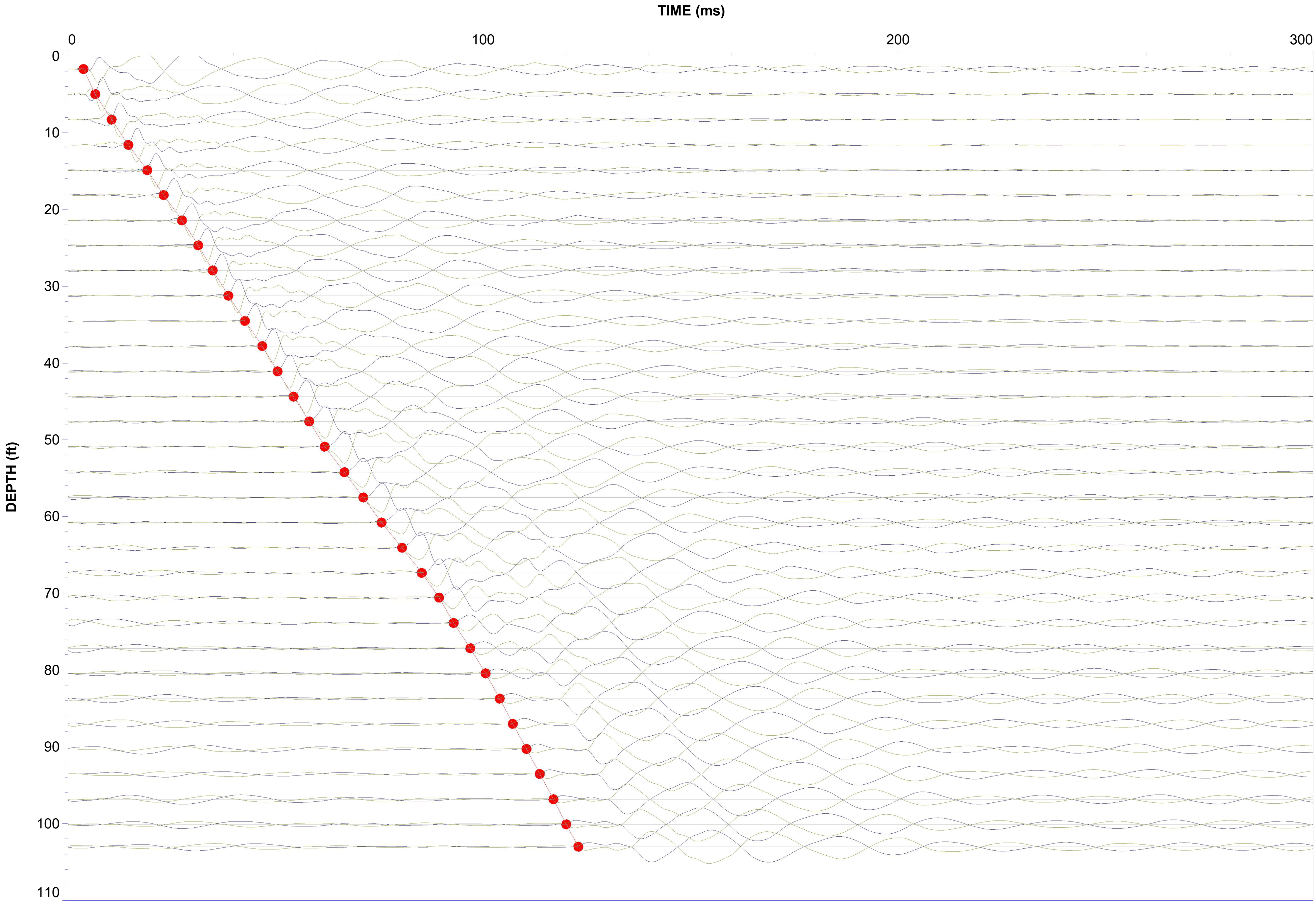
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

CONETEC

Job No: 24-59-27877 Client: GeoEngineers Project: 51st Ave NE, Arlington WA Filter: BP 0-300 Hz Sounding: CPT-02
Analysis: Shear Wave Date: 2024-06-26 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.

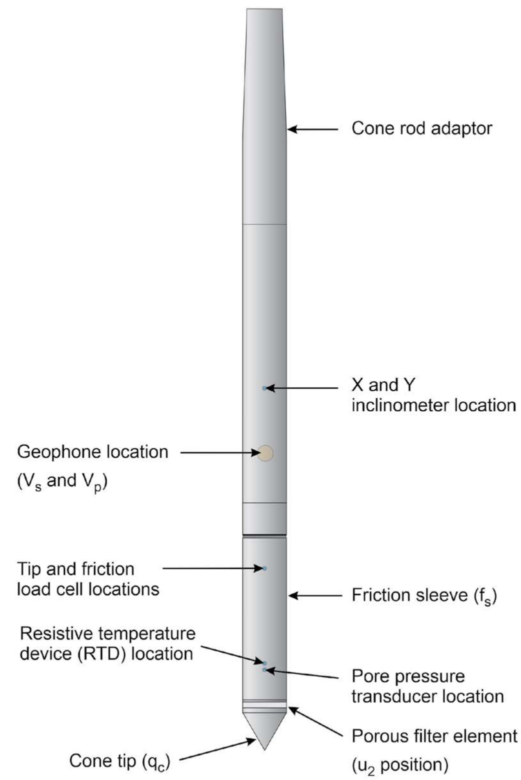


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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.

