Geotechnical MUP Report

Seattle Clinic Renovation
Seattle, Washington

for
Evergreen Treatment Services

August 16, 2023

GeoEngineers
Earth Science + Technology

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August 16, 2023

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1.0 INTRODUCTION

This report presents the preliminary results of GeoEngineers, Inc.'s (GeoEngineers') geotechnical engineering services for the development of the site located at 1700 Airport Way South in Seattle, Washington. The subject property is bounded by Airport Way South on the west, the Emerald Recycling Company, Inc. shed structure and tank farm on the north, the Interstate 5 (I-5) right-of-way on the east, and a business park on the south. The site is shown relative to surrounding physical features in the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

The purpose of this report is to provide preliminary geotechnical engineering conclusions and recommendations for the design and construction of the planned development. The approximately 32,516-square-foot site consists of one King County Parcel (parcel no. 766620-2855) that is currently occupied by a wood-framed building originally constructed in 1914 and a concrete parking lot. The site grade is relatively flat at about Elevation 26 to 28 feet. GeoEngineers' geotechnical engineering services have been completed in general accordance with our signed agreement executed on March 3, 2023. Our scope of services includes:

- Reviewing existing subsurface information available for the site and surrounding area;
- Performing an assessment of the Environmentally Critical Area (ECA) designations for the site, including steep slopes, landslide area, and liquefaction;
- Providing recommendations for seismic design in accordance with the 2018 International Building Code (IBC);
- Evaluating the geologic hazards at the site;
- Providing preliminary regarding appropriate foundation types and capacities;
- Providing preliminary soil pressures to be used for the design of cantilever and basement type retaining walls, along with recommended surcharge loading and seismic pressures;
- Providing preliminary recommendations for earthwork and groundwater control; and
- Preparing this Master Use Permit (MUP) report.

2.0 PROJECT DESCRIPTION

GeoEngineers understands that Evergreen Treatment Services (ETS) is considering a new six-story structure to be constructed at-grade. The new structure will occupy the footprint of the current North Building. New parking improvements, which may include a ½ level basement, will be constructed in place of the current South building. The development will be constructed in phases: the North Building will be demolished, and the new structure will be built; the demolition of the South building will then follow along with new parking improvements. Based on our review of the conceptual plans dated February 21, 2023, the new building will have a ground finished floor elevation matching the Airport Way South street grades, or about Elevation 24 feet (North American Vertical Datum of 1988 [NAVD88]).

The site lies at the eastern extent of the former tide flats, which were filled as part of the Beacon Hill regrading in the late 1800s.

3.0 FIELD EXPLORATIONS

We advanced four cone penetration tests (CPTs, CPT-1 through CPT-4) to depths between 34.5 and 44.9 feet. Each of the CPTs encountered practical refusal before their planned depths of 100 feet. The CPTs were advanced by Conetec, Inc. from April 28 through May 4, 2023. Within two of the CPTs, we measured the shear wave velocity (CPT-01 and CPT-04) of the subsurface soils. The locations of the CPT explorations are shown in Figure 2. The results of the CPTs are included in Appendix A.

3.1. Previous Site Evaluations

In preparing this report, we reviewed previous boring logs and laboratory test results from subsurface explorations completed in the site vicinity. The locations of the previous explorations completed by GeoEngineers and others are shown on the Site Plan (Figure 2). Boring logs in the immediate vicinity of the site are presented in Appendix B.

4.0 SITE CONDITIONS

4.1. Surface Conditions

The site is currently occupied by two wood-framed buildings (North Building and South Building), originally constructed in 1914, and a concrete parking lot. The existing buildings have a basement level with finished floor levels of about Elevation 19.5 feet (as-built drawings were not available at the time we prepared this report). The foundations of these buildings are currently unknown, but we expect that they may be supported on timber piles. The concrete parking lot area consists of 7- to 10-inch-thick layer of concrete. We understand that prior to the site's current use, it was originally used as a blower factory which had heavy vehicle traffic. The former Great Northern railway ran along the eastern property line prior to its removal. To the east of the site is a crib retaining wall and the I-5 right-of-way. The I-5 structure was constructed in the mid-1960s. As part of its construction, the area upslope of the site was re-graded, and the aerial structure was built.

The site is designated as an ECA for steep slopes, potential slide area, liquefaction-prone area, and known slide affected property in accordance with the Seattle Municipal Code (SMC) Chapter 25.09 – see Figure 2 for extents of these designations in relation to the site. Based on our review of the Seattle Landslide Inventory, we understand that the slope movement occurred uphill and east of the site in 1934 and did not extend onto the property. The slide debris was subsequently removed by the City of Seattle.

Numerous buried utilities are located within and near the project site and within the public right-of-way along the adjacent streets. These utilities include, but are not limited to, electrical, telecommunication, gas, overhead power, water, sanitary sewer, and storm drain.

4.2. Subsurface Conditions

GeoEngineers' understanding of subsurface conditions is based on the results of the four CPTs as well as our review of existing geotechnical information in the site vicinity. The approximate locations of the CPTs and previous explorations in the site vicinity are shown in Figure 2. Three generalized soil types were encountered in the previous explorations and include: fill, estuarine deposits, and glacially consolidated soils.

Fill was encountered in each of the explorations at the site vicinity and ranged in depth from approximately 11 to 16 feet below the ground surface (bgs). The fill generally consists of sand with variable amounts of silt and gravel. The fill is loose to dense where encountered.

Estuarine deposits were encountered below the fill and generally consist of clay and silt mixtures. These materials comprised the near-surface soils prior to the Beacon Hill regrading operations, where material was sluiced down the hill to fill the site vicinity. This deposit generally increases in thickness from east to west, as the site borders on the margin of the former tide flats. Estuarine deposits are 5 to 10 feet thick on the east side of the site, and 19 feet thick on the west side of the site where encountered.

Glacially consolidated soils were encountered below the estuarine deposits. The elevation of the top of bearing soil layer is presented in Figure 2. The ridge along the tide flats eastern margin is mapped as Possession-Age glaciolacustrine soils, which consist of glacially consolidated silt and clay that slope down to the west. These materials are very stiff to hard where encountered. The glacially consolidated soils extended to the depths explored.

Although not encountered during drilling, occasional boulders have been observed in glacially consolidated soils in nearby excavations and may be present at the site. Additionally, pile foundations for buildings and trestles that were constructed previously in the site vicinity, foundation elements, and other debris may be encountered in the fill soils.

4.3. Groundwater Conditions

During advancement of the CPTs, we performed pore pressure dissipation tests to measure hydrostatic pressure at the CPT locations. In addition, a monitoring well was previously installed as part of prior environmental studies at the site. Table 1 below provides a summary of the CPT measurements as well as the monitoring well at the site. Groundwater in the site vicinity is present in a shallow, unconfined aquifer.

TABLE 1. MONITORING WELL GROUNDWATER MEASUREMENTS

Well ID	Ground Surface Elevation (feet)	Approximate Depth to Groundwater (feet)	Measured Groundwater Elevation (feet, NAVD88)
AMW-1	26	5.5	20.5
AMW-2	26	5.5	20.5
AMW-3	26	8.5	17.5
AMW-4	28	6.0	22.0

Based on monitoring well and boring logs reviewed in the project vicinity, we anticipate that the groundwater table at the site is at Elevation 22 feet.

Groundwater levels are anticipated to vary as a function of location, precipitation, tidal influence, season, and other factors.

5.0 ENVIRONMENTALLY CRITICAL AREAS

GeoEngineers has reviewed the ECA maps available online through the City of Seattle Department of Construction and Inspections (SDCI) geographic information system (GIS) website. Based on our review of the SDCI GIS maps, the site is located within a mapped steep slopes area, potential slide area, liquefaction-prone area, and known slide affected property.

5.1. Steep Slope Assessment

Based on our review, the area mapped as a steep slope ECA meets the requirements for relief from prohibition on steep slope development per SDCI Tip 327A, which states the relief can be granted (subject to ECA review) when the "development is located on steep slope areas that have been created through previous legal grading activities, including rockeries or retaining walls resulting from rights-of-way improvements, if no adverse impact on the steep slope area will result."

The proposed renovation at the site will consist of demolishing the existing structures, which will be set back from the steep slope area, filling the existing North Building basement to grade, and constructing a new six-story building. Given that the existing buildings are set back from the steep slope area, which was created as part of legal grading and consists of a crib retaining wall, we judge there will be no adverse impacts to the planned development or existing adjacent improvements.

5.2. Potential Slide Area Assessment

As previously discussed, the landslide mapped uphill and east of the site occurred in 1934, as shown in Appendix D; the landslide debris was subsequently removed by City of Seattle. This area was then re-graded as part of the construction of I-5 in the early 1960s, which is supported on deep foundation drilled shafts and included the construction of the crib retaining wall. We judge that the construction of the I-5 structure stabilized the landslide-prone area.

5.3. Liquefaction-Prone Area Assessment

We evaluated the potential for liquefaction at the site. Our analysis indicates that the very loose to medium dense fill soils and estuarine deposits below the groundwater table have a high potential for liquefaction during the design earthquake event. However, liquefaction will not be mitigated, but rather the building will be supported on deep foundations that transfer the building loads to the competent non-liquefying glacially consolidated soils below the liquefiable layer. The deep foundations will be designed for both downdrag due to liquefaction settlement and the seismic loading.

5.4. Known-Slide Affected Property Assessment

As discussed in Section 5.2, the previous slide occurred uphill and east of the site, i.e., did not extend onto the site. The construction of the I-5 structure stabilized the landslide-prone area.

6.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of the primary geotechnical considerations 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 is designated as seismic Site Class F per the 2018 IBC due to the liquefiable soils below the site. We understand that the planned structures will have a fundamental period of vibration of less than 0.5 seconds.
- To construct the new improvements, the existing buildings, including their basements, will be demolished. The new building will be constructed at-grade, which will require grades to be raised within the former North Building basement footprint. The existing basement walls can likely be left in place during demolition; this may require lateral restraint following removal of the ground floor slab and prior to the backfilling of the basement to grade. The lateral restraint could consist of either: (1) installing temporary bracing (kicker braces), or (2) providing a soil buttress on the inside of the basement walls to resist lateral movement. Alternatively, soldier pile shoring may be used to accommodate the demolition of the existing basement structures. The basement floor slab will need to be demolished or fractured in place to allow for subsurface drainage.
- The soils at the anticipated foundation elevation consist of the backfill placed during the demolition of the North Building. The fill is underlain by potentially liquefiable soil, which will likely experience settlement during a major earthquake. These conditions are not suitable for shallow foundation support. Competent glacially consolidated soils are generally present 20 to 30 feet bgs. We recommend that the planned building be supported on deep foundations, either micropiles or augercast piles, extending below the potentially liquefiable material and gaining capacity within the glacially consolidated material.
- The development plans for the southern portion of the site have not been finalized – it may either consist of a ½ level below-grade parking structure, or new parking at-grade. If the below-grade structure is selected, it should be supported on deep foundations like the new six-story building. If the parking is constructed at-grade, it should be designed in accordance with other surface improvements.
- Based on the groundwater information from previous exploration in the site vicinity, groundwater is anticipated to be present at Elevation 22 feet. GeoEngineers recommends that the design groundwater table elevation for the design be taken as Elevation 22 feet. As part of the demolition of the existing basements, which will require either the removal or fracturing the basement slabs, there may be seepage which we expect can be controlled using sumps and pumps. If the planned parking includes the ½ level basement, the new structure would extend below the design groundwater table and will be required to consider it in design. Permanent drainage is not likely feasible; therefore, the structure would need to be designed to resist hydrostatic pressures below the design groundwater table elevation.
- Buoyant pressures acting on the portion of the parking extending below the design groundwater table should be evaluated to confirm that the structure has adequate resistance to buoyant forces.
- We recommend that underslab utilities be structurally suspended from structural slabs to mitigate the potential for damage caused by liquefaction-induced settlement in the fill soils. Flexible connections should be considered for utilities entering the building due to significant potential seismic settlement of soils outside the building footprint.

Our specific geotechnical recommendations are presented in the following sections of this report.

6.1. Earthquake Engineering

6.1.1. Liquefaction

Liquefaction refers to the condition by which 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 general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table.

The results of our analyses indicate that the very loose to medium dense fill soils and estuarine 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. 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 in each of the borings we completed at the site. We estimate 1½ to 4 inches of liquefaction-induced settlement across the site for free field conditions. Liquefaction will not be mitigated, but rather the building will be supported on deep foundations that transfer the building loads to the competent non-liquefying glacially consolidated soils below the liquefiable layer. The deep foundations will be designed for both downdrag due to liquefaction settlement and the seismic loading.

6.1.2. Lateral Spreading

Lateral spreading involves lateral displacement of large, surficial blocks of soil as the underlying soil layer liquefies. The potentially liquefiable material is generally continuous; however, the lithology of these materials is generally level, and the nearest free face is approximately 1 mile to the west. Therefore, we conclude the potential for lateral spreading is considered to be low for the project site.

6.1.3. Other Seismic Hazards

Due to the location of the site and the site's topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

6.1.4. 2018 IBC Seismic Design Information

We evaluated the shear wave velocity of the upper 30 meters at the site (Vs30) based on the measurements obtained within the CPTs during our investigation. The project site is Site Class F due to the presence of liquefaction. Per American Society of Civil Engineers (ASCE) 7 16, Site Class F requires performing a site-specific site response analysis. However, we understand that the proposed structures will have a fundamental period of vibration of less than 0.5 second. Therefore, the exception of ASCE 7-16 Section 20.3.1 can apply for this project.

We recommend using the following 2018 IBC, and by reference ASCE 7-16 parameters based on Site Class D, short period spectral response acceleration (SS), 1-second period spectral response acceleration

(S_1) and seismic coefficients (F_a and F_v) for the project site as presented in Table 2. Please note that the Site Class F designation and associated requirements of ASCE 7-16 Chapter 12 still apply.

TABLE 2. ASCE 7-16 MAPPED SEISMIC DESIGN PARAMETERS

ASCE 7-16 Parameter[1,2]	Recommended Value
Site Class	F
Mapped MCE_R spectral response acceleration at short period, S_S (g)	1.435
Mapped MCE_R spectral response acceleration at 1-second period, S_1 (g)	0.500
Short-period site coefficient, F_a	1.00
Long-period site coefficient, F_v	1.80[2]
MCE_R spectral response acceleration at short period adjusted or site class effects, S_{MS} (g)	1.440
MCE_R spectral response acceleration at 1-second period adjusted or site class effects, S_{M1} (g)	0.900[2]
Design spectral response acceleration at short period adjusted or site class effects, S_{DS} (g)	0.960
Design spectral response acceleration at 1-second period adjusted or site class effects, S_{D1} (g)	0.600[2]

Notes:

[1] Parameters developed based on latitude 47.587515 and longitude -122.321046 using the ASCE 7 Hazards online tool (https://asce7hazardtool.online/).

[2] These values are valid for structures with fundamental periods less than 0.5 seconds.

MCE_R – risk-targeted maximum-considered earthquake

6.2. Temporary Dewatering

Temporary dewatering may be required during the demolition of the existing basements. We anticipate this can be achieved through the use of sumps and pumps.

One consideration associated with temporary dewatering is disposal of the dewatering effluent. Disposal options include discharge to the public combined sewer line, if feasible. Disposal of dewatering effluent will require that water quality requirements are met.

6.3. Excavation Support

We understand that that development may include a ½ level below-grade parking structure on the south portion of project. Shoring may be needed to accommodate the demolition of the existing basement structure and new parking structure construction. This could consist of a cantilevered soldier pile with lagging system. Alternatively, the existing basement walls may be left in place.

Where sufficient space is available, such as along the northern and eastern sides of the existing Southern Building basement, temporary cut slopes are considered feasible for the excavation, provided that the recommended inclinations are maintained between adjacent structures/walls and the base of the excavation. Temporary excavations should not encroach within a 1.5H:1V (horizontal to vertical) prism extending from the base of adjacent structures/walls.

The shoring system should be designed to limit deformations of the shoring wall and adjacent improvements to 1 inch or less. The City of Seattle requires that remedial measures be implemented when lateral deflections reach 1 inch.

6.3.1. Excavation Considerations

The site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers, however, the contractor should consider the site constraints in selecting the appropriate equipment. The contractor should be prepared for occasional cobbles and boulders in the site soils. Likewise, the surficial fill may contain foundation elements and/or utilities from previous site development, abandoned timber piles, waste timber, ballast, debris, rubble and/or cobbles and boulders. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with obstructions. The existing basement slabs should either be removed or fractured in place to accommodate the installation of new deep foundations and also to allow for drainage.

6.3.2. Cantilevered Soldier Piles

We recommend that cantilevered soldier pile walls be designed using the earth pressure diagram presented in Figure 5, Earth Pressure Diagrams—Temporary Cantilever Soldier Pile Wall. The pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 5 include the loading from traffic surcharge. No seismic pressures have been included in Figure 5 because it is assumed that the shoring will be temporary. Other surcharge loads, such as cranes, construction equipment, or construction staging areas can be determined using the recommended surcharge pressures presented on Figure 6.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 5 kips per square foot (ksf) for piles supported on fill/estuarine deposits and 30 ksf for piles supported on glacially consolidated soils. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement.

6.3.2.1. Lagging

Table 3 below presents recommended lagging thicknesses (roughcut) as a function of soldier pile clear span and depth.

TABLE 3. RECOMMENDED LAGGING THICKNESS FOR CLEAR SPANS

Depth (feet)	Recommended Lagging Thickness (Roughcut) for Clear Spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 18	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater is present or where clean sand and gravel soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable. The City of Seattle requires that voids be backfilled immediately or within a single shift, depending on the selected method of backfill.

Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Material used as backfill in voids located behind the lagging should not cause buildup of hydrostatic pressure behind the wall. Lean concrete or controlled density fill (CDF) are suitable options for use as backfill behind the walls. Lean concrete and CDF will reduce the volume of voids present behind the wall. Alternatively, lean concrete or CDF may be used for backfill behind the upper 5 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete or CDF lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

6.3.2.2. Drainage

Drainage for soldier pile and lagging walls is achieved through seepage through the timber lagging. Seepage flows at the bottom of the excavation should be contained and controlled to prevent loss of soil from behind the lagging.

6.3.2.3. Construction Considerations

Shoring construction should be completed by a qualified shoring contractor. A shoring contractor is qualified if they have successfully completed at least 10 projects of similar size and complexity in the Seattle/Bellevue area during the previous 5 years. Interested shoring contractors should prepare a submittal documenting their qualifications unless this requirement is waived by GeoEngineers. The shoring contractor’s superintendent should have a minimum of 3 years’ experience supervising soldier pile construction and shoring construction, and the drill operators and on-site supervisors should have a minimum of 3 years’ experience installing shoring. The personnel experience should be included in the qualifications submittal.

Temporary casing will be required to install the soldier piles. GeoEngineers should be allowed to observe and document the installation and testing of the shoring to verify conformance with the design assumptions and recommendations.

6.3.3. Shoring Wall Performance

Temporary shoring walls typically move up to 1 inch. Deflections and settlements are usually highest at the excavation face and decrease to negligible amounts beyond a distance behind the wall equal to the height of the excavation. Deflections of the shoring system can be affected by local variations in soil conditions (such as around side sewers) or may be the result of the workmanship of the construction for the shoring wall (completed by the shoring contractor). Given that some movement is expected, existing improvements located adjacent to the temporary shoring system will also experience movement. The deformations discussed above are not likely to cause structural damage to structurally sound existing improvements; however, cosmetic damage is possible (for instance, cracks in drywall finishes; widening of existing cracks; minor cracking of slabs-on-grade/hardscapes; cracking of sidewalks, curbs/gutter, and pavements/pavement panels; etc.). For this reason, it is important to complete pre-construction survey and photo documentation of existing buildings and nearby improvements, including the crib retaining wall on the eastern edge of the site, prior to shoring construction. Refer to Appendix C for more detailed recommendations for shoring monitoring and preconstruction surveying.

6.3.4. Temporary Cut Slopes

The stability of open-cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height, and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent improvements/work areas; could affect existing utilities; and could endanger personnel.

Temporary unsupported cut slopes more than 4 feet high in the fill and recent deposits may be inclined at maximum of 1.5H:1V. For open cuts at the site, we recommend that:

- No adjacent foundations, traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut;
- Exposed soil along the slope be protected from surface erosion by using waterproof tarps or plastic sheeting;
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable;
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- Surface water be diverted away from the slope; and
- The general condition of the slopes be observed daily by the general contractor and periodically by the geotechnical engineer to confirm adequate stability.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state, and federal safety regulations.

Temporary cut slopes should be planned such that they do not encroach on a 1.5H:1V influence line projected down from the edges of nearby or planned foundation elements.

Water that enters the excavation must be collected and routed away from prepared subgrade areas. We anticipate that this may be accomplished by installing a system of drainage ditches and sumps along the toe of the cut slopes. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting with appropriate ballast, should be used to protect these slopes during periods of wet weather. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using berms, drainage ditches, swales, or other appropriate methods.

6.4. Foundation Support

The soils at the anticipated foundation elevation for the new building will consist of newly-placed structural fill underlain by potentially liquefiable soils. The planned below-grade parking structure on the south would also be underlain by the same liquefiable soils. These conditions are not suitable for shallow foundation support. In addition, the underlying fill is liquefiable and subject to settlement. Competent glacially consolidated soils are generally present at depths between 20 and 30 feet below existing site grades.

Based on discussions with the project team, micropiles are likely the preferred option for foundation support of the new building given the thick layer of unsuitable/liquefiable soils, moderate structural loading, and depth to competent glacially consolidated soils. We also present recommendations for 24-inch-diameter augercast piles for the team's consideration.

6.4.1. Micropiles

For foundation support, 6- to 10-inch-diameter micropiles will be installed in plumb orientations. Structural detailing at the top of the piles is made to connect to the foundation.

6.4.1.1. Design Parameters

We recommend that 6- to 10-inch-diameter micropiles be drilled into the native glacially consolidated soils encountered at the project site. We recommend that the diameter of the micropiles be at least 6 inches and designed using a design load transfer (for side resistance) of 4 kips per foot for compressive and uplift capacity within the glacially consolidated soils; for 10-inch-diameter micropiles, a design load transfer of 6.7 kips per foot may be used. An allowable end bearing capacity of 40 ksf can be used for compressive loading. The design load transfer value and allowable end bearing capacity include a factor of safety of 2. The side resistance capacities should neglect contributions from the fill and estuarine deposits; therefore, the design load transfer (side resistance) should begin based on the top of the bearing layer as shown in Figure 4.

Downdrag loads induced by liquefaction during a major earthquake for the post-earthquake condition should be considered on downward compression loading. For a 6-inch-diameter micropile, the downward compression capacity should be reduced by a downdrag load of 35 kips. For a 10-inch-diameter micropile, the downward compression capacity should be reduced by a 60-kip downdrag load.

The capacities apply to single piles. We recommend a minimum pile spacing of 3 feet. In our opinion, if piles are spaced at least 3 feet on center, no reduction of axial capacity for group action is needed.