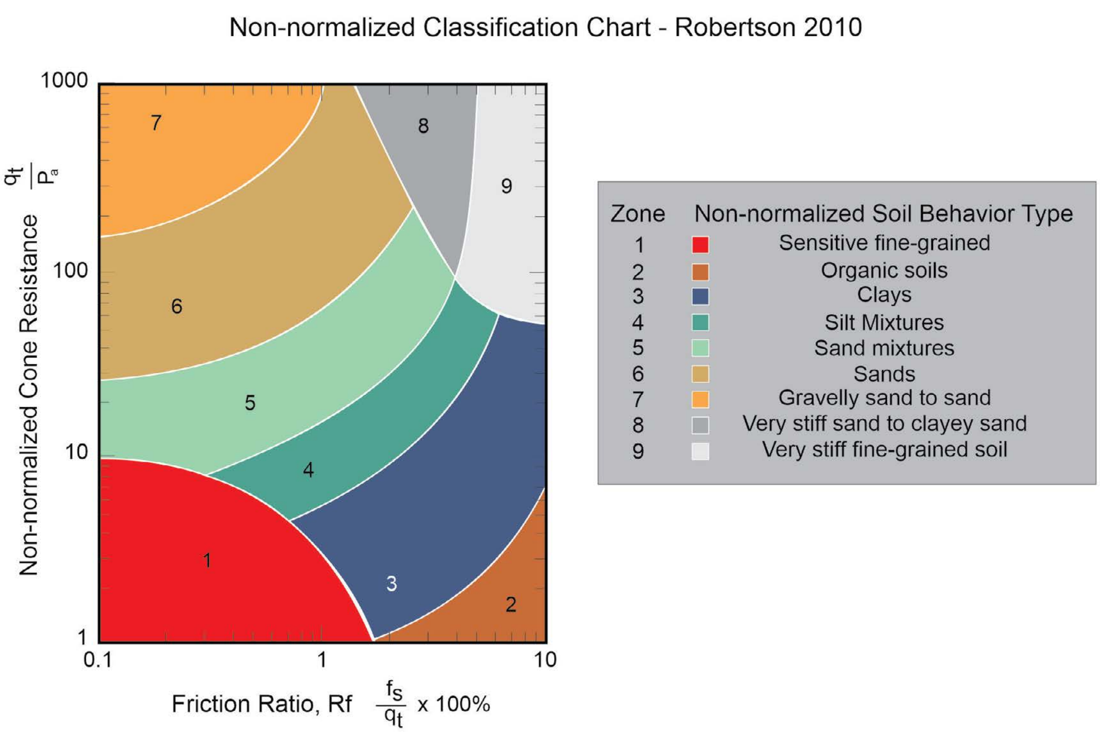


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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):

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

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

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

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

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

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

REFERENCES

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

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

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

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

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

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

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

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

PORE PRESSURE DISSIPATION TEST

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

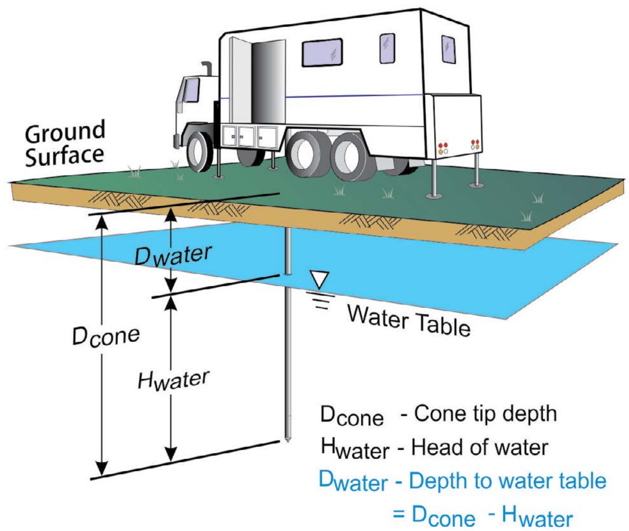


Figure PPD-1. Pore pressure dissipation test setup

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

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

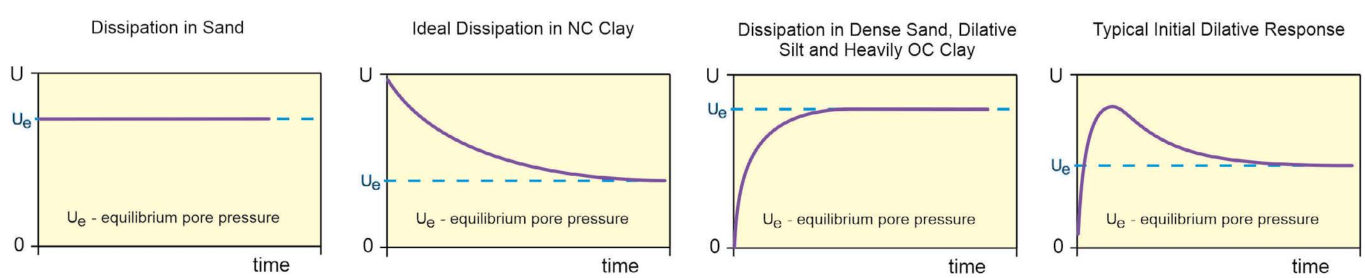


Figure PPD-2. Pore pressure dissipation curve examples

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

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.

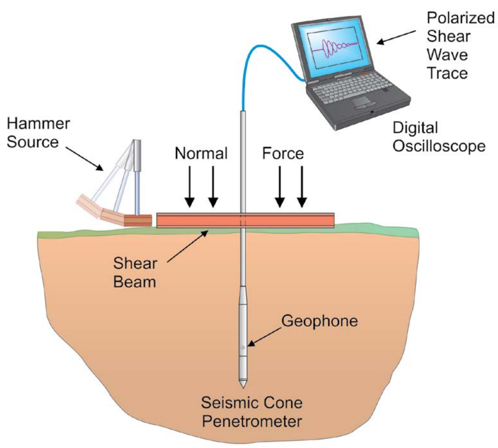


Figure SCPTu-1. Illustration of the SCPTu system

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).

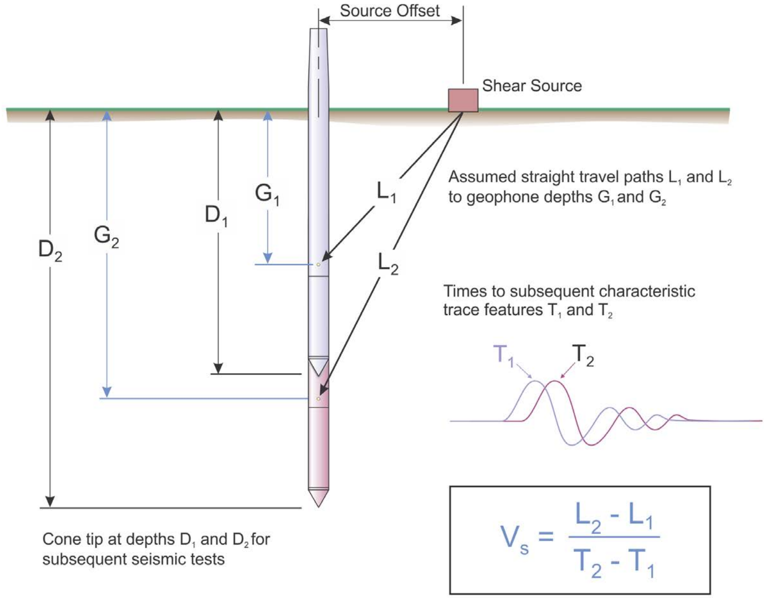


Figure SCPTu-2. Illustration of a seismic cone penetration test

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_s100 or V_s30.

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

REFERENCES

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

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.

ASTM D7400/D7400M-19, 2019, “Standard Test Methods for Downhole Seismic Testing”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

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

Description of Methods for Calculated CPT Geotechnical Parameters

ConeTec

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 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.