Micropiles have relatively small cross-section areas, and therefore have limited resistance to lateral loading and bending moments. Where used for lateral support, micropiles may be battered to resist lateral demand. Typical batter angles range from 5 to 30 degrees from vertical.

6.4.1.2. Installation Recommendations

We recommend that micropiles be installed by a competent foundation contractor experienced with this type of construction. Micropiles should be drilled with straight drilling equipment with sufficient torque to penetrate through the very dense glacial soils. Drilling mud should not be used unless approved by GeoEngineers before the start of construction.

After the hole is drilled to the planned depth, cuttings must be removed from the hole, either mechanically or by using pressurized air. Water should not be used to remove cuttings from the hole. The installation of each micropile should be observed by a representative from GeoEngineers. If the hole is within tolerance with respect to location, depth, and verticality, it should be grouted immediately using a proper grout mix. After grouting is completed, properly sized steel bars should be installed with centering devices.

6.4.1.3. Test Pile Program

We recommend that a test pile program be established to confirm that the required capacities of micropile foundations have been achieved. We recommend that at least one sacrificial pile load test be completed. Tension load tests should be completed in general accordance with ASTM D3689 Section 8 Procedure for Standard Test Methods for Deep Foundations Under Static Axial Tensile Load.

Pile load testing should be completed using a load frame capable of distributing large test loads into the near-surface soils without damaging existing improvements. Large test loads frequently cause damage to slabs-on-grade and other nearby improvements, and the location of pile load tests should be reviewed during the design phase to minimize impacts to existing improvements.

6.4.1.4. Deep Foundation Settlement

We estimate that the post-construction settlement of deep foundations, designed and installed as recommended, will be on the order of ½ inch or less. Maximum differential settlement should be less than about one-half the post-construction settlement. Most of this settlement will occur rapidly as loads are applied.

6.4.2. Augercast Piles

Augercast piles are constructed using a continuous-flight, hollow-stem auger attached to a set of leads supported by a crane or installed with a fixed-mast drill rig. The first step in the pile casting process consists of drilling the auger into the ground to the specified tip elevation of the pile. Grout is then pumped through the hollow stem during steady withdrawal of the auger, replacing the soils on the flights of the auger. The final step is to install a steel reinforcing cage into the column of fresh grout. One benefit of using augercast piles is that the auger provides support for the soils during the pile installation process, thus eliminating the need for temporary casing or drilling fluid.

Installation of augercast piles also produces minimal ground vibrations, which is beneficial given the proximity of the adjacent properties. Geotechnical recommendations for augercast piles are provided in the following sections.

6.4.2.1. Axial Capacity

Axial pile capacity is developed from side frictional resistance and end bearing for loads in compression. Uplift pile capacity is development from side frictional resistance.

We developed axial capacities for 24-inch augercast piles below in Table 4. Axial pile capacities were evaluated for three conditions:

1. Before earthquake (static conditions);
2. During earthquake; and
3. After earthquake.

The pile capacities were evaluated using allowable stress design (ASD) procedures and are for combined dead plus long-term live loads. Each of the three cases include a factor of safety of 2, per the Seattle Building Code. The allowable post-earthquake capacities include the effects of downdrag from liquefaction-induced settlement in the liquefiable fill and estuarine deposits around the pile.

Augercast pile capacities for static and seismic conditions are summarized in the following table. The pile lengths can be determined by the embedment depths needed to develop the required axial capacity in compression and tension. Pile embedment starts at the bearing soil elevation contours shown on Figure 4.

TBALE 4. 24-INCH DIAMETER AUGERCAST PILE ALLOWABLE AXIAL CAPACITIES

Embedment Depth in Bearing Soils (feet)	Pile Tip Elevation	Static Conditions		During Earthquake		Post-Earthquake	
		Compression (kips)	Uplift (kips)	Compression (kips)	Uplift (kips)	Compression (kips)	Uplift (kips)
40 feet	Elevation -20 ft	380	90	370	80	260	45
50 feet	Elevation -30 ft	500	140	480	130	380	95
60 feet	Elevation -40 ft	620	200	600	190	500	155

Notes:

[1] See Figure 4 for bearing soil elevation contours.

[2] Post-earthquake condition considers liquefaction and the effect of downdrag.

Pile capacities can be provided for additional bearing contour elevations at the request of the structural engineer. The capacities apply to single piles. If piles are spaced at least three pile diameters on center, as recommended, no reduction of axial capacity for group action is needed. The structural characteristics of pile materials and structural connections may impose limitations on pile capacities and should be evaluated by the structural engineer.

6.4.2.2. Lateral Capacity

Lateral loads can be resisted by soil pressure on the vertical piles and by the passive soil pressures on the pile cap. Because of the potential separation between the pile-supported foundation components and the underlying soil from settlement, base friction along the bottom of the pile cap should not be included in calculations for lateral capacity.

We recommend using the soil parameters presented in Table 5. If needed, we can prepare lateral pile capacities once the foundation type has been selected and lengths finalized during final design.

TABLE 5. LATERAL PILE DESIGN SOIL PARAMETERS

Soil Unit	Bottom Elevation of Soil Unit[1] (feet, NAVD88)	LPILE Soil Model	Effective Unit Weight, (pcf)	Friction Angle (degrees)	Cohesion, (psf)	K (pci)
Fill[2]	Elevation -1 feet	API Sand	52.6	34	0	20
Estuarine Deposits[2]	Varies from Elevation 15 to 10 feet	API Sand	52.6	32	0	100
Possession Age Glacial Deposits	Pile terminates in this unit- See Figure 4	API Sand	65	40	0	125

Notes:

[1] LPILE analysis assumes that the top of pile is at approximate Elevation 5 feet, NAVD88.

[2] P-multiplier of 0.1 used for liquefied fill and 0.3 for liquefied estuarine deposits below the groundwater, per WSDOT procedure.

pci – pounds per cubic inch

pcf - pounds per cubic foot

psf – pounds per square foot

We recommend that the passive soil pressure acting on the pile cap be estimated using an equivalent fluid density of 250 pounds per cubic foot (pcf) where the soil adjacent to the pile cap consists of adequately compacted structural fill. This passive resistance value includes a factor of safety of 1.5 and assumes a minimum lateral deflection of 1 inch to fully develop the passive resistance. Deflections that are less than 1 inch will not fully mobilize the passive resistance in the soil.

Shafts spaced closer than five shaft diameters apart will experience group effects that will result in a lower lateral load capacity for trailing rows of shafts with respect to leading rows of shafts for an equivalent deflection. We recommend that the lateral load capacity for trailing shafts in a shaft group spaced less than five pile diameters apart be reduced in accordance with the factors in Table 6.

TABLE 6. SHAFT P-MULTIPLIERS, P_M, FOR MULTIPLE ROW SHADING

Shaft Spacing[1] (in terms of shaft diameter)	P-Multipliers, P_m[2, 3]		
	Row 1 (leading row)	Row 2 (1st trailing row)	Row 3 and higher (2nd trailing row)
3D	0.8	0.4	0.3
5D	1.0	0.85	0.7

Notes:

[1] The P-multipliers in the table above are a function of the center to center spacing of shafts in the group in the direction of loading expressed in multiples of the shaft diameter, D.

[2] The values of P_m were developed for vertical shafts only per 2017 ASHTO LRFD Table 10.7.4-1.

[3] The P-multipliers are dependent on the shaft spacing and the row number in the direction of the loading to establish values of P_m for other shaft spacing values, interpolation between values should be conducted.

6.5. Slab Design

The new building slab will not extend below the groundwater table and therefore does not need to consider hydrostatic/uplift pressures but should consider the estimated liquefaction-induced settlement of up to 4 inches. If the slab cannot accommodate this estimated settlement, the slab should be designed as a structural slab. If the new parking structure includes the ½ level below grade, it will extend below the design groundwater table. As previously discussed, we do not expect that permanent drainage will be feasible. As a result, the portion of the structure extending below the design groundwater elevation should be designed to resist hydrostatic/uplift pressures.

The uplift force acting on the proposed parking structure can be estimated by multiplying the volume of the structure located below the design groundwater elevation, in cubic feet, by the unit weight of water, 62.4 pcf. We assume that resistance to the uplift force will be provided by the weight of the structure. Where moisture or water may be detrimental to the performance of the below-grade structure, a waterproofing consultant should be engaged.

6.5.1. Subgrade Preparation

If the new structure will be supported on-grade, the subgrade should be thoroughly compacted to a uniformly firm and unyielding condition. Probing should be used to evaluate the subgrade. The exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

A 4-inch-thick capillary break layer is recommended below the building slab. The capillary break material should meet the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

6.5.2. Vapor Barrier

A vapor barrier should be used below slab-on-grade floors located in occupied portions of the buildings. Specification of the vapor barrier requires consideration of the performance expectations of the occupied space, the type of flooring planned and other factors, and is typically completed by other members of the project team.

6.6. Underslab Utility Support

We recommend that underslab utilities be structurally suspended from structural slabs to mitigate the potential for damage caused by liquefaction-induced settlement in the fill and estuarine soils. Pea gravel should be placed as backfill above the underslab utilities in order to reduce the soil loads acting on the suspended utilities. The pea gravel is anticipated to flow around the suspended utilities as settlement occurs. Utility connections into the pile supported building should be designed with flexible connections.

6.7. Below-Grade Walls

Permanent basement walls should be checked for seismic conditions, per the 2018 Seattle Building Code. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures (Sitar et al. 2012; Lew et al. 2010). We used the procedures outlined in Sitar et al. (2012) and the peak ground acceleration based on the Design Earthquake (DE) ground motion level, corresponding to a peak ground acceleration of 0.38g, to compute the seismic pressure increment. We conclude that the basement walls should be designed for the equivalent fluid weights (triangular distribution) presented in Table 6.

If the basement walls are designed as drained, they should be designed in accordance with the recommendations in Section 6.7.1. If backdrains are not used, the basement walls should be designed to accommodate full hydrostatic pressure acting on the wall using the values presented in Table 6 for "undrained." Undrained walls should be waterproofed to protect against moisture migration.

TABLE 6. LATERAL EARTH PRESSURES FOR BELOW-GRADE WALLS

Wall Condition	Equivalent Fluid Weights for Static Conditions		Seismic Conditions[1]
	Unrestrained Walls (Active)	Restrained Walls (At-Rest)	Total Pressure – Active Plus Seismic Pressure Increment
Drained	35	55	60
Undrained	75	85	90

Lateral resistance for below-grade walls can be provided by passive resistance in front of the wall. The allowable passive resistance may be computed using an equivalent fluid density of 250 pcf (triangular distribution). These values are appropriate for foundation elements that are surrounded by structural fill or on-site soils. The above passive equivalent fluid density value is for saturated soil conditions and incorporates a factor of safety of about 1.5.

6.7.1. Drainage

Drainage behind the permanent below-grade walls is typically provided using prefabricated drainage board attached to the temporary shoring walls. Weep pipes that extend through the permanent below grade wall should be installed around the perimeter of the building at the foundation elevation. The weep pipes through the permanent below grade wall should have a minimum diameter of 2 or 4 inches and be spaced no more than 10 or 20 feet on center, respectively, and should be hydraulically connected to the sump.

Prefabricated vertical geocomposite drainage material, such as Aquadrain 15X, should be installed vertically to the face of the timber lagging. The vertical drainage material should extend to the bottom of foundation elevation. The weep pipes that penetrate the basement wall should be connected to the vertical drainage material with a drain grate. For soldier pile shoring walls, the drainage material should be installed on the excavation side of the timber lagging, with the fabric adjacent to the timber lagging.

Where basement walls are drained, full wall face coverage is recommended to minimize seepage and/or wet areas at the face of the permanent wall. Full wall face coverage should extend from the bottom of foundation elevation up to about 3 to 5 feet below site grades to reduce the potential for surface water to enter the wall drainage system. Although the use of full wall face coverage will reduce the likelihood of seepage and/or wet areas at the face of the permanent wall, the potential still exists for these conditions to occur. If this is a concern, waterproofing should be specified.

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9 03.14, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger diameter pipe will allow for easier maintenance of drainage systems.

6.8. Earthwork

6.8.1. Clearing and Stripping

Based on our observations at the site, we do not expect significant stripping work for this site which consists of asphalt concrete (AC) and portland cement concrete (PCC) surfacing. The AC can be used as structural fill provided that it is free of organic matter, rebar, and any other deleterious material. In addition, recycled AC for use as structural fill should be broken down to particle size smaller than 2 inches.

During demolition of existing structures/pavements or hardscaping, excessive disturbance of surficial soils may occur, especially if left exposed to wet conditions. Disturbed soils may require additional remediation during construction and grading.

6.8.2. Subgrade Preparation

Subgrade areas that will support slab-on-grade floors, roadways, and parking areas should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping and prior to placing structural fill. We recommend that subgrade areas be compacted to at least 95 percent of the theoretical maximum dry density (MDD) per ASTM International (ASTM) D 1557. We recommend that subgrades for structures and roadways be evaluated, as appropriate, to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

6.8.3. Subgrade Protection and Wet Weather Protection

During wet weather, some of the exposed soils could become muddy and unstable. The wet weather season generally begins in October and continues through May in western Washington; however, periods of wet weather can occur during any month of the year. The optimum earthwork period is typically June through September. If wet weather earthwork is unavoidable, we recommend the following:

- 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 so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- 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 other 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 exposed soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.

Protective surfacing such as placing asphalt-treated base (ATB), or haul roads made of quarry spalls or a layer of free-draining material such as well-graded pit-run sand and gravel may be necessary to protect completed areas. Minimum quarry spall thicknesses should be on the order of 12 to 18 inches. Typically, minimum gravel thicknesses on the order of 18 inches are necessary to provide adequate subgrade protection.

6.8.4. Structural Fill

Fill placed to support structures, placed behind retaining structures, and placed below pavements and sidewalks will need to be specified as structural fill as described below:

- Structural fill placed behind cast-in-place retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.19.
- Structural fill placed as capillary break should meet the requirements of Mineral Aggregate Type 22 (3/4-inch crushed gravel), City of Seattle Standard Specification 9-03.19.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.19.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.19.

6.8.4.1. On-site Soils

The on-site soils are moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. If the contractor wants to use on-site soils for structural fill, GeoEngineers can evaluate the on-site soils for suitability as structural fill, as required.

6.8.4.2. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in building areas (supporting foundations or slab-on-grade floors) and in pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the MDD estimated in general accordance with ASTM D 1557.
- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against subsurface walls to avoid over-compaction and hence overstressing the walls.

We recommend that GeoEngineers be present during probing of the exposed subgrade soils in building and pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

6.9. Recommended Additional Geotechnical Services

GeoEngineers, Inc. should be retained to prepare a final geotechnical engineering report and review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should observe the installation of the foundations, review/collect shoring monitoring data, evaluate the suitability of the subgrade, observe installation of subsurface drainage measures, evaluate structural backfill, observe the condition of temporary cut slopes, and provide a summary letter of our construction observation services. The purpose 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 E, Report Limitations and Guidelines for Use.

7.0 LIMITATIONS

We have prepared this report for the exclusive use of Evergreen Treatment Services and their authorized agents for the Seattle Clinic Renovation project in Seattle, Washington.

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 E for additional information pertaining to use of this report.

8.0 REFERENCES

Aspect Consulting. "Phase II Environmental Site Assessment, 1700 Airport Way South, Seattle, WA".

City of Seattle, 2023, "Standard Specifications for Road, Bridge and Municipal Construction."

International Code Council, 2018, "International Building Code."

Robert M. Pride, LLC, 2005. "Report on Geotechnical Investigation, Proposed Office Workspaces, 1762 Airport Way South."

Shannon & Wilson, Inc, 2004. "Geotechnical Report, Metro's Atlantic/Central Base Expansion, Seattle, Washington.

Washington State Department of Transportation, 2023, "Standard Specifications for Road, Bridge and Municipal Construction."

FIGURES

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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.

Vicinity Map

Seattle Clinic Renovation
Seattle, Washington

GeoEngineers

Figure 1

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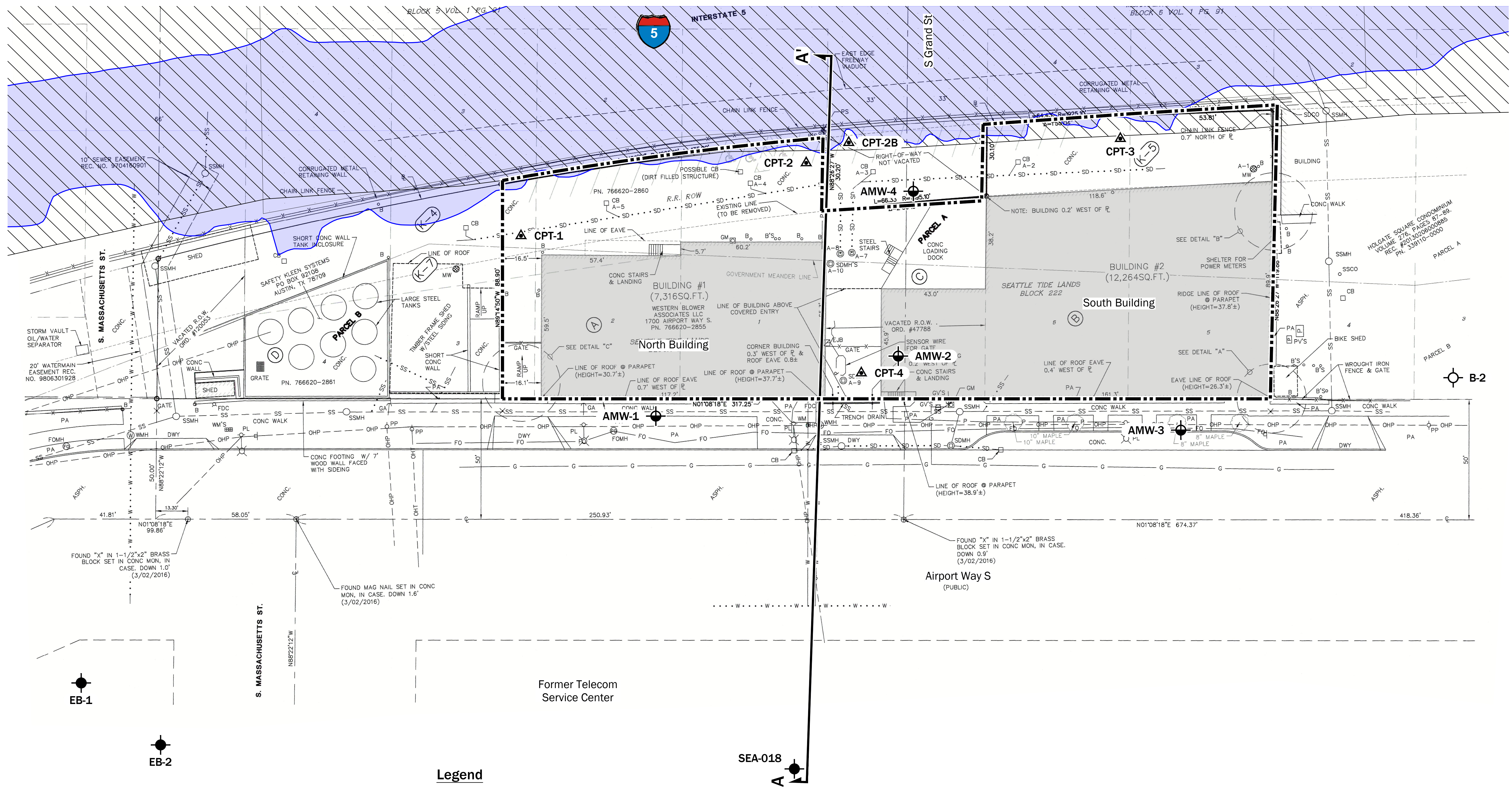

Legend

Site Extents

CPT-1 ▲ Approximate Location of Cone Penetration Test by GeoEngineers, May 2023

AMW-1 ⊕ Approximate Location of Boring by Aspect Consulting, 2019

EB-1 ● Approximate Location of Boring by Shannon & Wilson, February 2004

B-2 ⊕ Approximate Location of Boring by Price LLC, March 2007

🟪 Sleep Slope ECA1 per SMC Chapter 25.09

▨ Potential Slide Area ECA2 per SMC Chapter 25.09

☐ Known Slide ECA8 per SMC Chapter 25.09

A A' Cross Section Location

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background from David Evans and Associates Inc., dated 09/28/2020.