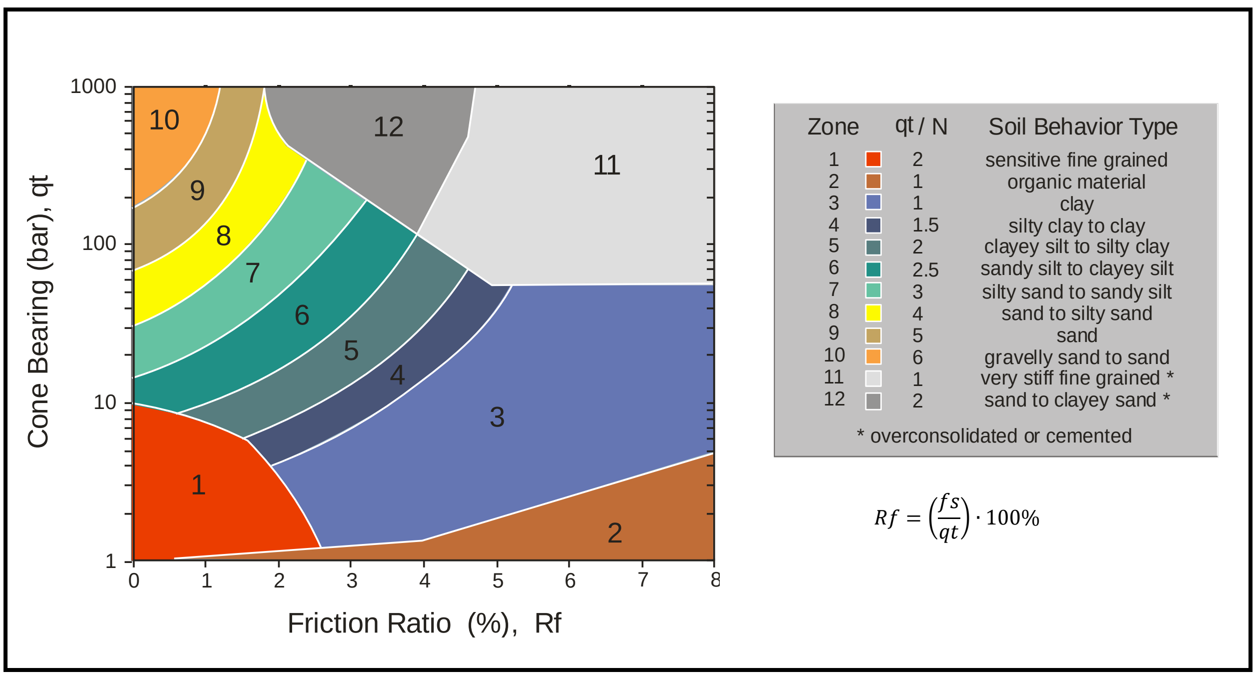


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

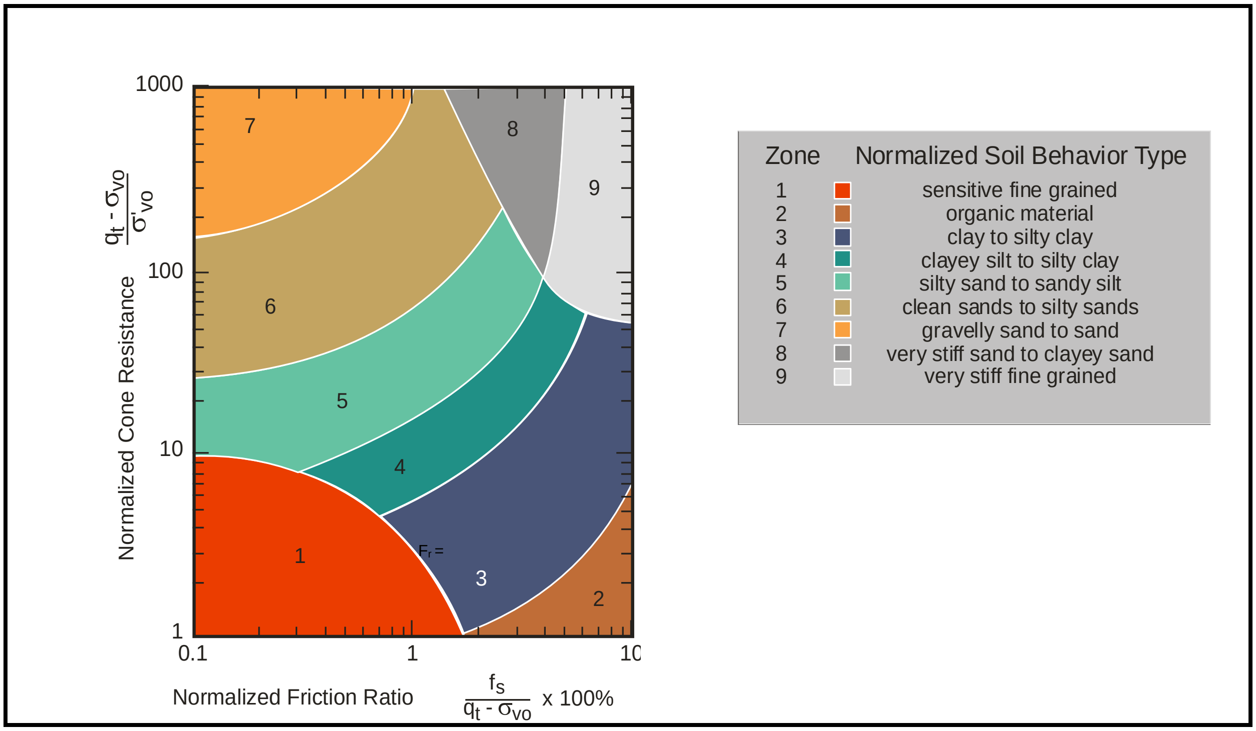


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

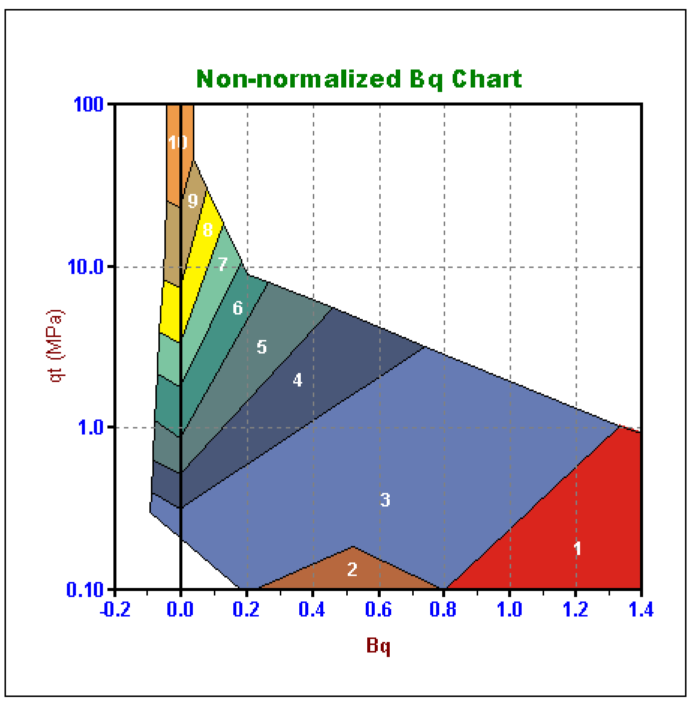


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