Projection: Washington State Plane, North Zone, NAD83, US Foot

Site Plan

Seattle Clinic Renovation
Seattle, Washington

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Figure 2

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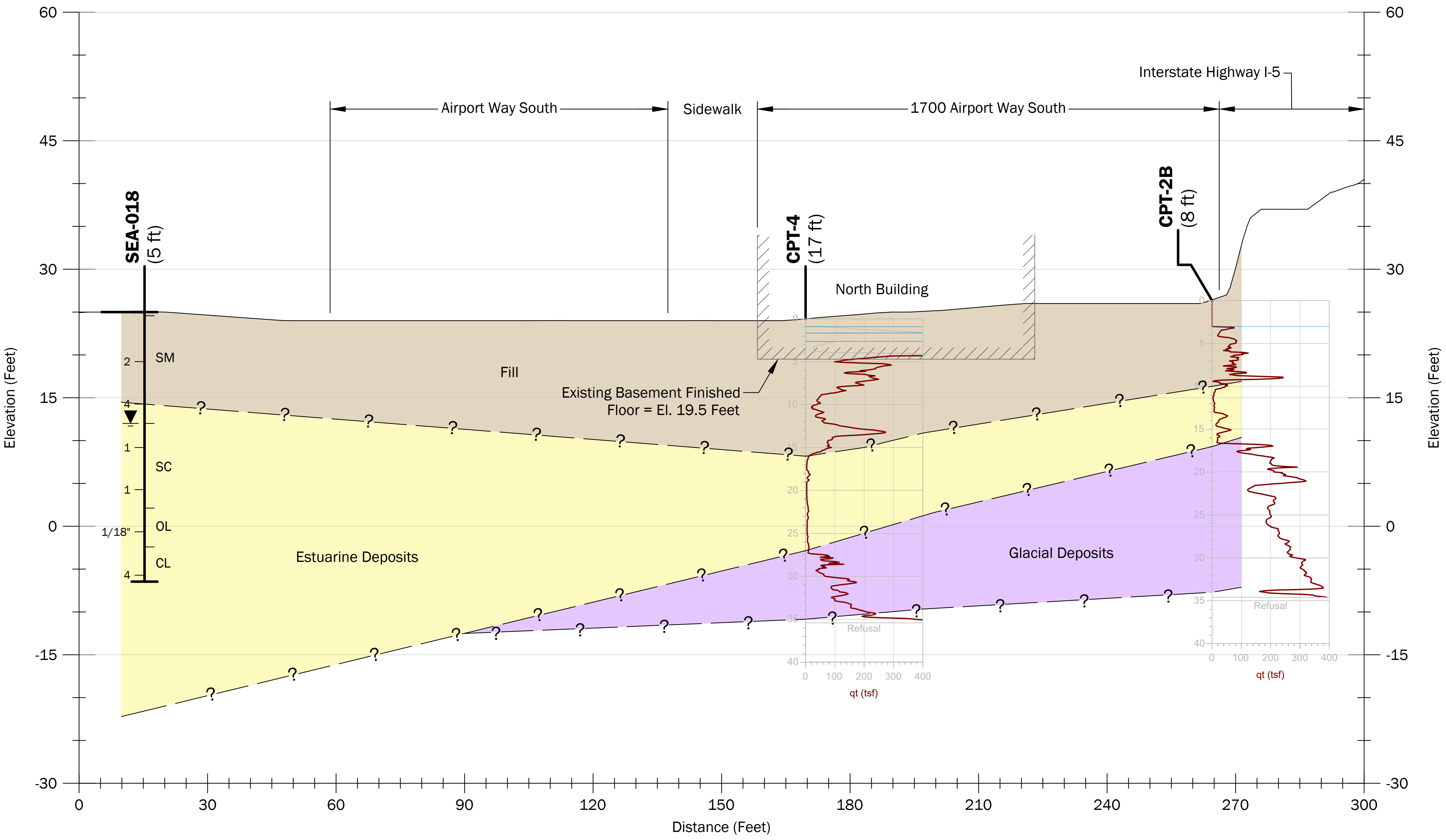
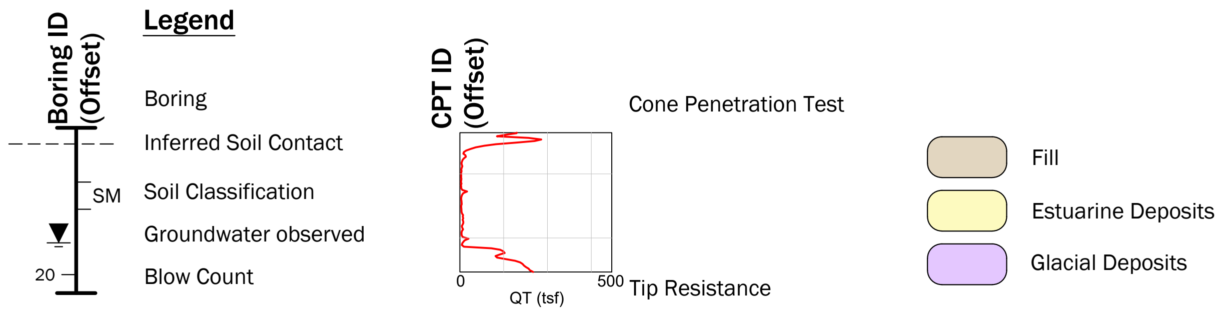


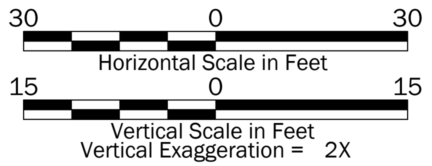
Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Datum: NAVD 88, unless otherwise noted.

Legend





Cross Section A-A'

Seattle Clinic Renovation
Seattle, Washington

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Figure 3

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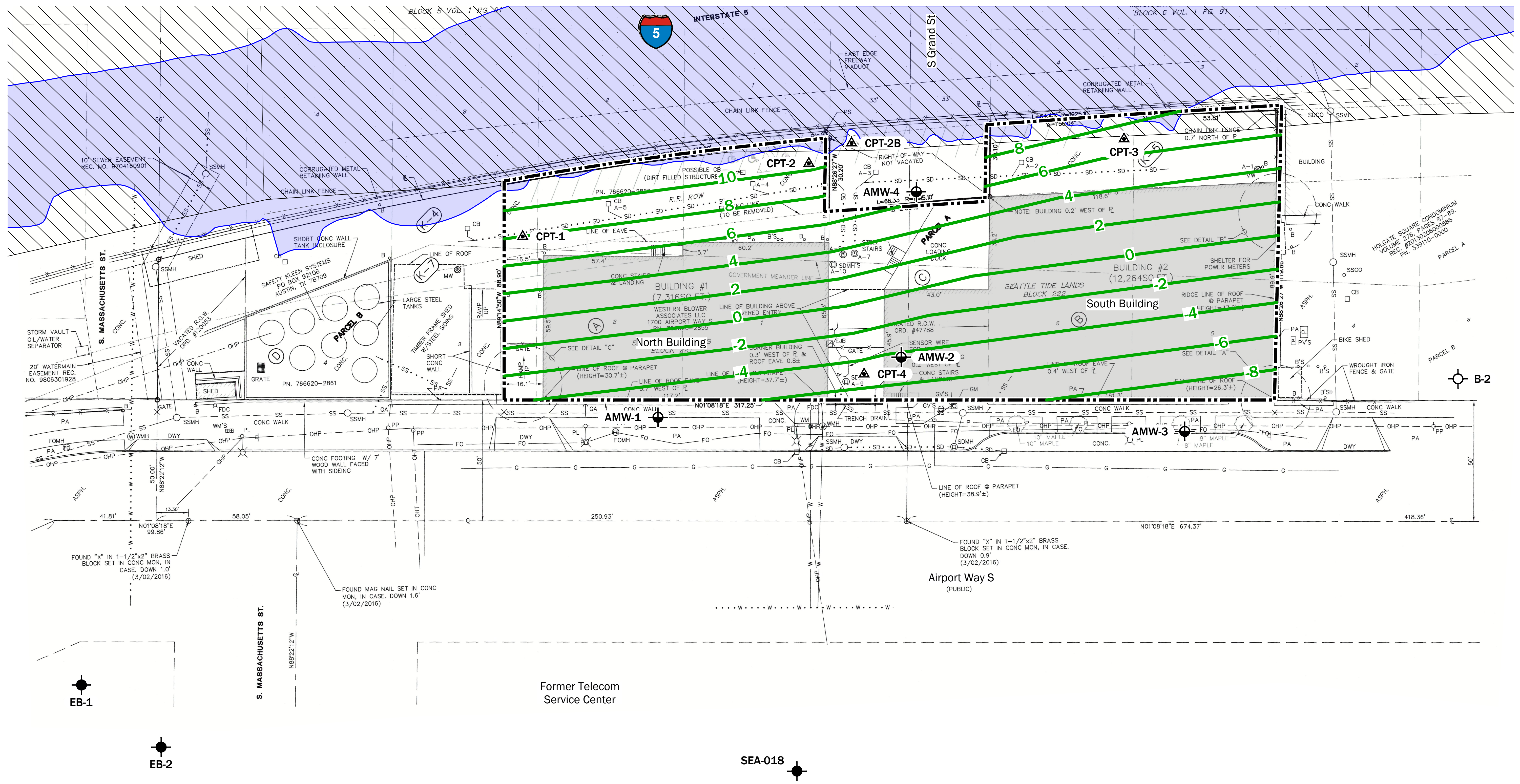

Legend

▬ · ▬ · Site Extents

—4— Bearing Contours

CPT-1 ▲ Approximate Location of Cone Penetration Test by GeoEngineers, May 2023

AMW-1 ⊕ Approximate Location of Boring by Aspect Consulting, 2019

EB-1 ⊕ Approximate Location of Boring by Shannon & Wilson, February 2004

B-2 ⊕ Approximate Location of Boring by Price LLC, March 2007

🟦 Sleep Slope ECA1 per SMC Chapter 25.09

▨ Potential Slide Area ECA2 per SMC Chapter 25.09

░ Known Slide ECA8 per SMC Chapter 25.09

40 0 40
Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background from David Evans and Associates Inc., dated 09/28/2020.

Projection: Washington State Plane, North Zone, NAD83, US Foot

Bearing Contour Map

Seattle Clinic Renovation
Seattle, Washington

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Figure 4

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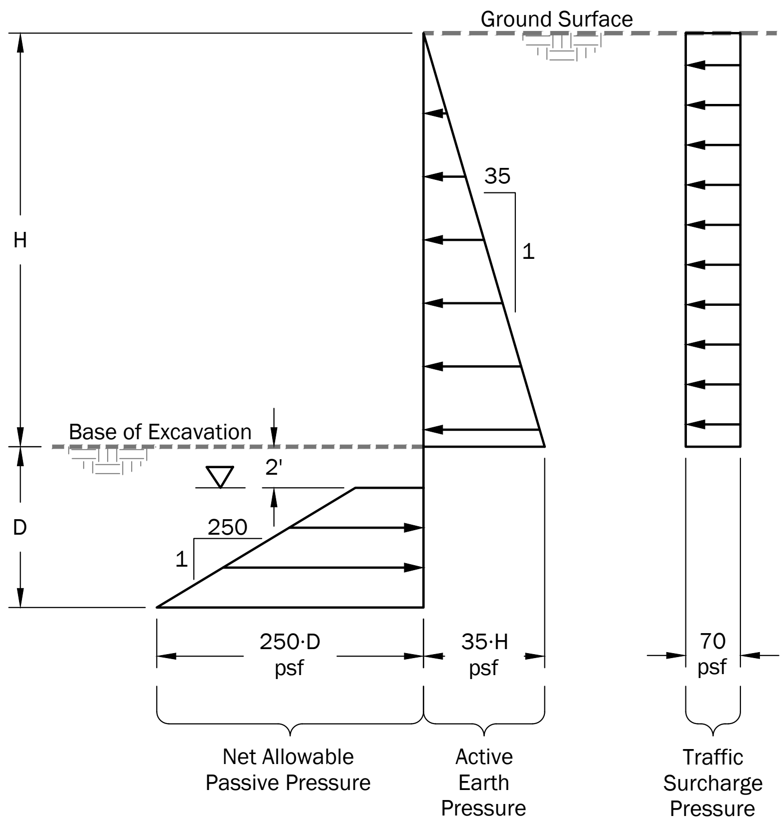
Cantilever Soldier Pile



Legend

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Notes:

1. Active earth pressure and traffic surcharge pressure act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5 and includes the net pressure (difference between active and passive) below the base of excavation.
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
5. This pressure diagram is appropriate for temporary soldier pile and tieback walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Not To Scale

Earth Pressure Diagrams - Temporary
Cantilever Soldier Pile Wall

Seattle Clinic Renovation
Seattle, Washington

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Figure 5

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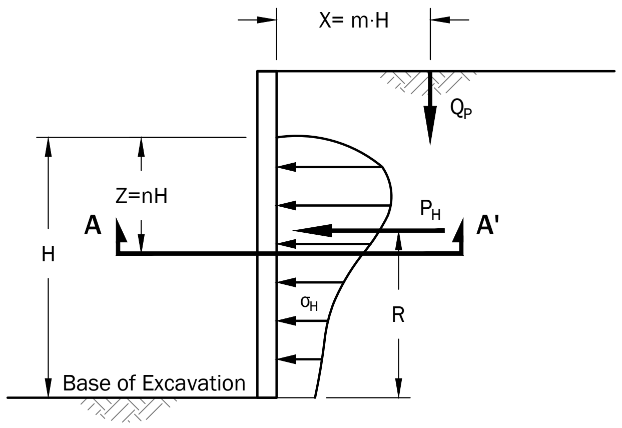
Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For $m \le 0.4$

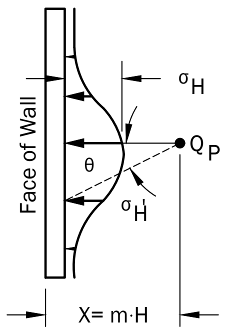
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

For $m > 0.4$

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

$$\sigma'_H = \sigma \ COS^2 (1.1\theta)$$

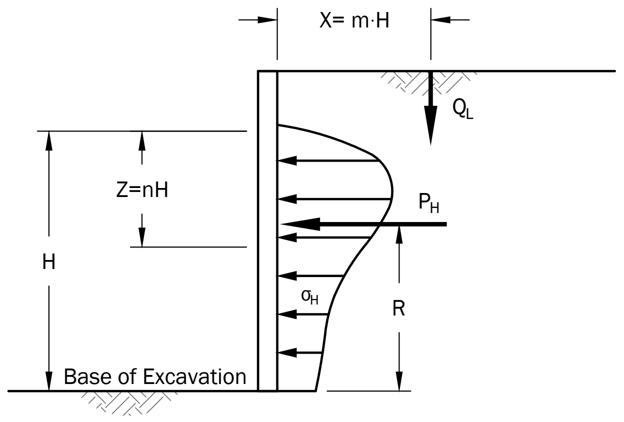
m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



Section A-A'

Pressures from Point Load Q_P

Lateral Earth Pressure from Line Load, Q_L (Continuous Wall Footing)



For $m \le 0.4$

$$\sigma_H = \frac{0.2n \cdot Q_L}{H(0.16+n^2)^2}$$

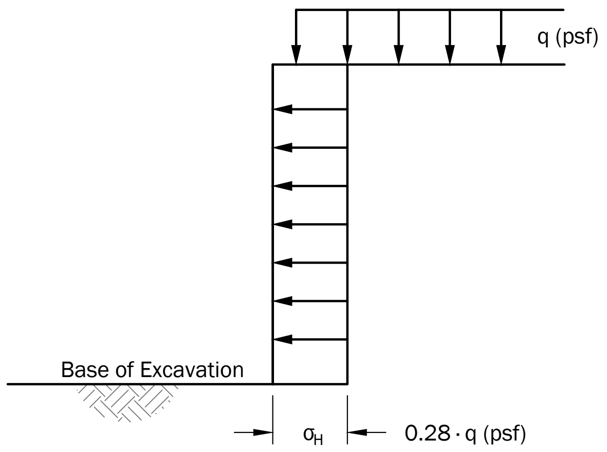
For $m > 0.4$

$$\sigma_H = \frac{1.28 m^2 n Q_L}{H(m^2+n^2)^2}$$

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

Uniform Surcharges, q (Floor Loads, Large Foundation Elements or Traffic Loads)



σ_H = Lateral Surcharge Pressure from Uniform Surcharge

Definitions:

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ'_H = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet
X = Resultant lateral force acting on wall, pounds
Z = Depth of σ_H to be evaluated below the bottom of Q_P or Q_L
m = Ratio of X to H
n = Ratio of Z to H

Notes:

1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figure 3.
3. See report text for where surcharge pressures are appropriate.

Not To Scale

Recommended Surcharge Pressure

Seattle Clinic Renovation
Seattle, Washington

GeoEngineers

Figure 6

P:\26\26686001\CAD\00\Geotech Report\2668600100_F05-F06_EPD and RSP.dwg TAB:F06 Date Exported: 07/21/23 - 11:28 by gregister

APPENDICES

GeoEngineers

APPENDIX A

Results of Cone Penetration Testing (Conetec, 2023)

PRESENTATION OF SITE INVESTIGATION RESULTS

ETS New Improvements CPT

Prepared for:

GeoEngineers, Inc.

ConeTec Job No: 23-59-25575

Project Start Date: 27-Apr-2023
Project End Date: 04-May-2023
Report Date: 16-May-2023

Prepared by:

ConeTec Inc.
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Seattle, WA 98108

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ConeTecWA@conetec.com
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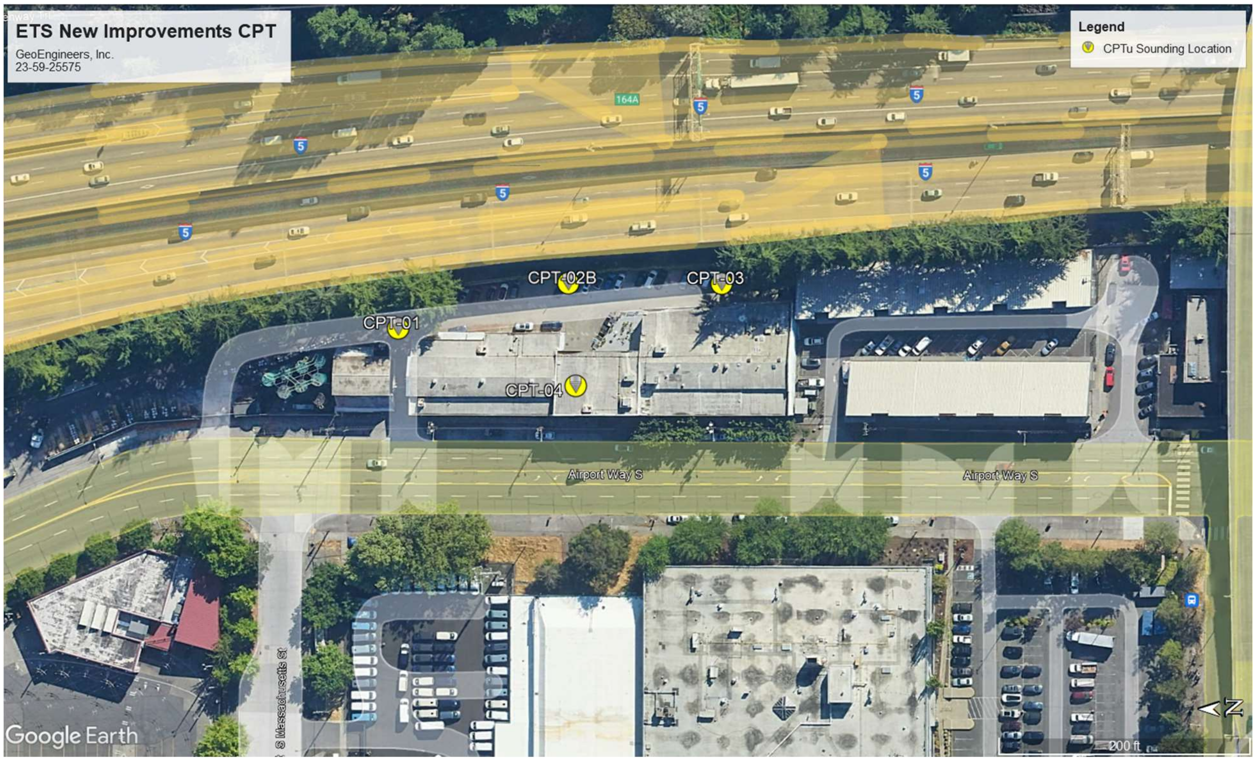
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for GeoEngineers, Inc. at 1700 Airport Way S, Seattle, WA 98134. The program consisted of two (2) cone penetration tests and two (2) seismic cone penetration tests. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information

Project	
Client	GeoEngineers, Inc.
Project	ETS New Improvements CPT
ConeTec project number	23-59-25575

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



Rig Description	Deployment System	Test Type
C02-023_25-Ton Truck Rig	Integrated Push Cylinders	CPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu	Consumer grade GPS	4326

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
907:T1500F15U35	907	15	225	1500	15	35
Cone 907 was used for all CPTu soundings						

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 meter This has been accounted for in the CPT data files.
Additional plots	• Advanced plots with Ic, Su(Nkt)/Su(Ndu), Phi and N(60)/N1(60) • Soil Behaviour Type (SBT) scatter plots • Seismic shear wave (Vs) plots • Seismic shear wave (Vs) Wave Trace plots

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

Limitations

3rd Party Disclaimer

This report titled “ETS New Improvements CPT”, referred to as the (“Report”), was prepared by ConeTec for GeoEngineers, Inc.. 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, Inc. to collect and provide the raw data (“Data”) which is included in this report titled “ETS New Improvements CPT”, which is referred to as 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. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. 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.

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

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

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

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

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

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

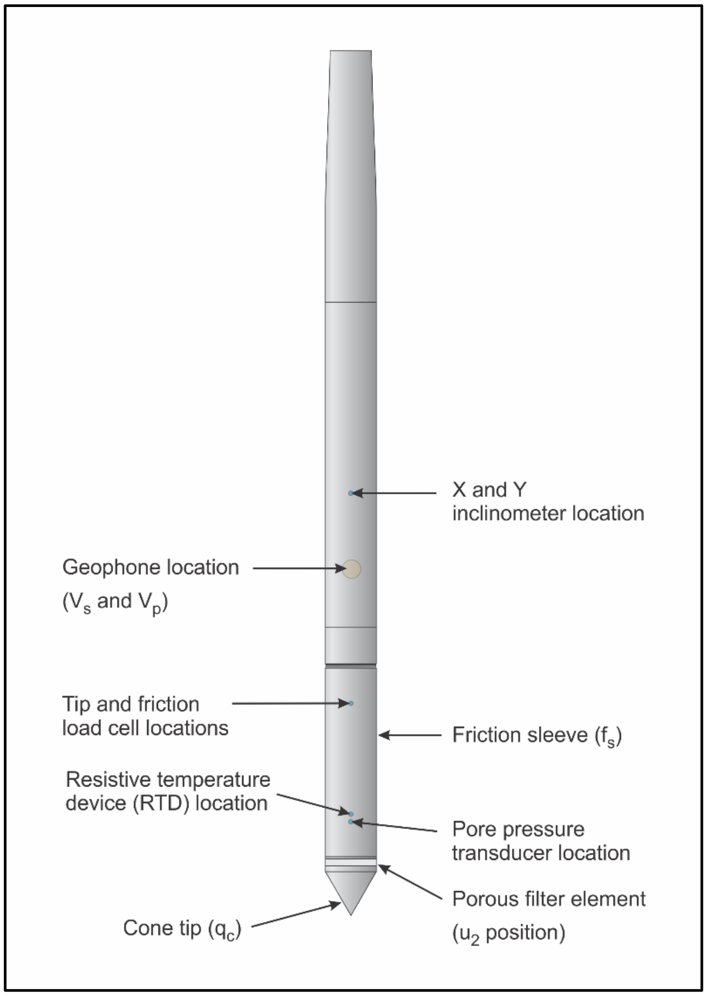


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

ConeTec's 15 cm^2 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; however, it is often affected by the compression wave travelling through the cone rods.

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.

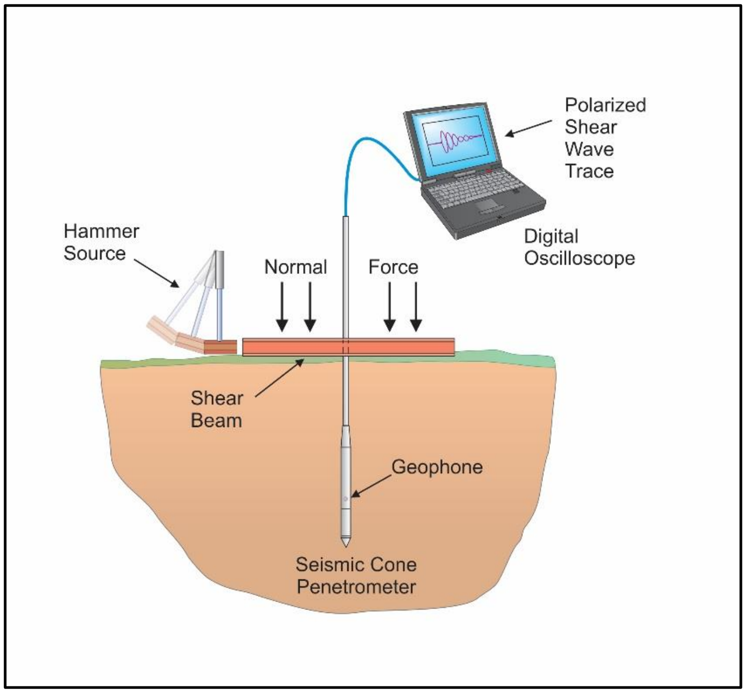


Figure SCPTu-1. Illustration of the SCPTu system

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).

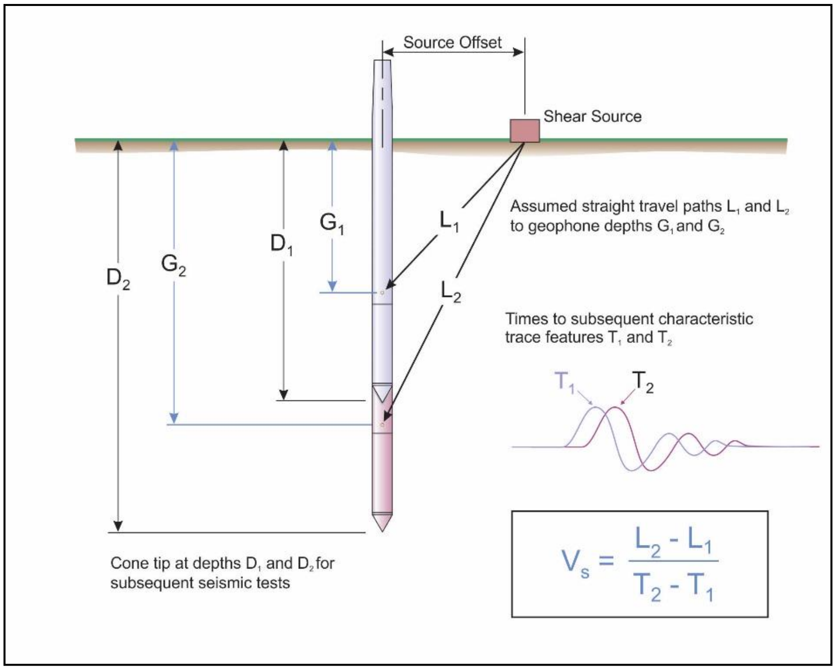


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the 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 ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

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

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)

d_i = the thickness of any layer between 0 and 100 ft (30 m)

v_{si} = the shear wave velocity in ft/s (m/s)

$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

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

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

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

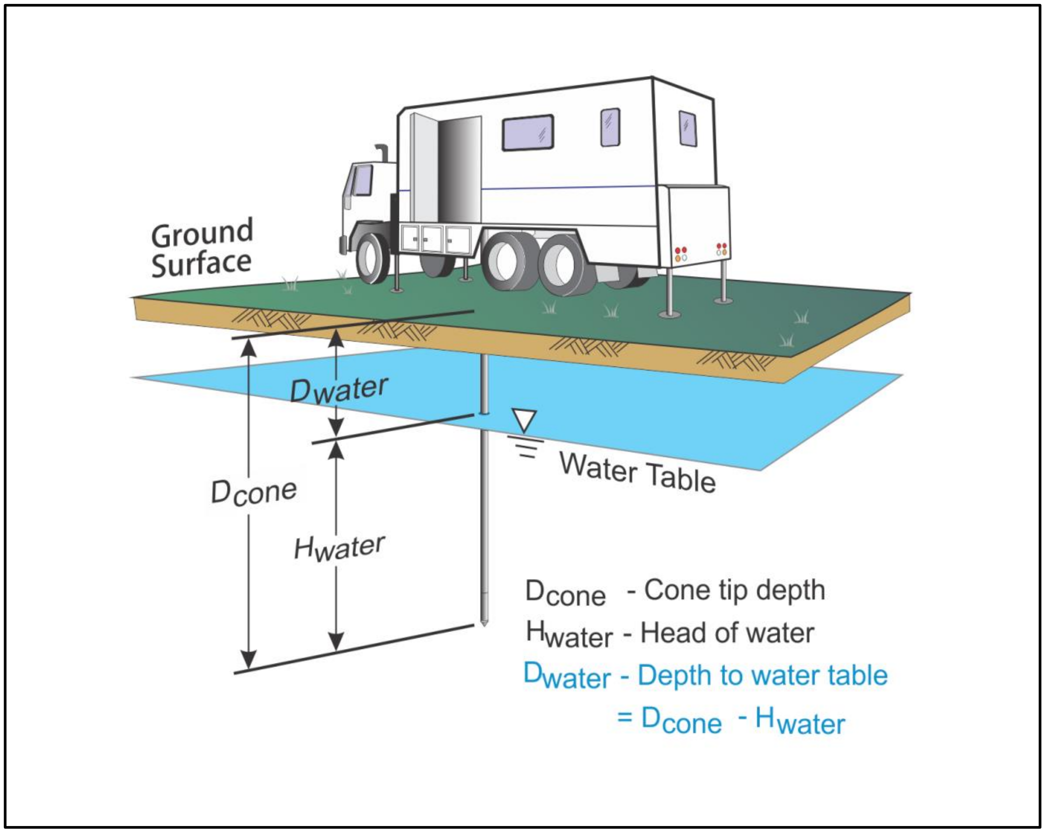


Figure PPD-1. Pore pressure dissipation test setup

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

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

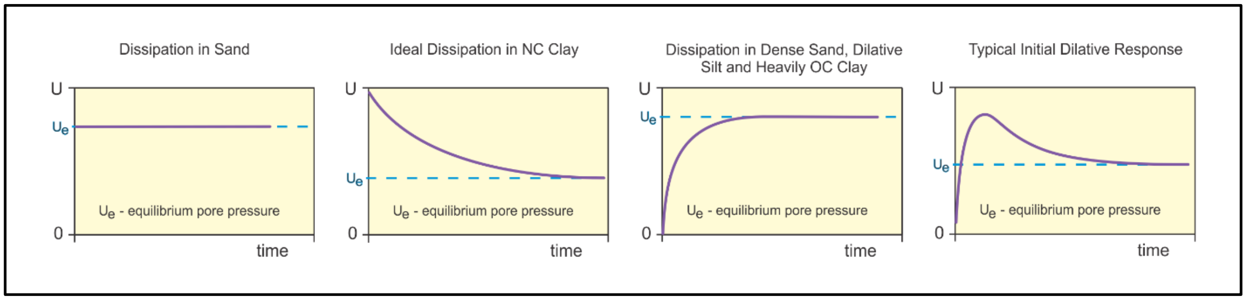


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

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

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

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

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

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

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

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

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/D5778-12.

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

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., 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).

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

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

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Su(Ndu), Phi, N(60) and N1(60)
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 23-59-25575
Client: GeoEngineers, Inc.
Project: ETS New Improvements CPT
Start Date: 27-Apr-2023
End Date: 04-May-2023

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed [1] Phreatic Surface (ft)	Final Depth (ft)	Shear Wave Velocity Tests	Latitude[2] (deg)	Longitude[2] (deg)	Refer to Notation Number
CPT-01	23-59-25575_SP01	27-Apr-2023	907:T1500F15U35	1.6	42.0	14	47.58799	-122.32093	
CPT-02B	23-59-25575_CP02B	04-May-2023	907:T1500F15U35	2.3	34.6		47.58762	-122.32078	
CPT-03	23-59-25575_CP03	28-Apr-2023	907:T1500F15U35	6.0	44.9		47.58728	-122.32078	3
CPT-04	23-59-25575_SP04	28-Apr-2023	907:T1500F15U35	6.0	35.4	10	47.58760	-122.32111	
Totals	4 soundings				156.9	24			

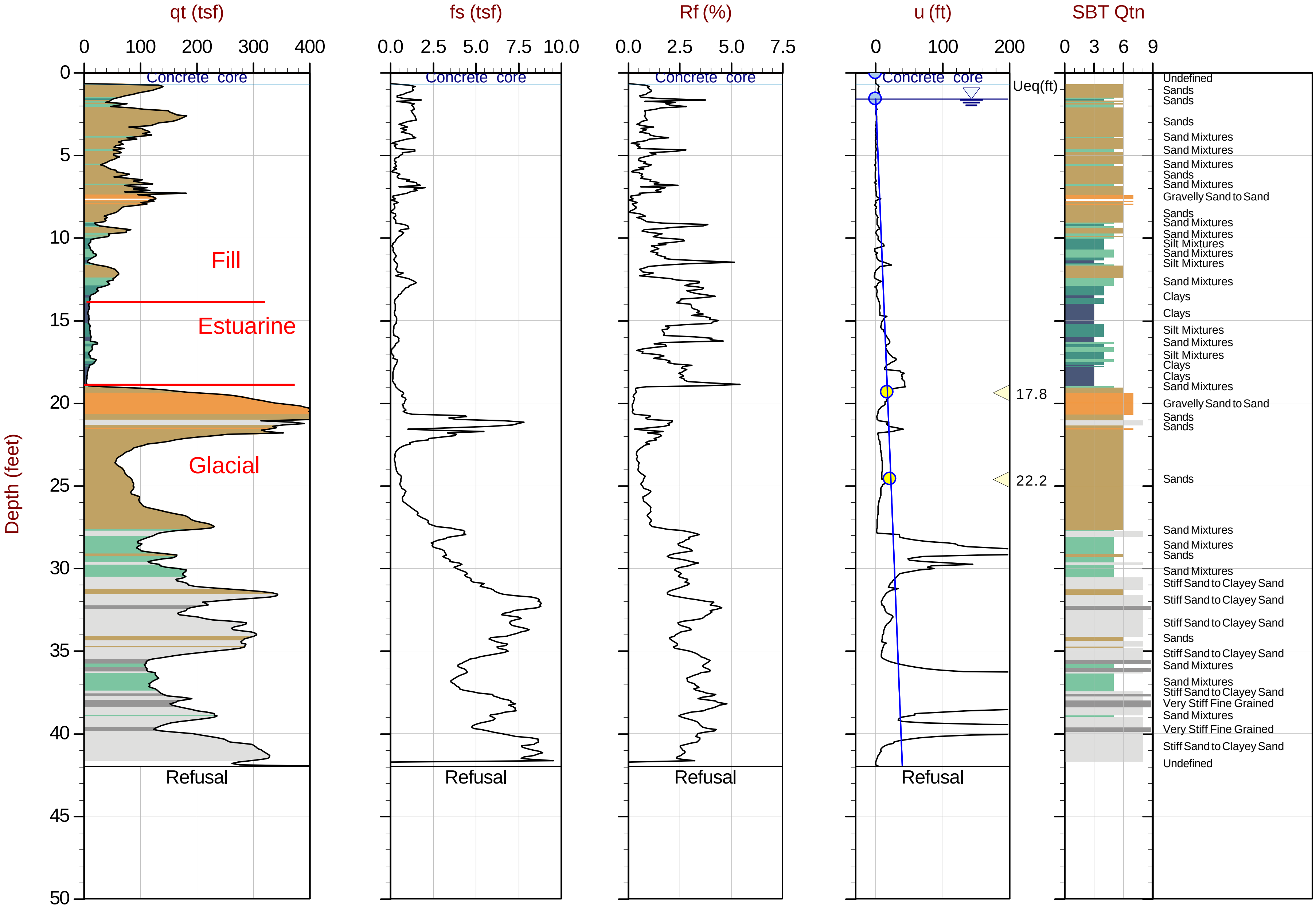
1. Phreatic surface based on pore pressure dissipation test unless otherwise noted. Hydrostatic profile applied to interpretation tables
2. Coordinates were collected using a consumer grade GPS - WGS 84 Lat/Long
3. Phreatic surface based on pore pressure dissipation test performed at an adjacent CPT location

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-27 14:18
Site: ETS New Improvements CPT

Sounding: CPT-01
Cone: 907:T1500F15U35



Max Depth: 12.800 m / 41.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58799 Long: -122.32093

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

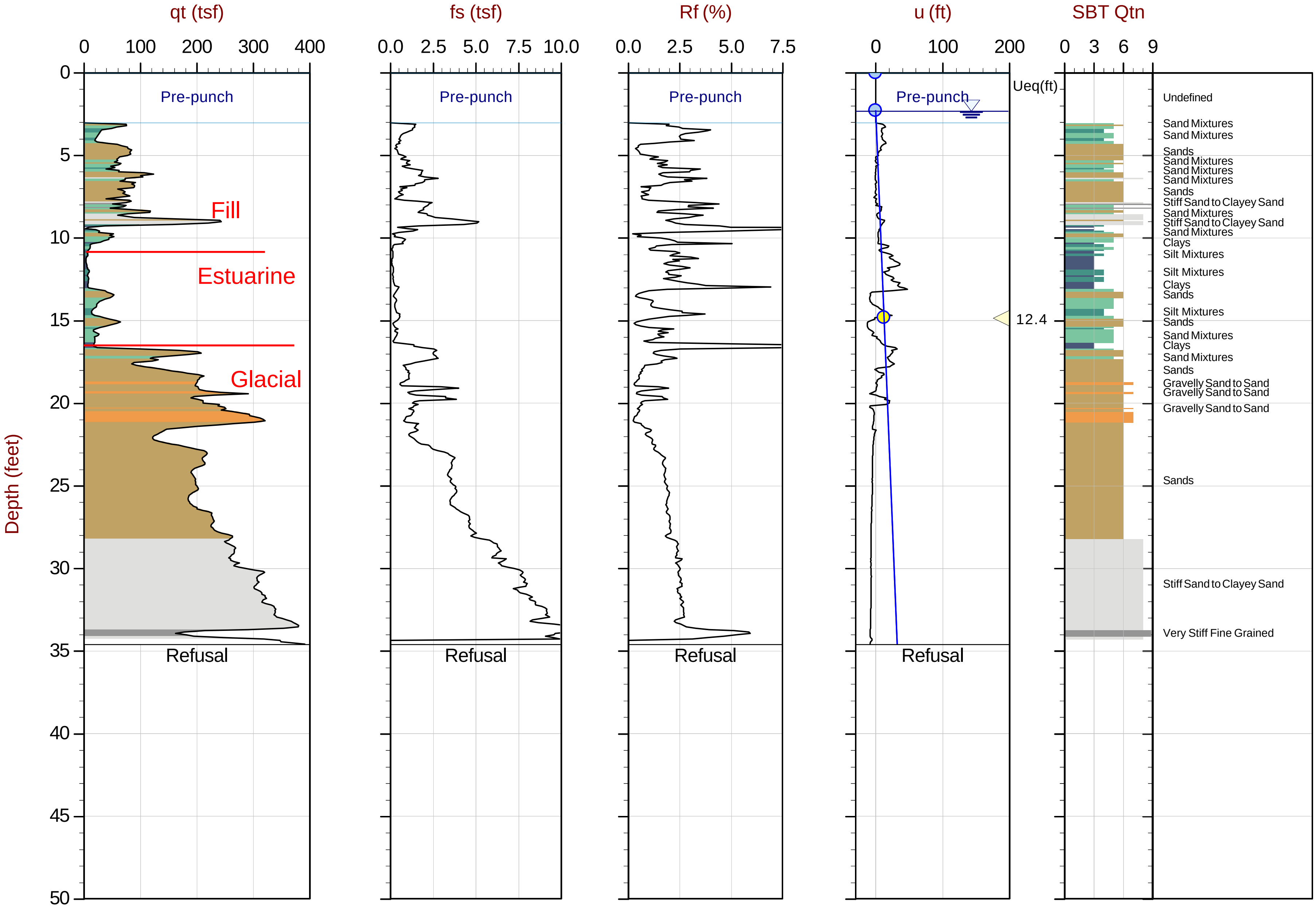
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-05-04 14:28
Site: ETS New Improvements CPT

Sounding: CPT-02B
Cone: 907:T1500F15U35



Max Depth: 10.550 m / 34.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58762 Long: -122.32078

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

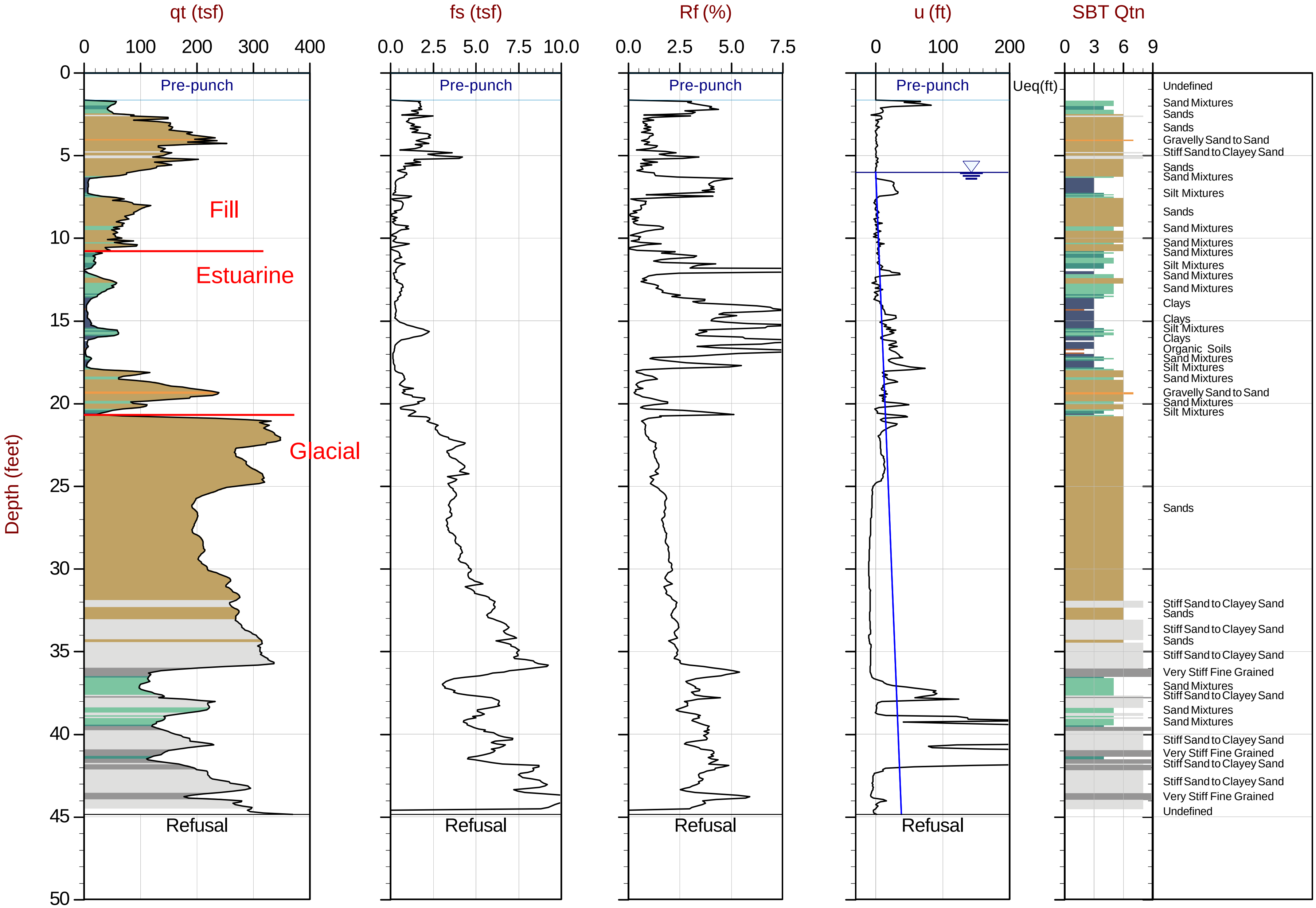
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-28 10:39
Site: ETS New Improvements CPT

Sounding: CPT-03
Cone: 907:T1500F15U35



Max Depth: 13.675 m / 44.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58728 Long: -122.32078

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

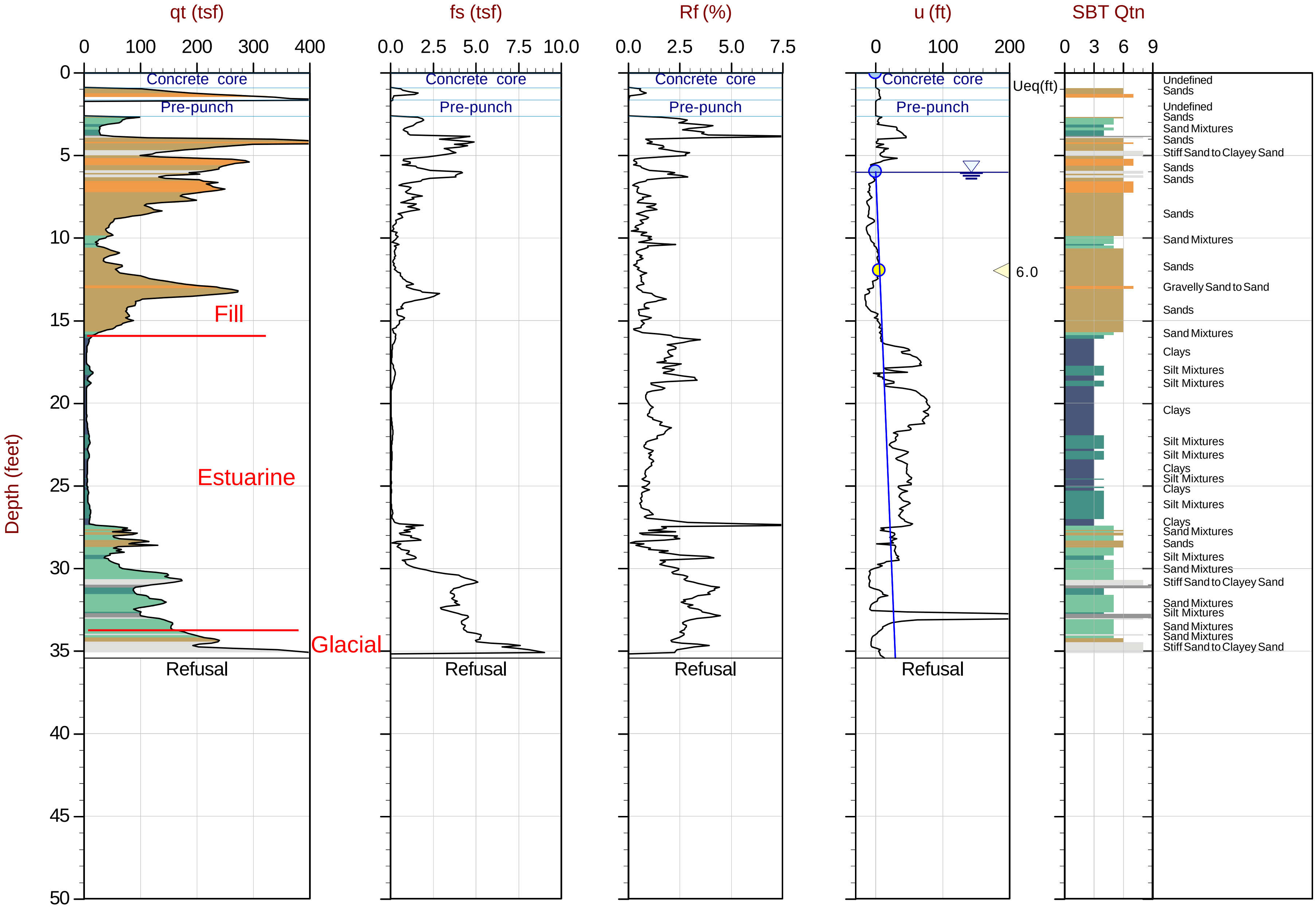
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-28 08:42
Site: ETS New Improvements CPT