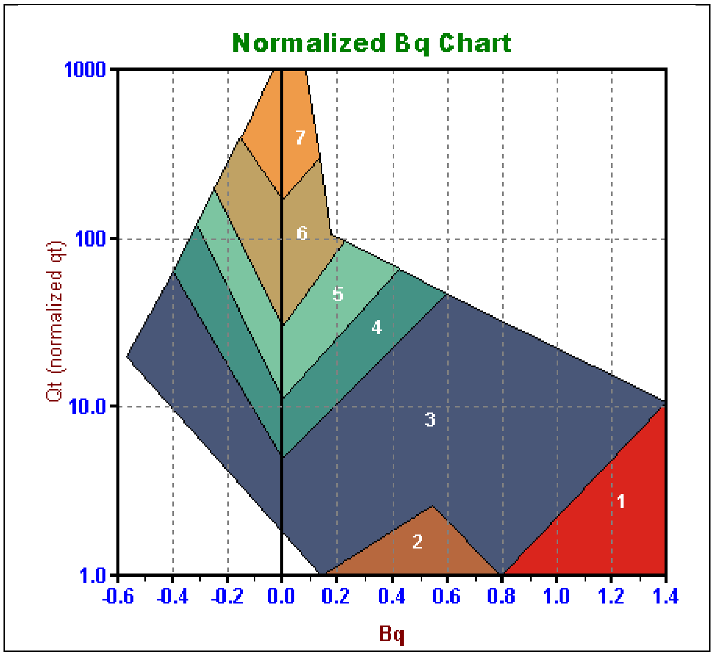


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

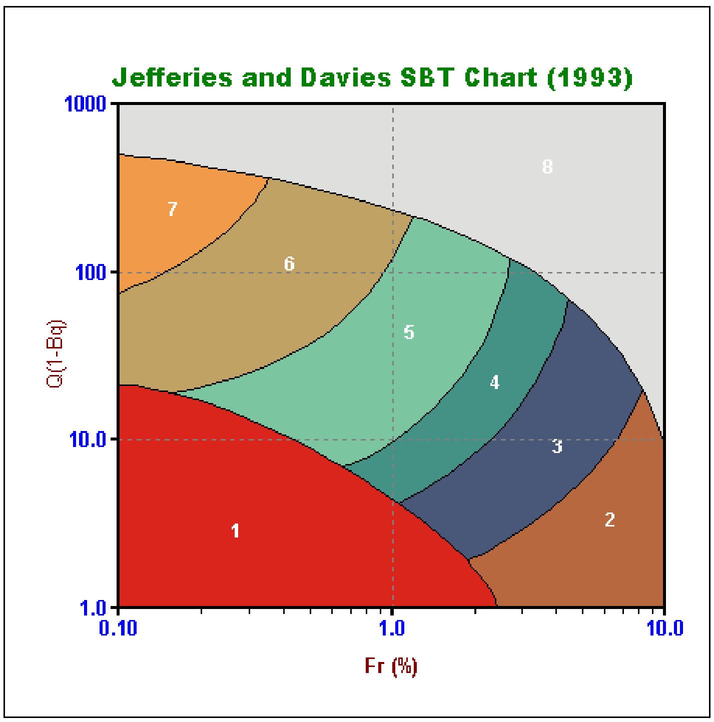


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

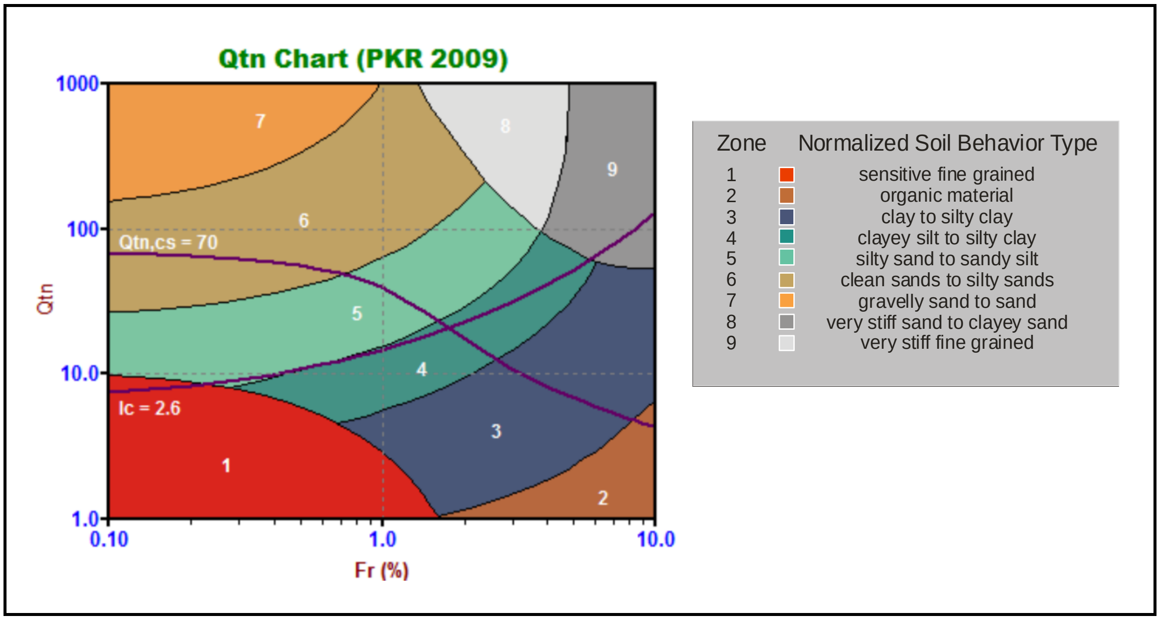


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