Sounding: CPT-04
Cone: 907:T1500F15U35



Max Depth: 10.800 m / 35.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58760 Long: -122.32111

● 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), Su(Ndu), Phi, N(60) and N1(60)

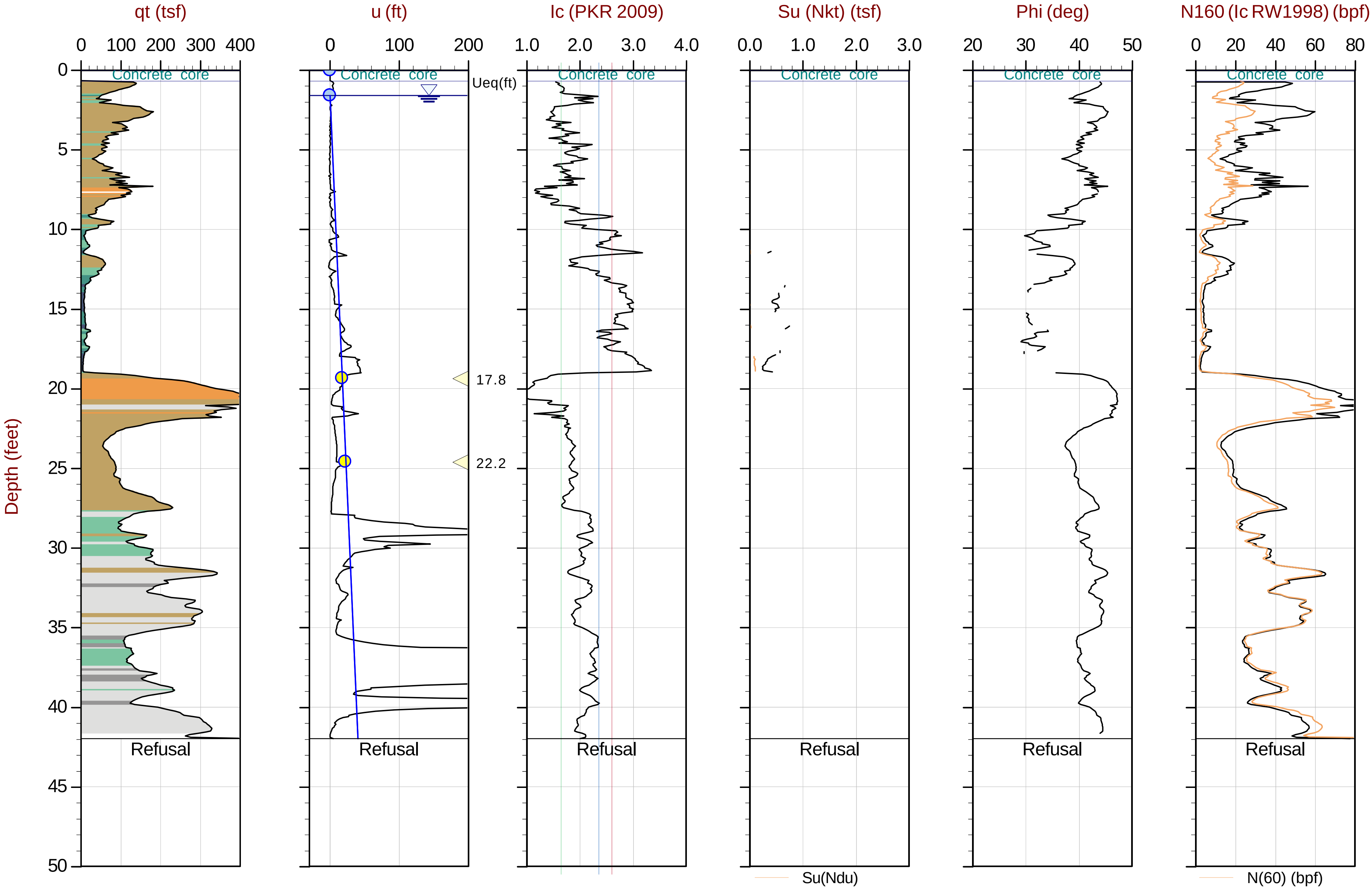
CONETEC

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-27 14:18
Site: ETS New Improvements CPT

Sounding: CPT-01
Cone: 907:T1500F15U35



Max Depth: 12.800 m / 41.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 8.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58799 Long: -122.32093

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

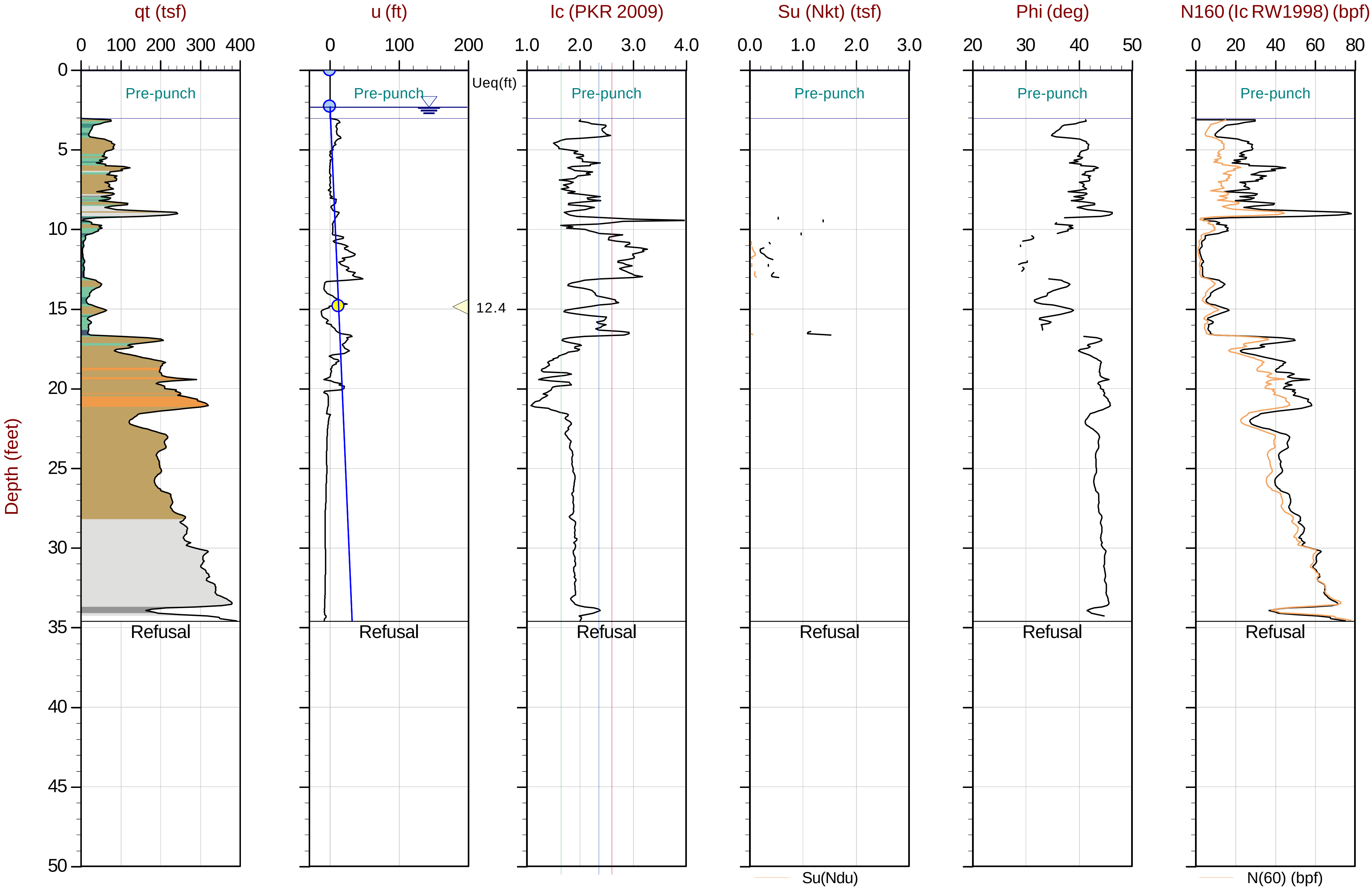
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-05-04 14:28
Site: ETS New Improvements CPT

Sounding: CPT-02B
Cone: 907:T1500F15U35



Max Depth: 10.550 m / 34.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_CP02B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 8.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58762 Long: -122.32078

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

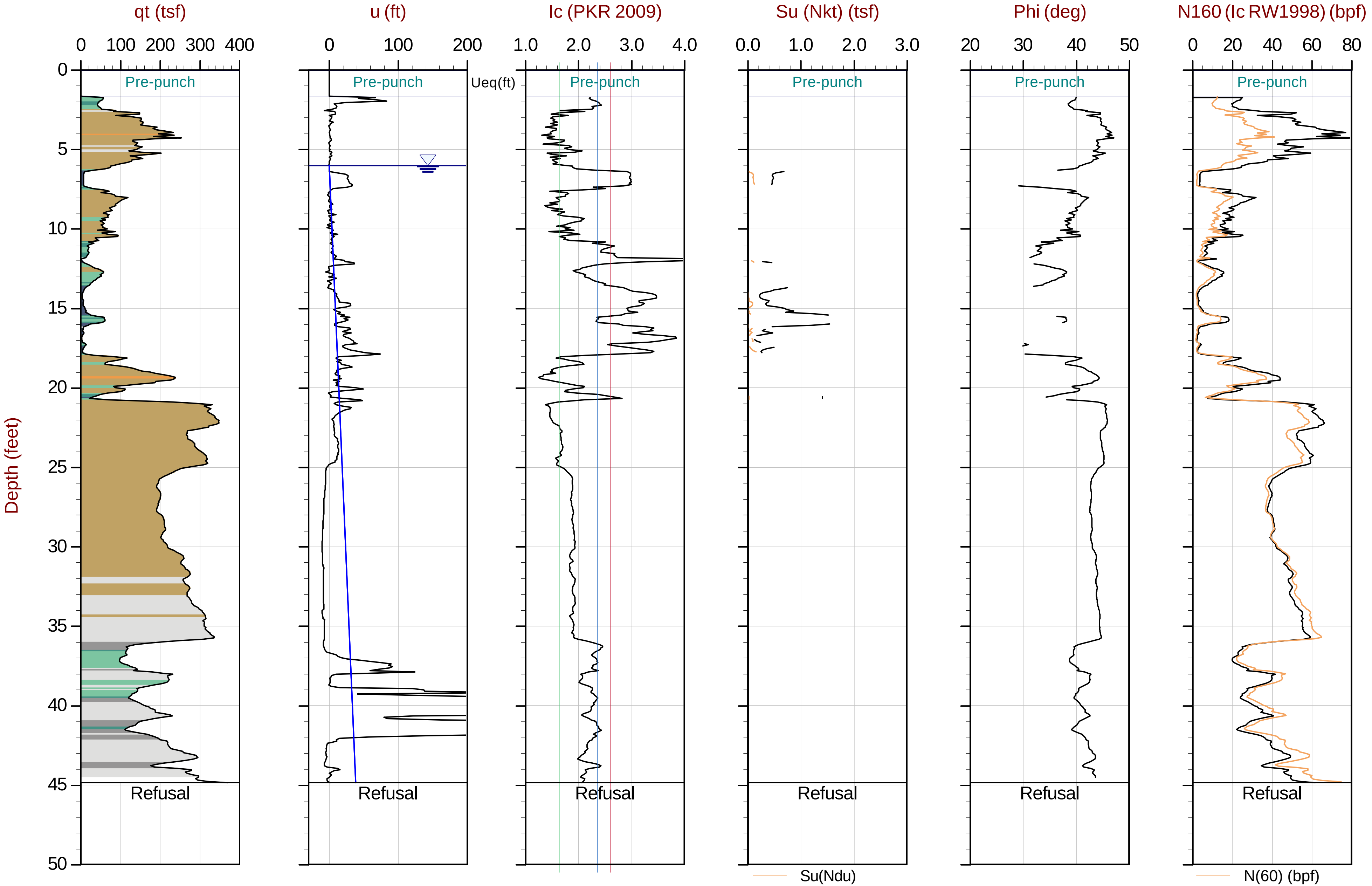
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-28 10:39
Site: ETS New Improvements CPT

Sounding: CPT-03
Cone: 907:T1500F15U35



Max Depth: 13.675 m / 44.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 8.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58728 Long: -122.32078

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

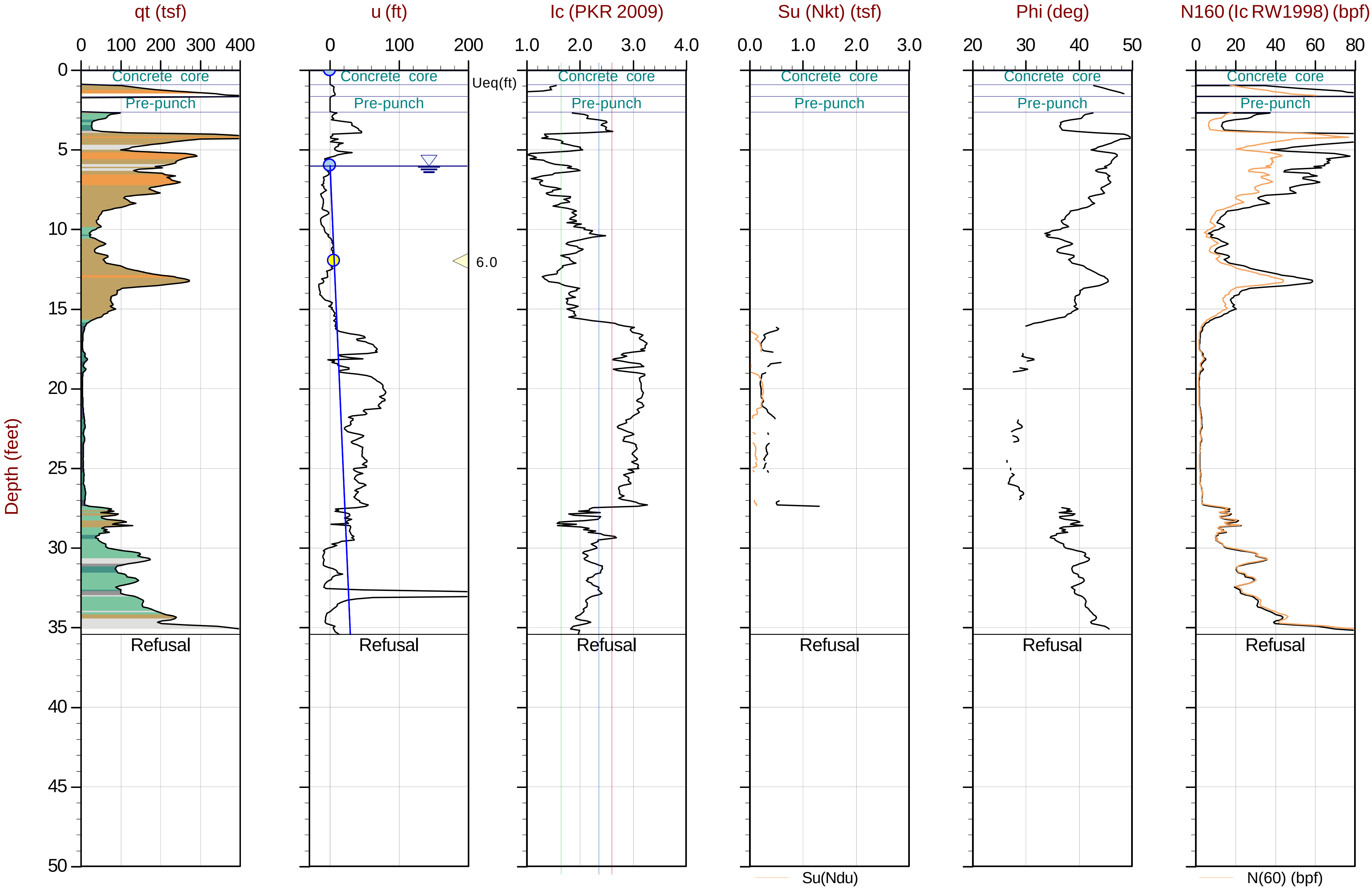
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-28 08:42
Site: ETS New Improvements CPT

Sounding: CPT-04
Cone: 907:T1500F15U35



Max Depth: 10.800 m / 35.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 8.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58760 Long: -122.32111

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

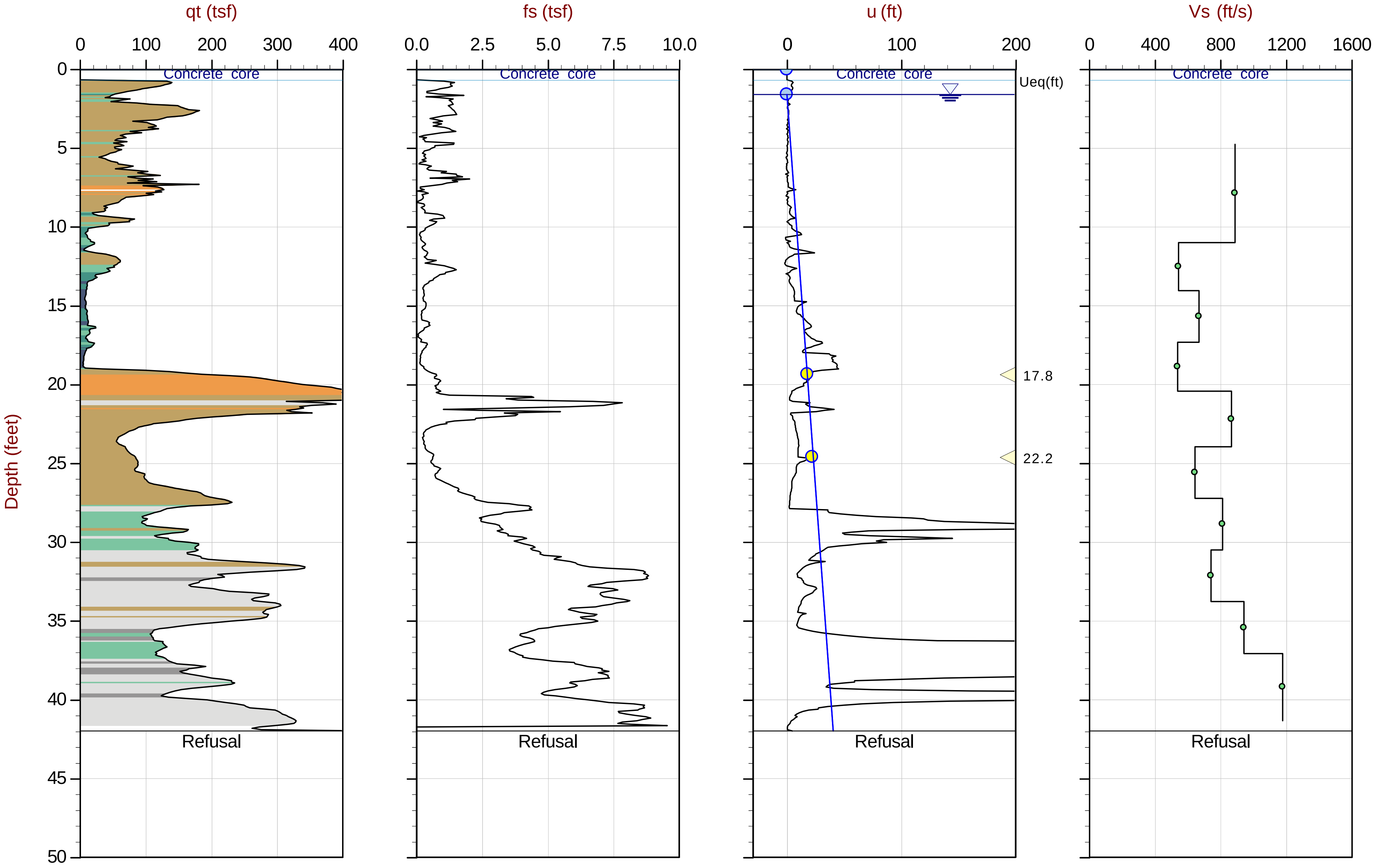
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-27 14:18
Site: ETS New Improvements CPT

Sounding: CPT-01
Cone: 907:T1500F15U35



Max Depth: 12.800 m / 41.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58799 Long: -122.32093

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

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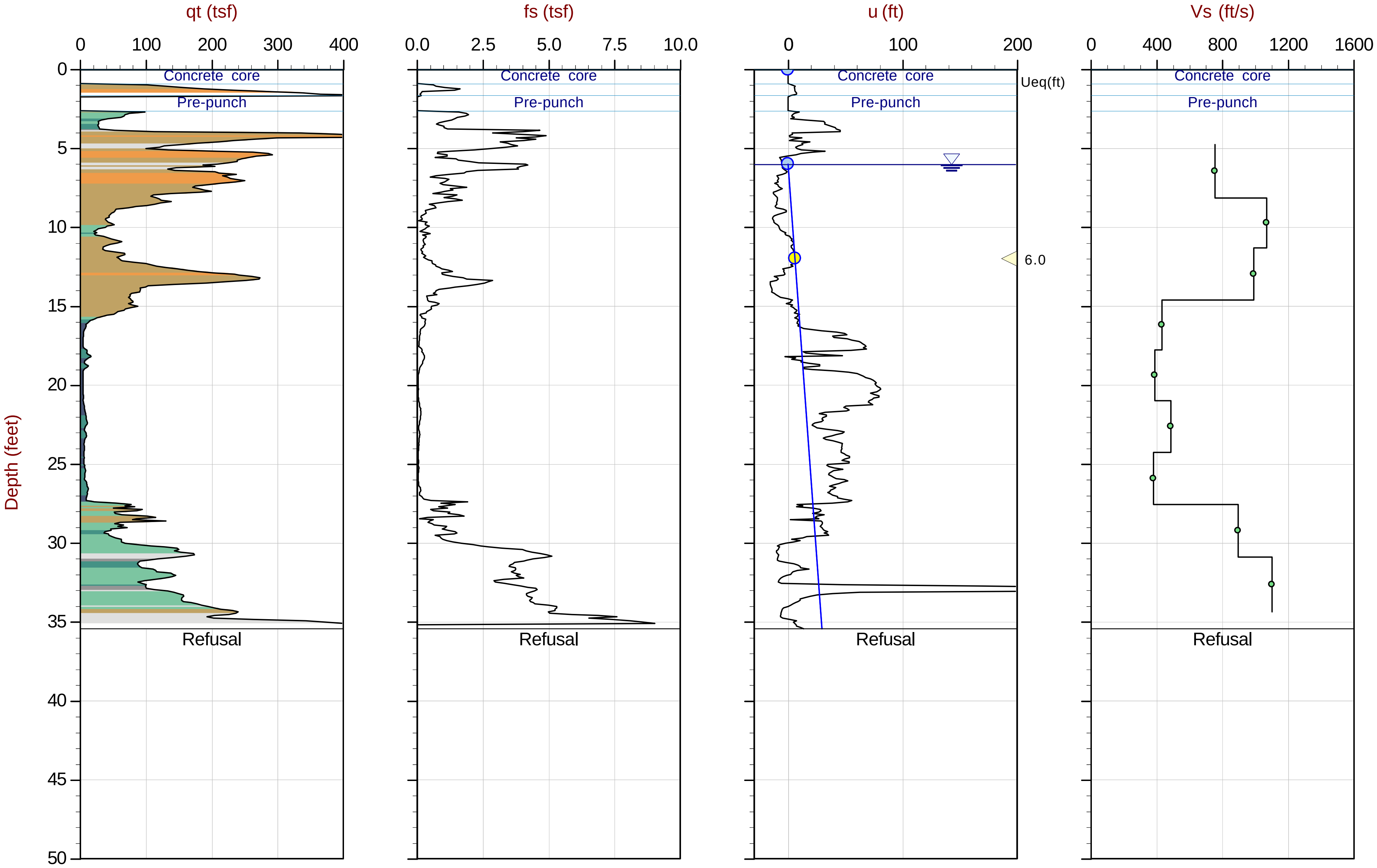
CONETEC

GeoEngineers

Job No: 23-59-25575
Date: 2023-04-28 08:42
Site: ETS New Improvements CPT

Sounding: CPT-04
Cone: 907:T1500F15U35



Max Depth: 10.800 m / 35.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25575_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58760 Long: -122.32111

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

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

Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results

Job No: 23-59-25575
Client: GeoEngineers, Inc.
Project: ETS New Improvements CPT
Sounding ID: CPT-01
Date: 27-Apr-2023

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)
5.41	4.76	5.06			
11.65	10.99	11.13	6.06	6.82	889
14.70	14.04	14.15	3.02	5.55	545
17.98	17.32	17.41	3.26	4.88	669
21.10	20.44	20.51	3.10	5.77	538
24.61	23.95	24.01	3.50	4.04	867
27.89	27.23	27.29	3.27	5.08	644
31.17	30.51	30.56	3.28	4.03	814
34.45	33.79	33.84	3.28	4.43	740
37.73	37.07	37.11	3.28	3.48	942
42.00	41.34	41.38	4.26	3.62	1178

Job No: 23-59-25575
Client: GeoEngineers, Inc.
Project: ETS New Improvements CPT
Sounding ID: CPT-04
Date: 28-Apr-2023

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)
5.41	4.76	5.06			
8.79	8.14	8.32	3.26	4.31	756
11.98	11.32	11.45	3.13	2.93	1071
15.26	14.60	14.70	3.25	3.28	992
18.44	17.78	17.87	3.16	7.32	432
21.65	21.00	21.07	3.20	8.24	389
24.93	24.28	24.34	3.27	6.71	488
28.22	27.56	27.61	3.27	8.58	382
31.56	30.91	30.95	3.34	3.72	898
35.04	34.38	34.43	3.47	3.15	1102

Seismic Cone Penetration Test Shear Wave (Vs) Traces

CONETEC

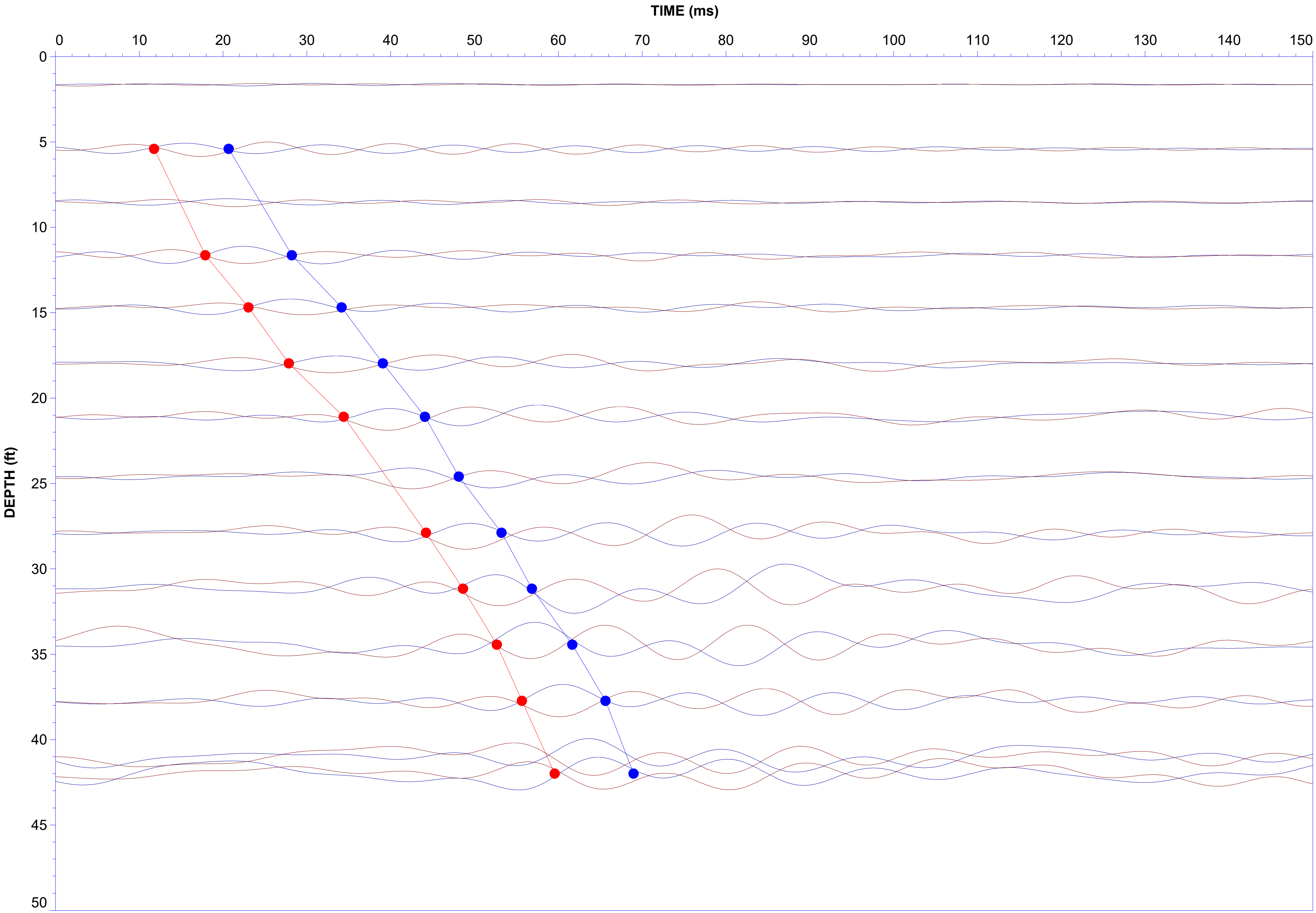
Job No: 23-59-25575
Filter: BP 10 - 80 Hz

Client: GeoEngineers, Inc.
Date: 27-Apr-2023

Project: ETS New Improvements CPT
Cone: 907:T1500F15U35

Analysis: Shear Wave

Sounding: CPT-01



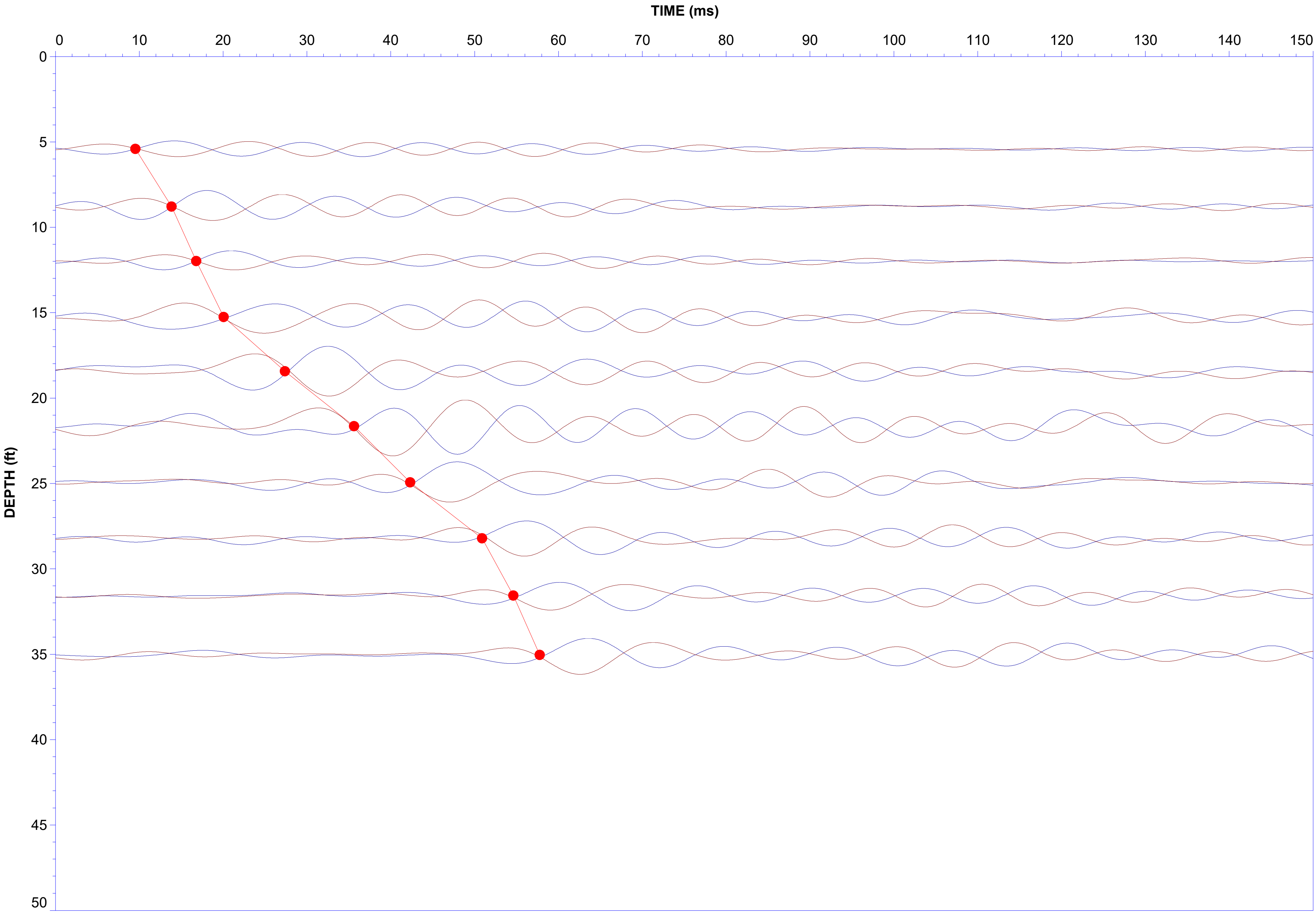
Job No: 23-59-25575
Filter: BP 10 - 90 Hz

Client: GeoEngineers, Inc.
Date: 28-Apr-2023

Project: ETS New Improvements CPT
Cone: 907:T1500F15U35

Analysis: Shear Wave

Sounding: CPT-04



Soil Behavior Type (SBT) Scatter Plots

CONETEC

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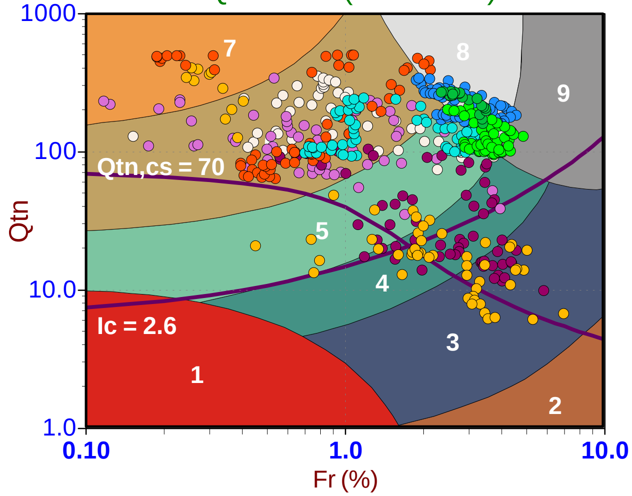
GeoEngineers

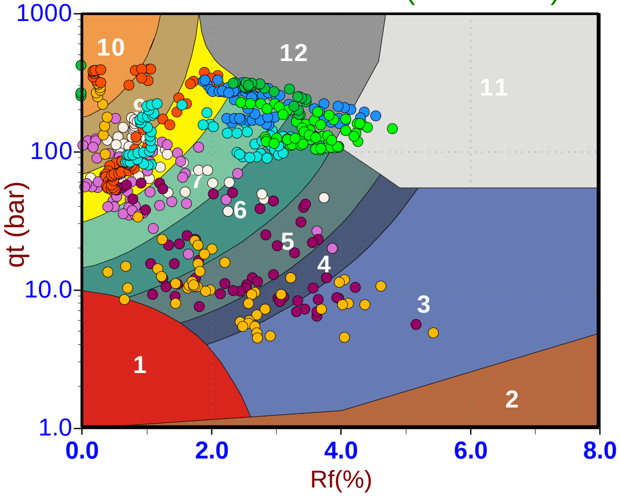
Job No: 23-59-25575
Date: 2023-04-27 14:18
Site: ETS New Improvements CPT

Sounding: CPT-01
Cone: 907:T1500F15U35

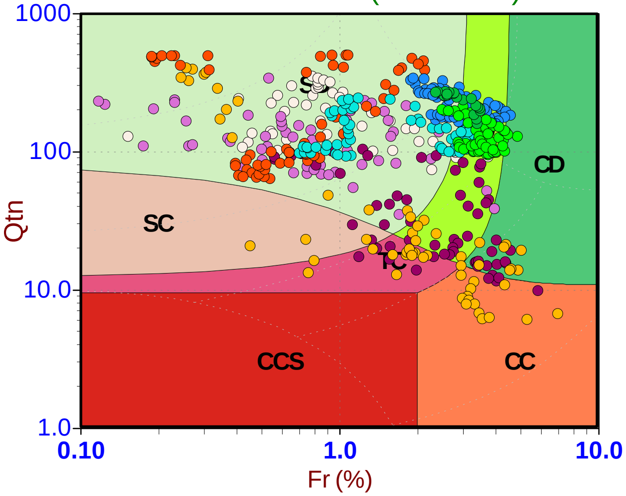
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

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

Legend

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

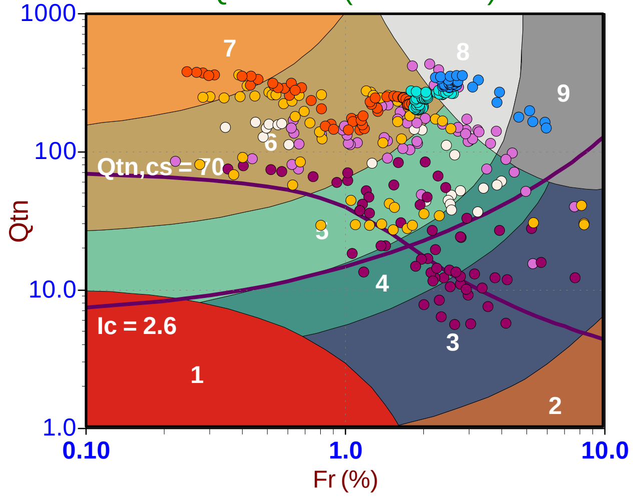
Legend

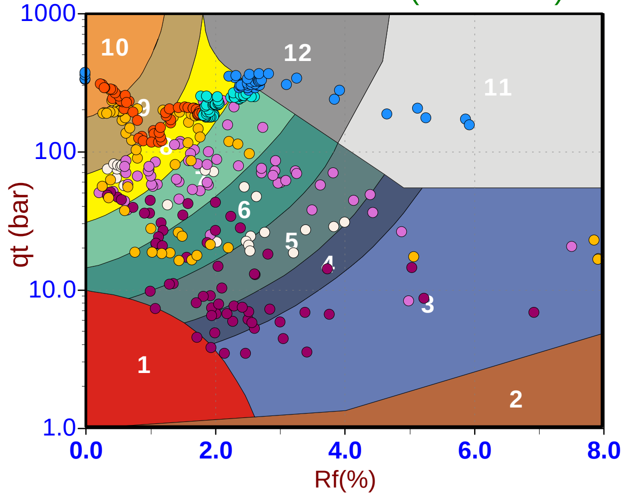
- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

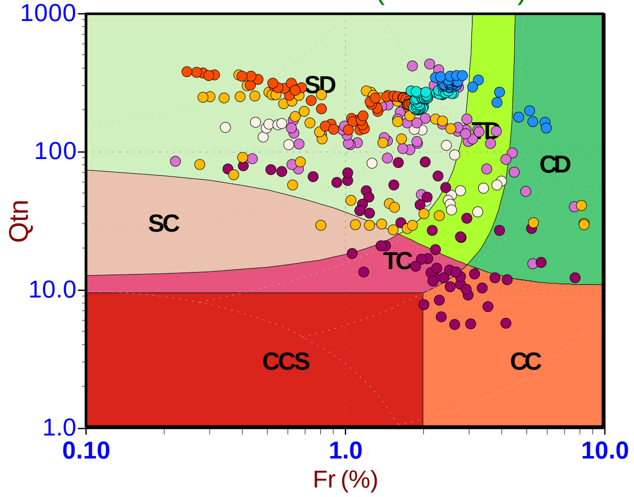
GeoEngineers

Job No: 23-59-25575
Date: 2023-05-04 14:28
Site: ETS New Improvements CPT

Sounding: CPT-02B
Cone: 907:T1500F15U35


Qtn Chart (PKR 2009)


Standard SBT Chart (UBC 1986)


Modified SBTn (PKR 2016)

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

Legend
- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
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- ■ Sand
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- ■ Stiff Fine Grained
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- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

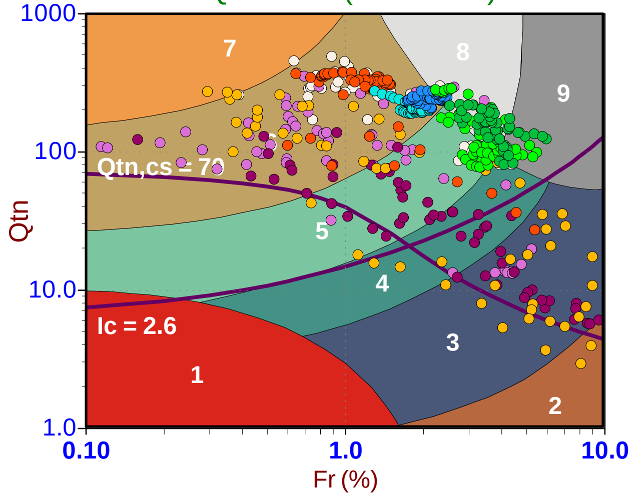
CONETEC

GeoEngineers

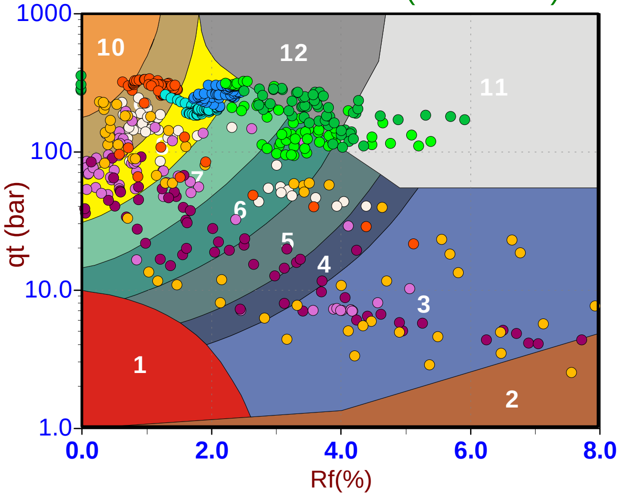
Job No: 23-59-25575
Date: 2023-04-28 10:39
Site: ETS New Improvements CPT

Sounding: CPT-03
Cone: 907:T1500F15U35

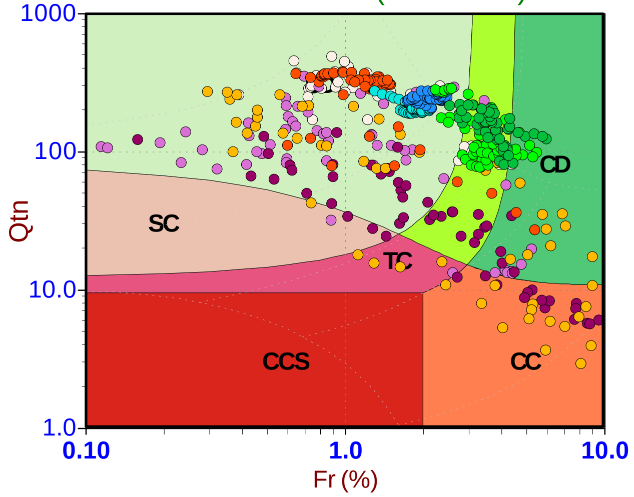
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

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- ■ Silty Sand/Sand
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Legend

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- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

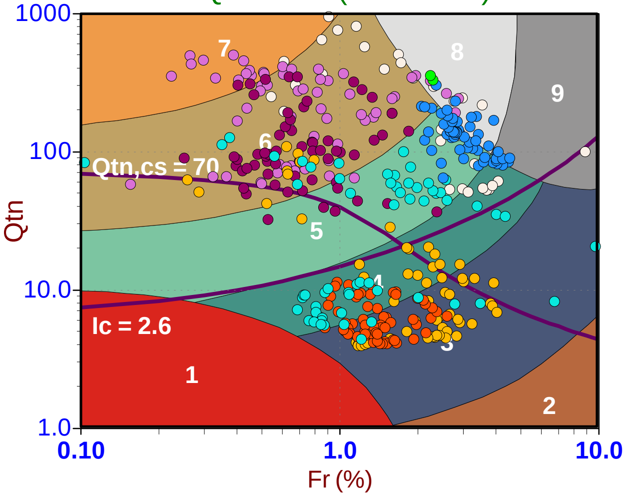
CONETEC

GeoEngineers

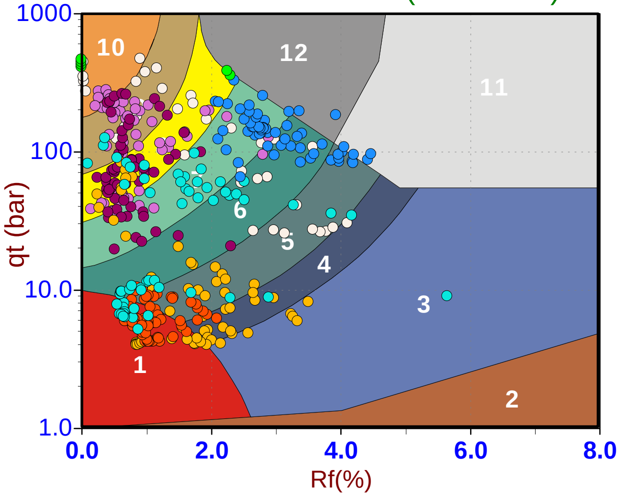
Job No: 23-59-25575
Date: 2023-04-28 08:42
Site: ETS New Improvements CPT