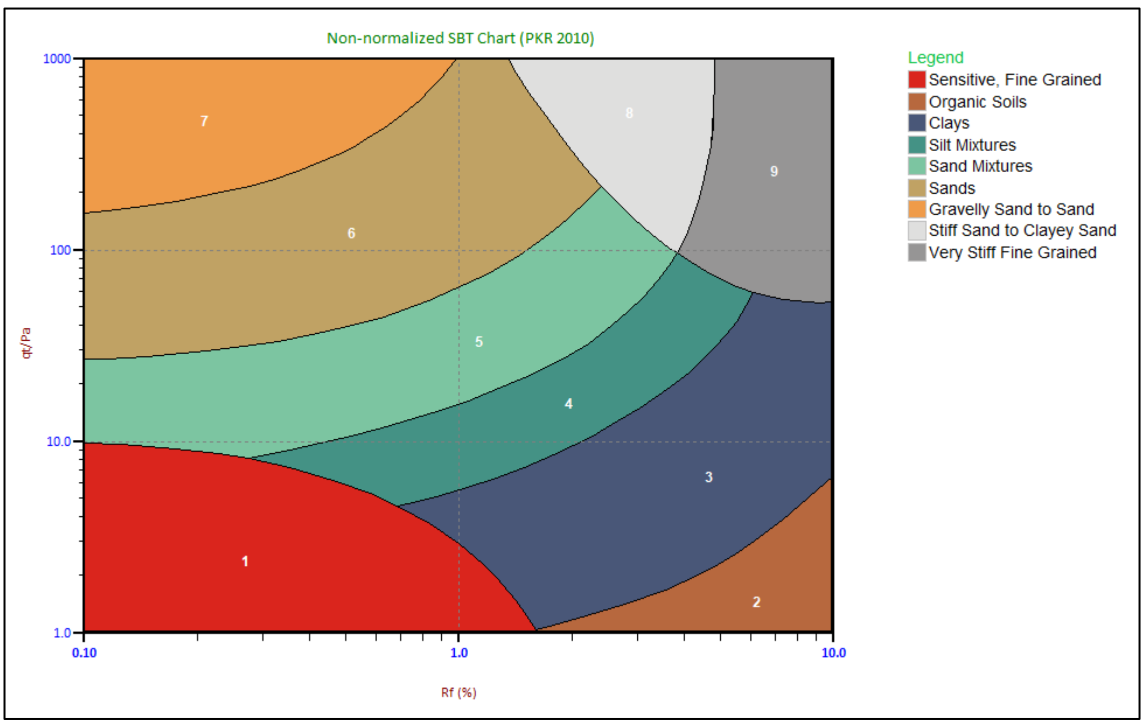


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

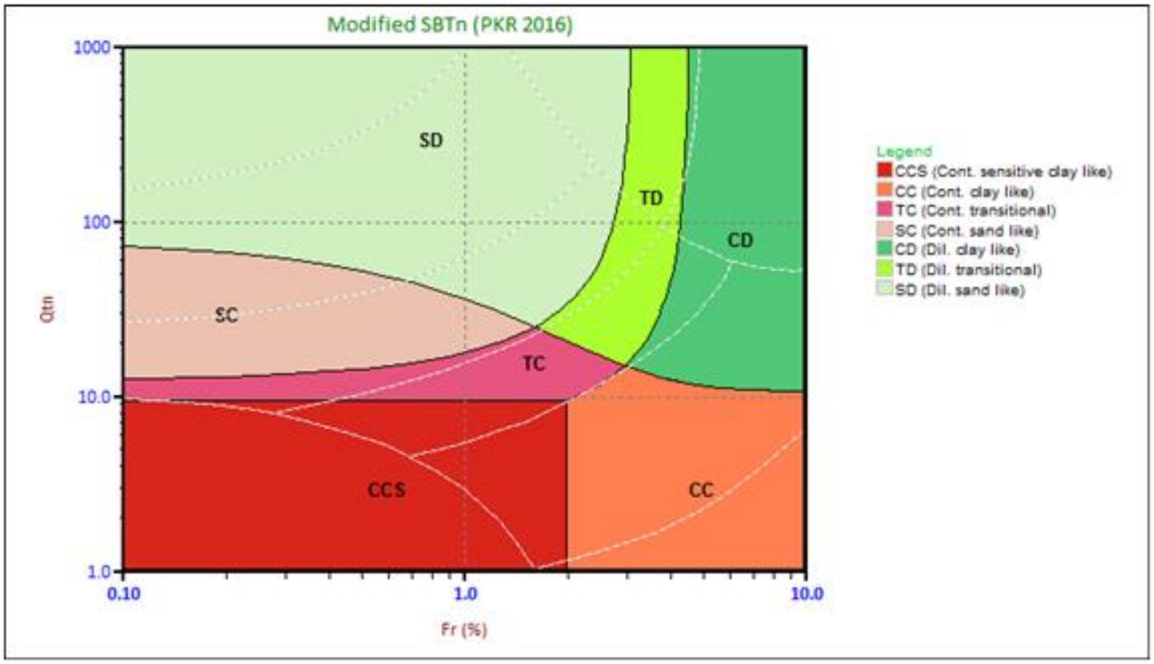


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

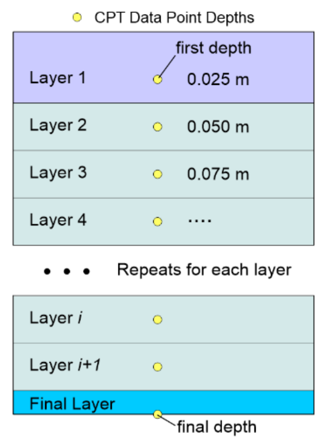
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