Sounding: CPT-04
Cone: 907:T1500F15U35

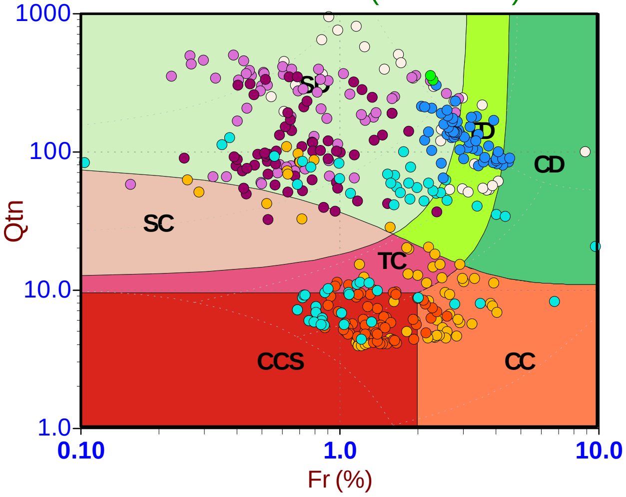
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

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- ■ Sand Mixtures
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- ■ Stiff Sand to Clayey Sand
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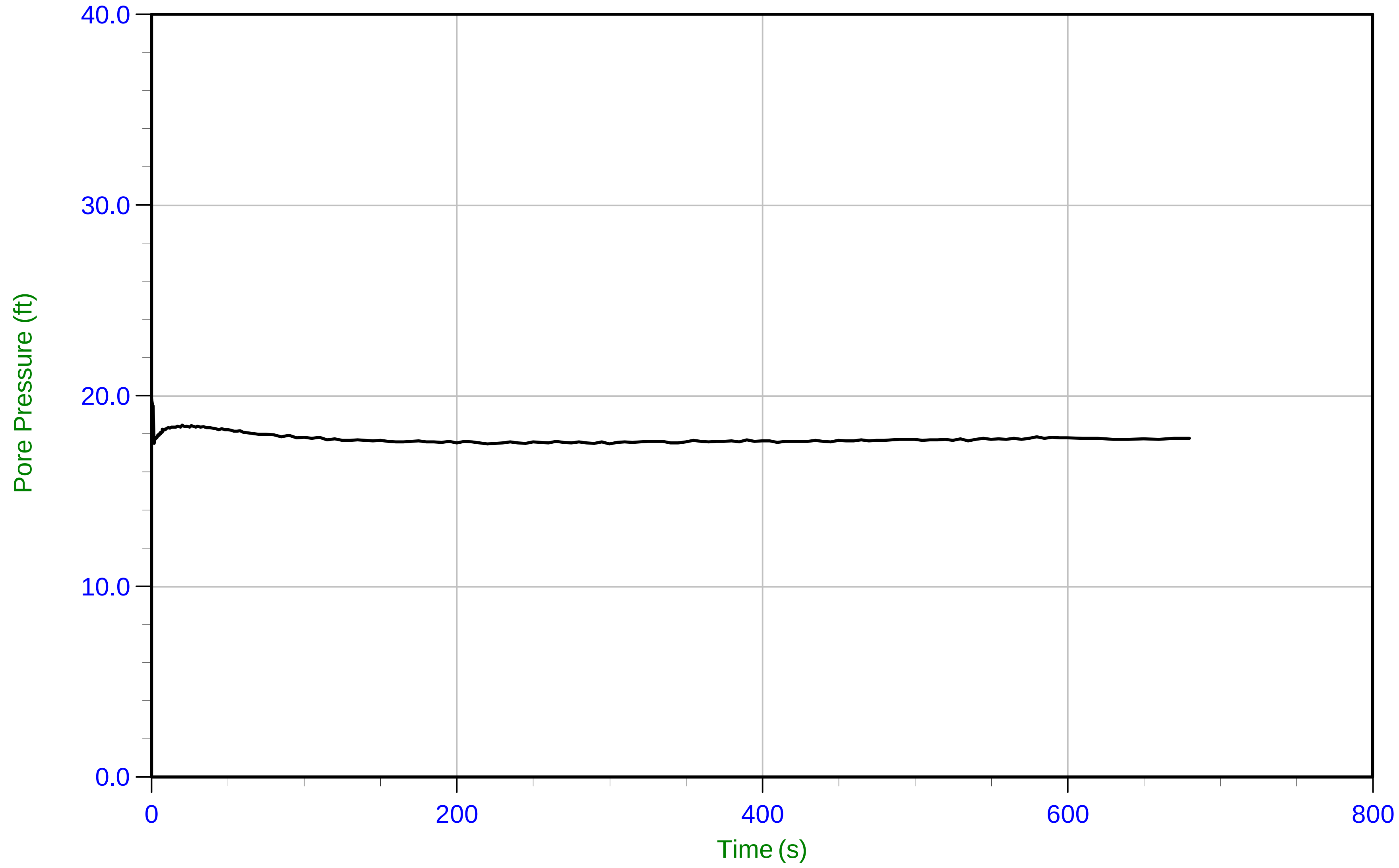
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 23-59-25575
Client: GeoEngineers, Inc.
Project: ETS New Improvements CPT
Start Date: 27-Apr-2023
End Date: 04-May-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-01	23-59-25575_SP01	15	680	19.4	17.8	1.6
CPT-01	23-59-25575_SP01	15	495	24.6	22.2	2.4
CPT-02B	23-59-25575_CP02B	15	305	14.8	12.5	2.3
CPT-04	23-59-25575_SP04	15	305	12.0	6.0	6.0
Total Duration			29.8 min			



Trace Summary:

Filename: 23-59-25575_SP01.ppd2
Depth: 5.900 m / 19.357 ft
Duration: 680.0 s

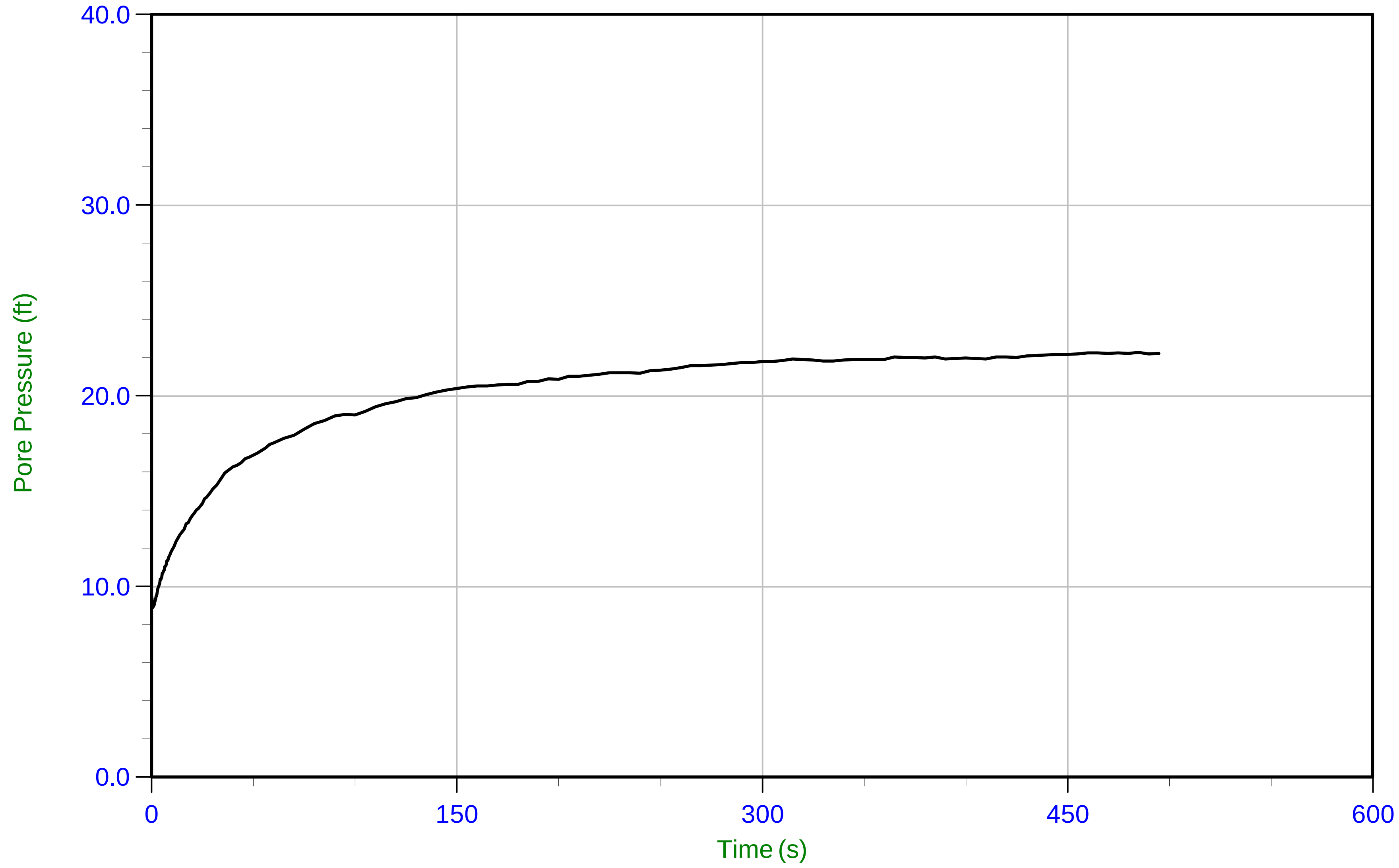
u Min: 17.5 ft
u Max: 19.9 ft
u Final: 17.8 ft

WT: 0.484 m / 1.589 ft
Ueq: 17.8 ft

GeoEngineers

Job No: 23-59-25575
Date: 04/27/2023 14:18
Site: ETS New Improvements CPT

Sounding: CPT-01
Cone: 907:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-25575_SP01.ppd2	u Min: 8.9 ft	WT: 0.724 m / 2.375 ft
Depth: 7.500 m / 24.606 ft	u Max: 22.3 ft	Ueq: 22.2 ft
Duration: 495.0 s	u Final: 22.2 ft	

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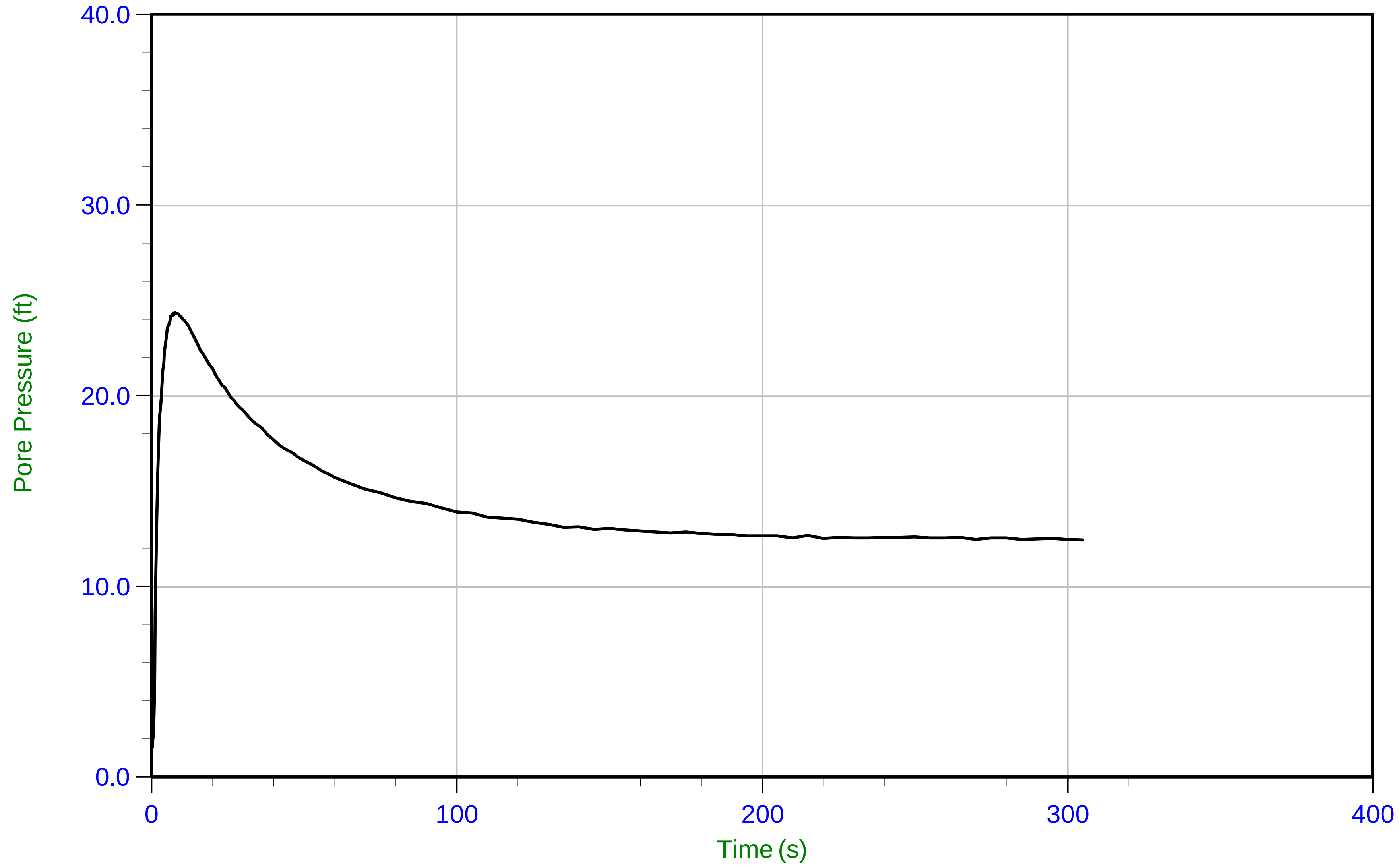
GeoEngineers

Job No: 23-59-25575
Date: 05/04/2023 14:28
Site: ETS New Improvements CPT

Sounding: CPT-02B
Cone: 907:T1500F15U35 Area=15 cm²

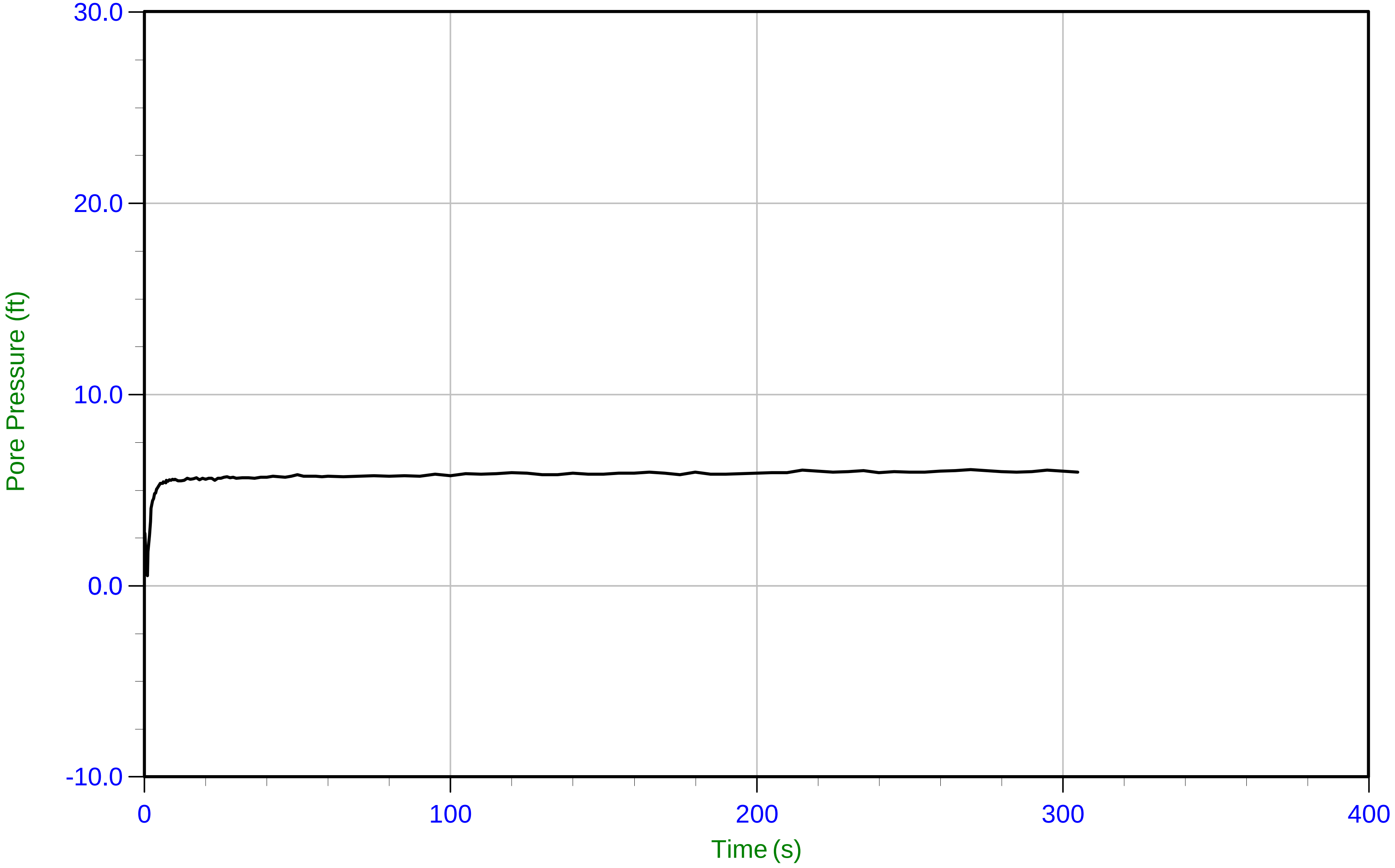


Trace Summary:

Filename: 23-59-25575_CP02B.ppd2
Depth: 4.525 m / 14.846 ft
Duration: 305.0 s

u Min: 1.5 ft
u Max: 24.4 ft
u Final: 12.4 ft

WT: 0.705 m / 2.314 ft
Ueq: 12.5 ft



Trace Summary:

Filename: 23-59-25575_SP04.ppd2
Depth: 3.650 m / 11.975 ft
Duration: 305.0 s

u Min: 0.5 ft
u Max: 6.1 ft
u Final: 5.9 ft

WT: 1.832 m / 6.009 ft
Ueq: 6.0 ft

APPENDIX B
Boring Logs from Previous Studies

APPENDIX B
BORING LOGS FROM PREVIOUS STUDIES

Included in this section are logs from previous studies completed in the immediate vicinity of the project site.

- One boring by Gary Flowers, PLLC in 2005 for the 1762 Airport Way South project.
- Two borings completed by Shannon & Wilson in 2004 for the Metro Atlantic/Central Base Expansion project.
- One boring completed by GeoEngineers, Inc. in 1996 for the 811 South Massachusetts cellular transmission tower project.
- Four borings completed by Aspect Consulting in 2019 for the 1700 Airport Way South Phase II Environmental Site Assessment.

EXPLORATION BORING LOG

Number **EB-2** PAGE 1 OF 2

SEDIMENT DESCRIPTION	DEPTH	SAMPLE	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows/Foot (10 20 30 40)
0-6" Concrete		5 13 25		38
FILL Dense, moist, silty sand with gravel; upper 6" brownish black, middle 6" brown, lower 6" gray		3 4 3		7
becomes medium stiff to soft with depth, saturated, blackish brown to dark brown, sandy silt with gravel	5	2 1 1	▼ ATD	2
ALLUVIUM Loose/soft, saturated, bluish gray to gray, intermixed silt, sand, and gravel with some organics, shells and wood fiber, Old Beach Deposit?		2 2 3		5
Very soft, saturated, gray, sandy silt with gravel	10	2 1 1		2
Interbedded, very soft, saturated, gray sandy silt to silty clay or clayey silt, plastic and very soft, saturated, blackish gray, organic silt with layers of fibrous peat	15	0 0 1		1
	20	0 0 1		1
	25	0 0 1		1
at 29' drilling change to gravels Medium dense, saturated, gray, sandy gravel	30	3 5 6		11

GARY A. FLOWERS, PLLC

RITCHEY PROPERTY
1762 AIRPORT WAY SOUTH
SEATTLE, WASHINGTON
FOR ROBERT M. PRIDE, INC.
JUNE 2005
PROJECT NO. G05050

EXPLORATION BORING LOG

Number **EB-2** PAGE 2 OF 2

SEDIMENT DESCRIPTION	DEPTH	SAMPLE	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows/Foot 10 20 30 40
TRANSITIONAL BEDS Very dense, saturated, gray, interbedded medium to coarse sand with occasional gravel; tip is brown, silty sand		3 14 44		58
Very hard, moist, gray silt with trace sand, tiny striations of ash, small piece of wood	40	29 24 50/5"		100+
BOH @ 41-1/2'				
	45			
	50			
	55			
	60			
	65			

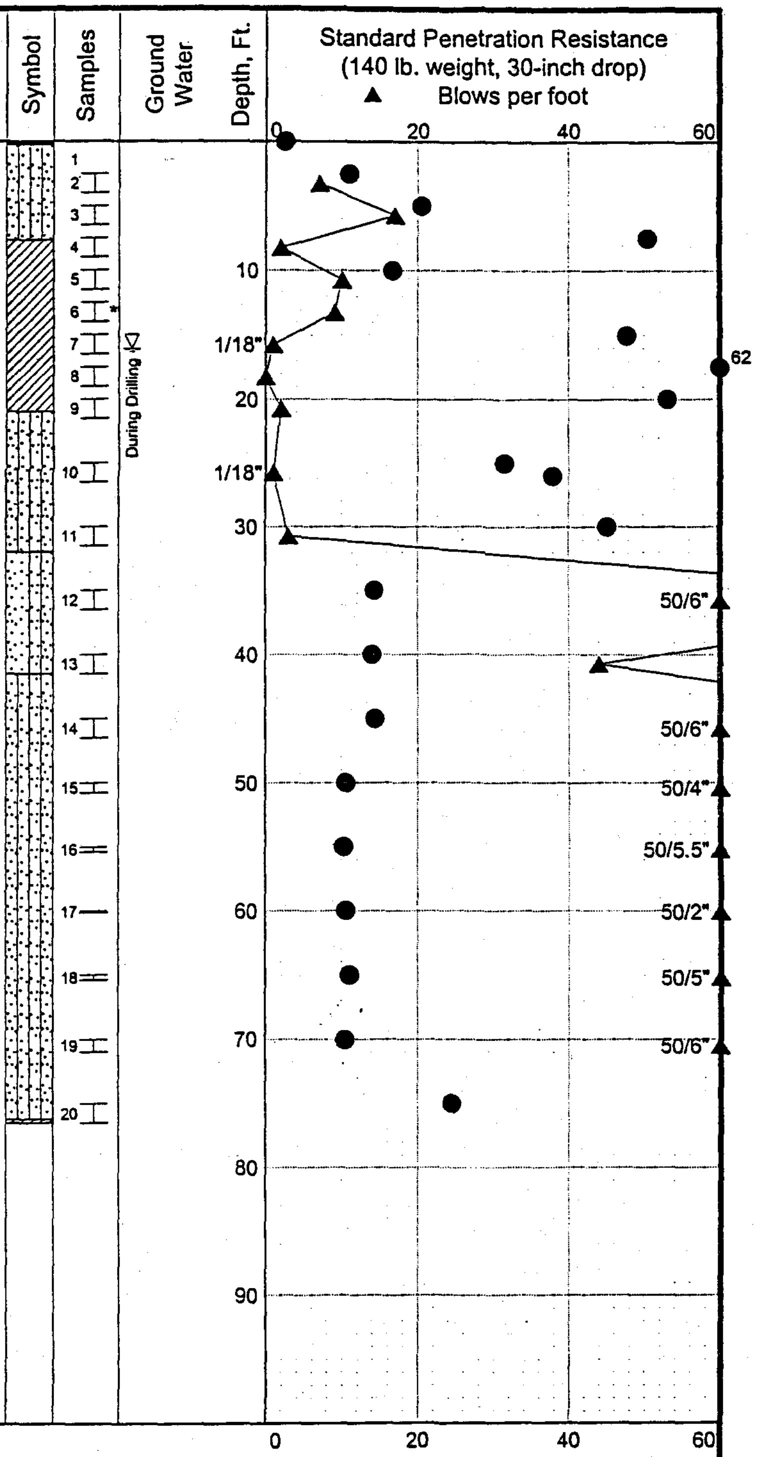
Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by geologic interpretations, engineering analysis, and judgment. They are not necessarily representative of other times and locations. We will not accept responsibility for the use or interpretation by others of information presented on this log.