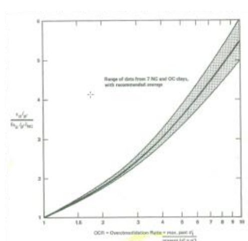
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where *0.0 < C_q < 2.0 (user adjustable)* *P_a is atmospheric pressure (100 kPa)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5\ (1 – I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 – 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\ I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5\ (1 – I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 – 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	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.	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	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)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

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

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

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

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

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	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	$\dfrac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\dfrac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}{}^{-m}]$ m = empirical exponent, typically 0.75	26

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

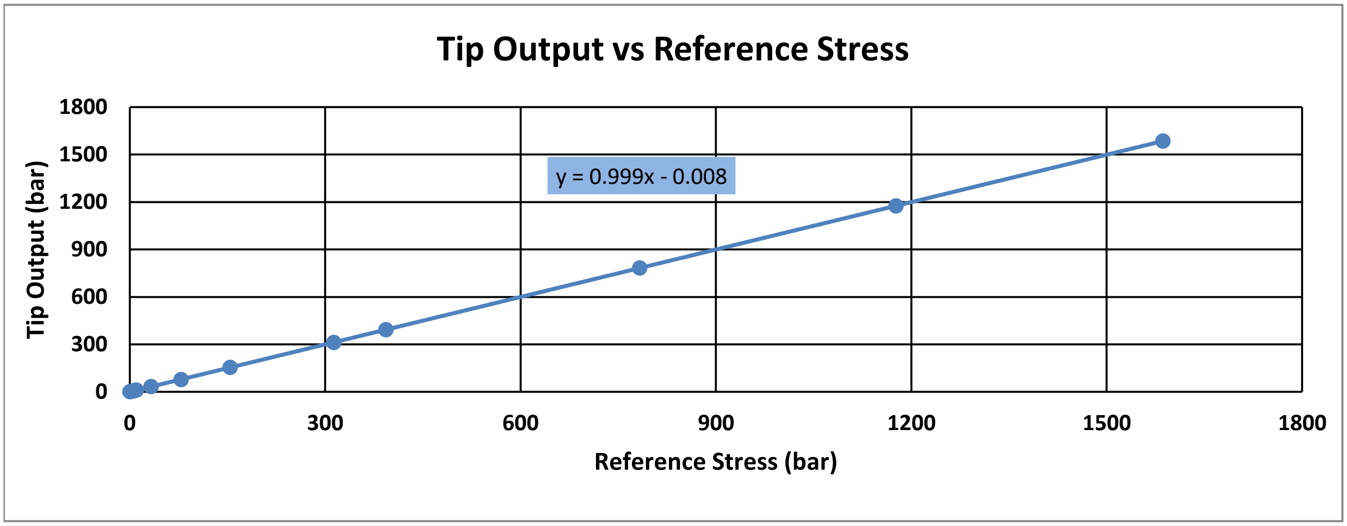
Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

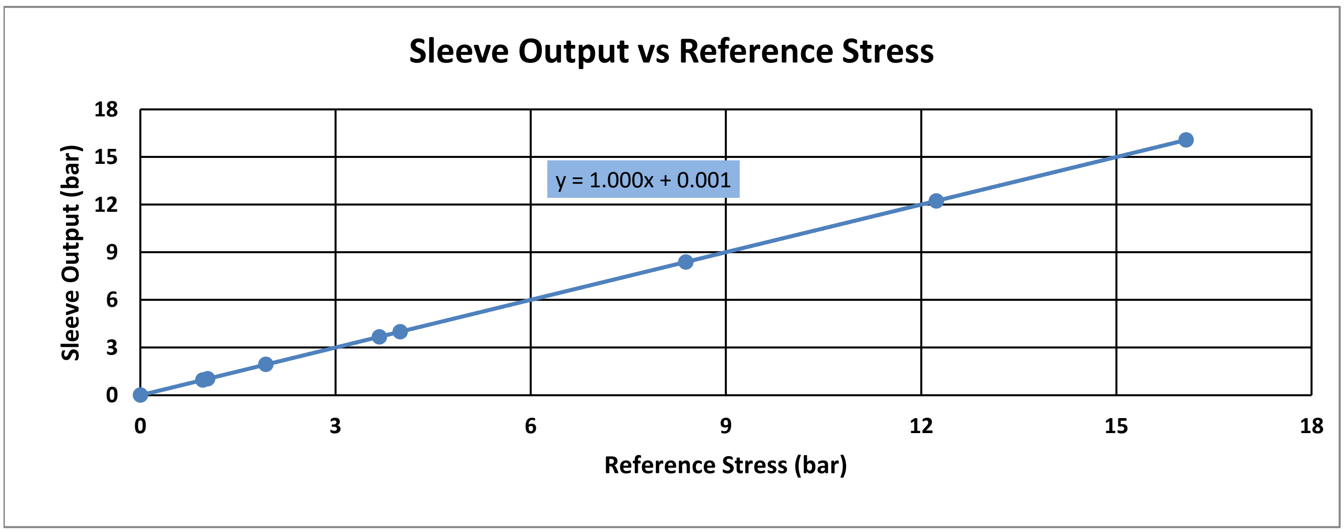
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

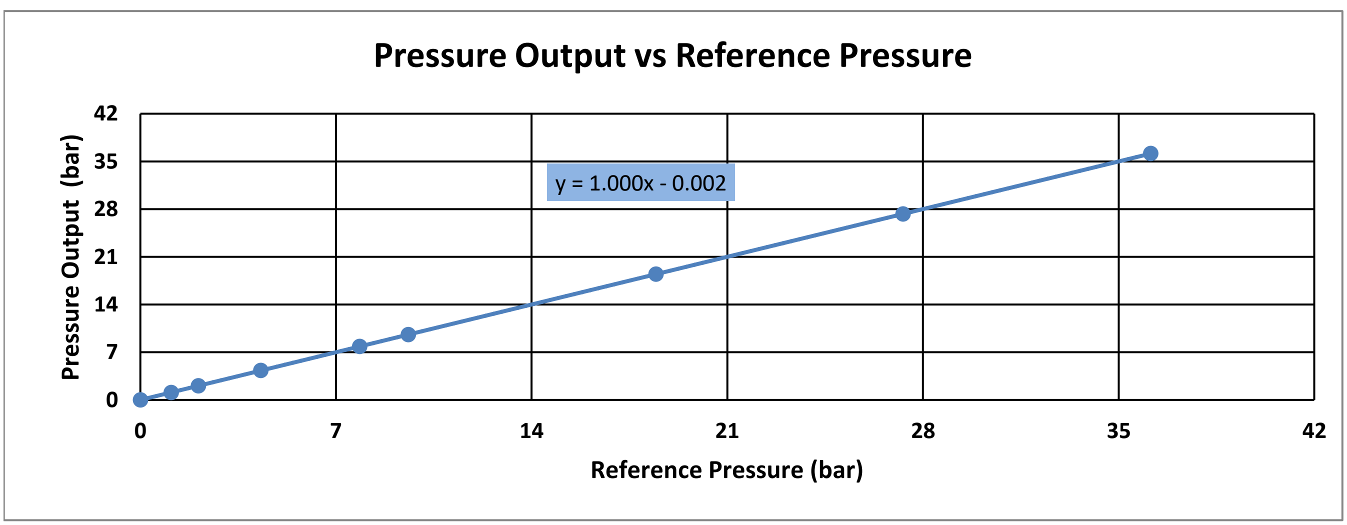
Calibrated with Adara calibration procedure EC_CPTCAL-2.2

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

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

Adara Error Definitions

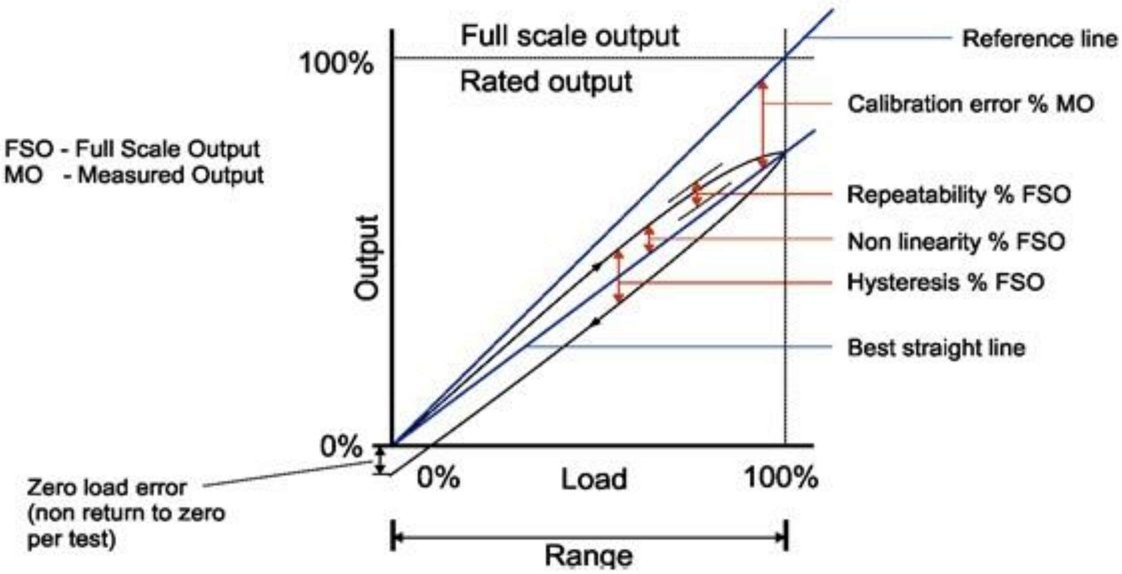

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

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 D
Report Limitations and Guidelines for Use

Appendix D
Report Limitations and Guidelines For Use[2]

This appendix provides information to help you manage your risks with respect to the use of this report.

READ THESE PROVISIONS CLOSELY

It is important to recognize that the geoscience practices (geotechnical engineering, geology, and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory “limitations” provisions in its reports. Please confer with GeoEngineers if you need to know more how these “Report Limitations and Guidelines for Use” apply to your project or site.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS, AND PROJECTS

This report has been prepared for the Rockefeller Group and for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Rockefeller Group dated June 10, 2024, and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the Northsound Logistics Center project in Arlington, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

[2] Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations

GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to conduct additional study to obtain the specific types of information they need or prefer.

CONTRACTORS ARE RESPONSIBLE FOR SITE SAFETY ON THEIR OWN CONSTRUCTION PROJECTS

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule, or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

BIOLOGICAL POLLUTANTS

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.