GARY A. FLOWERS, PLLC

RITCHEY PROPERTY
1762 AIRPORT WAY SOUTH
SEATTLE, WASHINGTON
FOR ROBERT M. PRIDE, INC.
JUNE 2005
PROJECT NO. G05050

SOIL DESCRIPTION Surface Elevation: Approx. 24.0 Ft.	Depth, Ft.
Asphalt.	0.3
Loose to medium dense, brown-gray, silty, gravelly SAND; moist; (Fill) SM.	7.7
Very soft, gray, silty CLAY; wet; massive, with layers of clayey, silty fine sand; CL/SC.	21.0
Very soft to very loose, silty, fine SAND to fine sandy SILT; wet; SM/ML.	32.0
Dense to very dense, slightly silty, gravelly SAND; wet; SP-SM.	41.5
Very dense, gray, silty fine gravelly SAND; moist; cohesive; (Till) SM.	76.2
Stiff, dark gray, silty CLAY, trace of fine sand and gravel; moist; slightly blocky; (Disturbed Sample) CL.	76.5

BOTTOM OF BORING
COMPLETED 6/11/2003



Log: WRL Rev: MWP Typ: EET

LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

⊻ Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

MASTER LOG2 21-09886.GPJ TEMP.GDT 2/13/04

NOTES

1. The boring was performed using drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

Metro Atlantic/Central Base Expansion
Seattle, Washington

LOG OF BORING EB-1

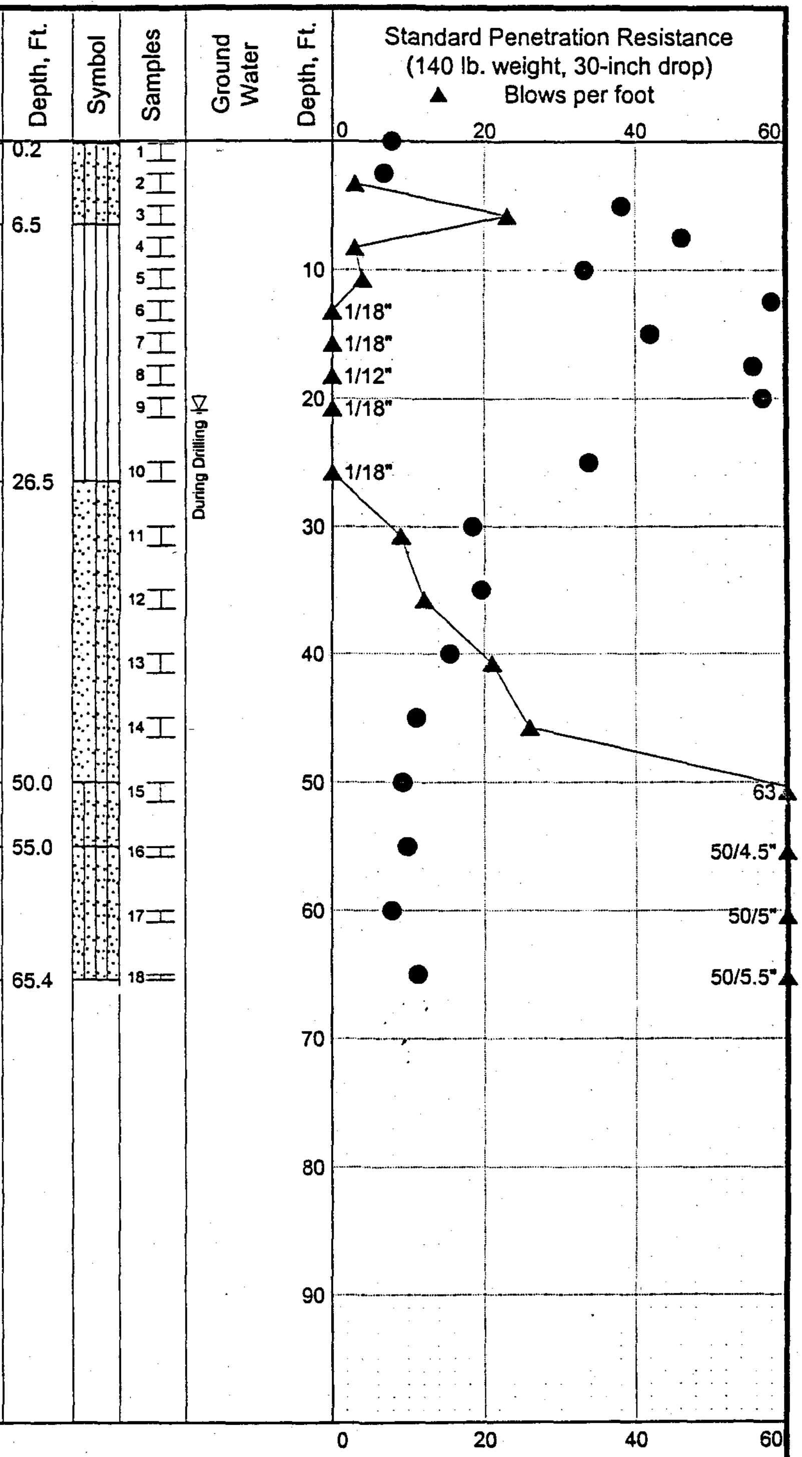
February 2004 21-1-09886-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

SOIL DESCRIPTION	Depth, Ft.
Surface Elevation: Approx. 24.0 Ft.	
Asphalt.	0.2
Loose to medium dense, gray-brown, silty SAND; moist; gravelly at top and scattered brick debris at bottom; (Fill) SM.	6.5
Very soft, gray, clayey SILT/silty CLAY; wet; with interbedded, very loose to loose, silty SAND pockets; ML/CL/SM. - scattered shell fragments from 25.0 to 26.5 feet deep	26.5
Loose to medium dense, gray to light brown, trace to slightly gravelly, slightly silty to silty SAND; wet; SP-SM/SM.	50.0
Very dense, gray, silty, gravelly SAND; moist; cohesive; (Weathered Till) SM.	55.0
Very dense, gray, slightly silty to silty, fine gravelly SAND; moist; cohesive; (Till) SM.	65.4

BOTTOM OF BORING
COMPLETED 6/10/2003



LEGEND

- * Sample Not Recovered
- ⌶ Standard Penetration Test
- ⊻ Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

Log: WRL Rev: MWP Typ: LKD

NOTES

1. The boring was performed using drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

MASTER LOG2 21-09886.GPJ TEMP.GDT 2/13/04

Metro Atlantic/Central Base Expansion
Seattle, Washington

LOG OF BORING EB-2

February 2004 21-1-09886-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

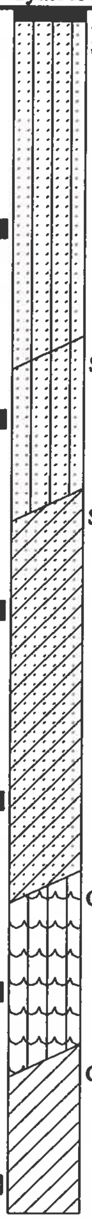
FIG. A-3

TEST DATA

BORING SEA-018

DESCRIPTION

Surface Elevation (ft.):



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
					ASPHALT	3 inches asphalt
					SM	Gray silty fine to medium sand with occasional gravel (very loose, moist) (fill)
MD CS	20 19	95 110	2	■		
					SM	Gray silty fine sand with organic matter (loose, moist)
MD	32	84	4	■		
					SC	Gray clayey sand (loose, moist)
MD	43	92	1	■		
			1	■		
					OL	Brown organic silt (very soft, moist)
MD	41	78	1/18"	■		
					CL	Gray silty clay with a trace of fine sand (soft, moist)
			4	□		

Boring completed at 31.5 feet on 10/02/96
Ground water encountered at a depth of 13.0 feet

DEPTH IN FEET

0 5 10 15 20 25 30 35 40

Note: See Figure 2 for explanation of symbols

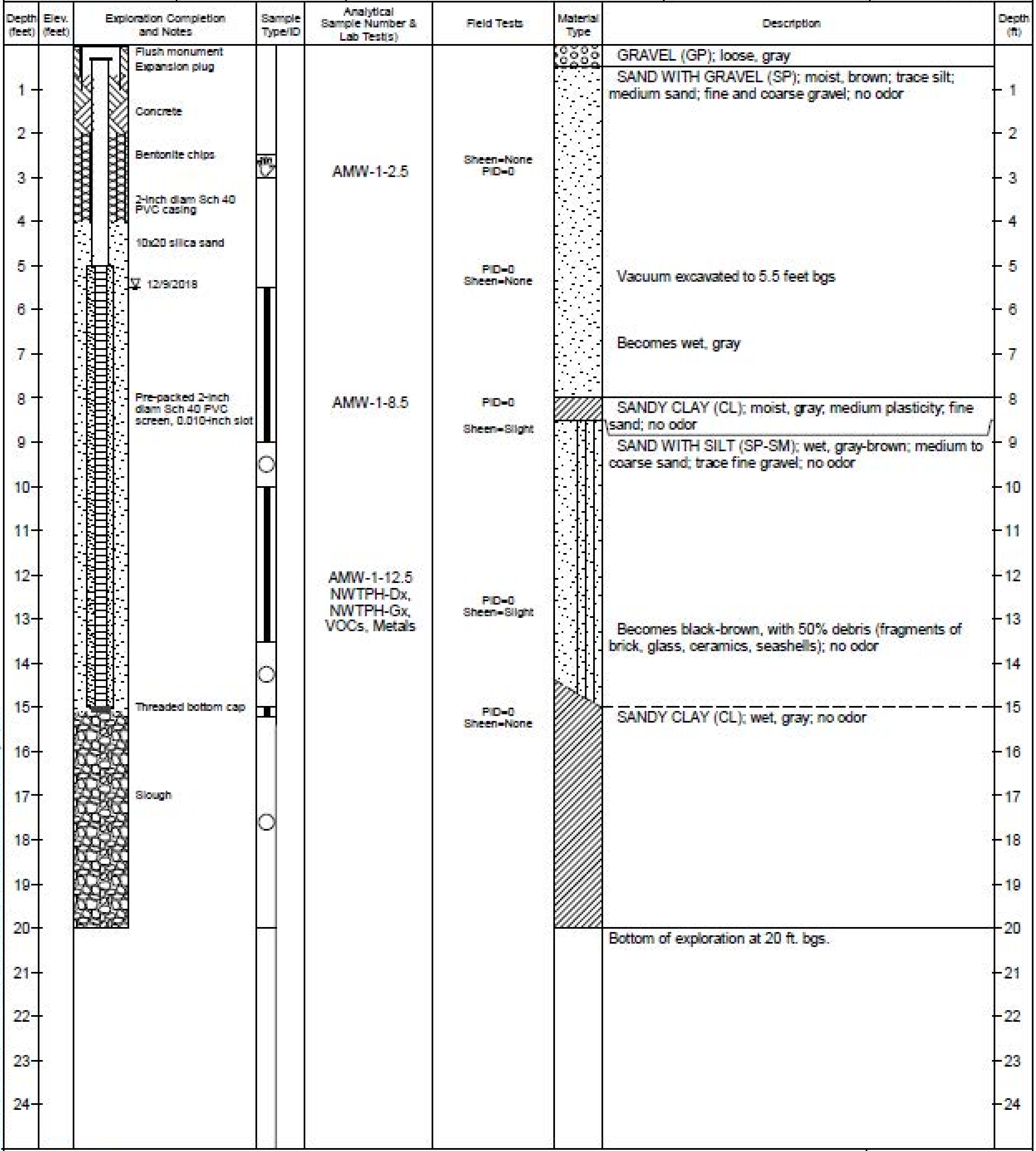
5408-001-01-1130 T2 :SJT:JJM:JKT:CMS 10/28/96

GeoEngineers

LOG OF BORING

FIGURE 4

Aspect Consulting	Evergreen Treatment Services - 180043			Environmental Exploration Log	
	Project Address & Site Specific Location 1700 Airport Way S, Seattle, WA, Parking strip			*Coordinates (Lat,Lon WGS84)* 47.58778, -122.32123 (est)	*Exploration Number* **AMW-1**
Contractor Cascade Drilling	*Equipment* Geoprobe 7822 DT	*Sampling Method* Percussion hammer		*Ground Surface (GS) Elev.*	Ecology Well Tag No. BKF905
Operator Tim	*Exploration Method(s)* Direct push	*Work Start/Completion Dates* 12/9/2018		*Top of Casing Elev. (NAVD88)* NA (est)	*Depth to Water (Below GS)* 5.5' (ATD)



Depth (feet)	Exploration Completion and Notes	Analytical Sample Number & Lab Test(s)	Field Tests	Description
0	Flush monument Expansion plug			GRAVEL (GP); loose, gray
0.5				SAND WITH GRAVEL (SP); moist, brown; trace silt; medium sand; fine and coarse gravel; no odor
1.5	Concrete			
2.5	Bentonite chips	AMW-1-2.5	Sheen=None PID=0	
3.5	2-inch diam Sch 40 PVC casing			
4.5	10x20 silica sand			
5.5	12/9/2018		PID=0 Sheen=None	Vacuum excavated to 5.5 feet bgs
7				Becomes wet, gray
8	Pre-packed 2-inch diam Sch 40 PVC screen, 0.010-inch slot	AMW-1-8.5	PID=0 Sheen=Slight	SANDY CLAY (CL); moist, gray; medium plasticity; fine sand; no odor
9				SAND WITH SILT (SP-SM); wet, gray-brown; medium to coarse sand; trace fine gravel; no odor
12.5		AMW-1-12.5 NWTPH-Dx, NWTPH-Gx, VOCs, Metals	PID=0 Sheen=Slight	Becomes black-brown, with 50% debris (fragments of brick, glass, ceramics, seashells); no odor
15	Threaded bottom cap		PID=0 Sheen=None	SANDY CLAY (CL); wet, gray; no odor
17	Slough			
20				Bottom of exploration at 20 ft. bgs.

Legend

Sample Type
- ☐ No Soil Sample Recovery
- ▨ Grab sample
- ▯ Continuous core 1.85" ID

Water Level
- ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

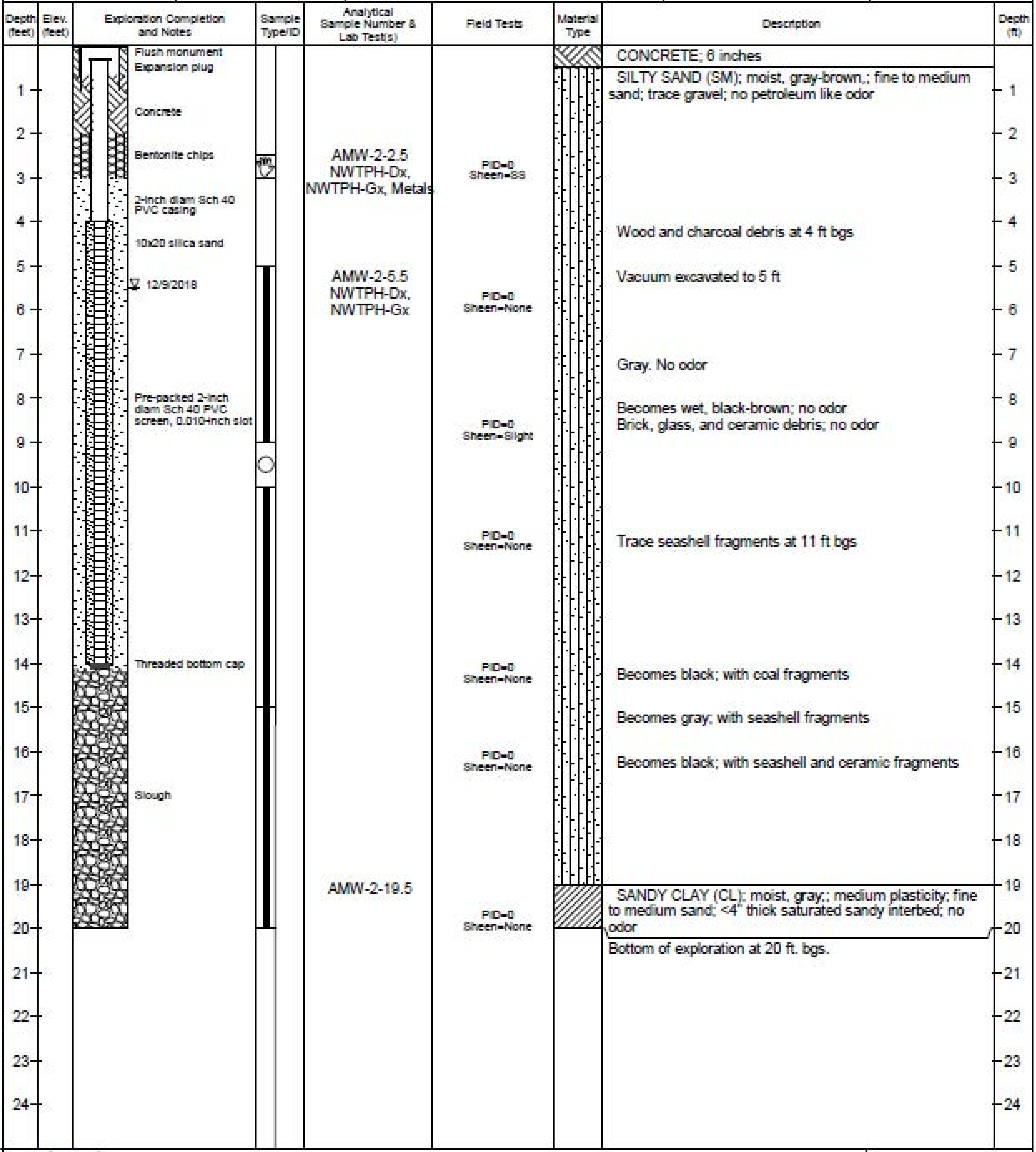
Logged by: KB
Approved by: DC, 1/15/2019

Exploration Log
AMW-1
Sheet 1 of 1

Evergreen Treatment Services - 180043

Environmental Exploration Log

Project Address & Site Specific Location	1700 Airport Way S, Seattle, WA, Between buildings	*Coordinates (Lat,Lon WGS84)*	47.58756, -122.32112 (est)	*Exploration Number* AMW-2
Contractor	Cascade Drilling	*Equipment*	Geoprobe 7822 DT	*Sampling Method* Percussion hammer
Ground Surface (GS) Elev.		*Ecology Well Tag No.*	BKF906	
Operator	Tim	*Exploration Method(s)*	Direct push	*Work Start/Completion Dates* 12/9/2018
Top of Casing Elev. (NAVD88)	NA (est)	*Depth to Water (Below GS)*	5.5' (ATD)	



Exploration Completion and Notes	Analytical Sample Number & Lab Test(s)	Field Tests	Description
Flush monument Expansion plug			CONCRETE; 6 inches
			SILTY SAND (SM); moist, gray-brown,; fine to medium sand; trace gravel; no petroleum like odor
Concrete			
Bentonite chips	AMW-2-2.5 NWTPH-Dx, NWTPH-Gx, Metals	PID=0 Sheen=SS	
2-inch diam Sch 40 PVC casing			
10x20 silica sand			Wood and charcoal debris at 4 ft bgs
	AMW-2-5.5 NWTPH-Dx, NWTPH-Gx		Vacuum excavated to 5 ft
∇ 12/9/2018		PID=0 Sheen=None	
			Gray. No odor
Pre-packed 2-inch diam Sch 40 PVC screen, 0.010-inch slot			Becomes wet, black-brown; no odor Brick, glass, and ceramic debris; no odor
		PID=0 Sheen=Slight	
		PID=0 Sheen=None	Trace seashell fragments at 11 ft bgs
Threaded bottom cap		PID=0 Sheen=None	Becomes black; with coal fragments
			Becomes gray; with seashell fragments
		PID=0 Sheen=None	Becomes black; with seashell and ceramic fragments
Slough			
	AMW-2-19.5		SANDY CLAY (CL); moist, gray,; medium plasticity; fine to medium sand; <4" thick saturated sandy interbed; no odor
		PID=0 Sheen=None	
			Bottom of exploration at 20 ft. bgs.

Legend

Sample Type
- ☒ No Soil Sample Recovery
- ▣ Grab sample
- ◫ Continuous core 1.85" ID

Water Level
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: KB
Approved by: DC, 1/15/2019

Exploration Log AMW-2

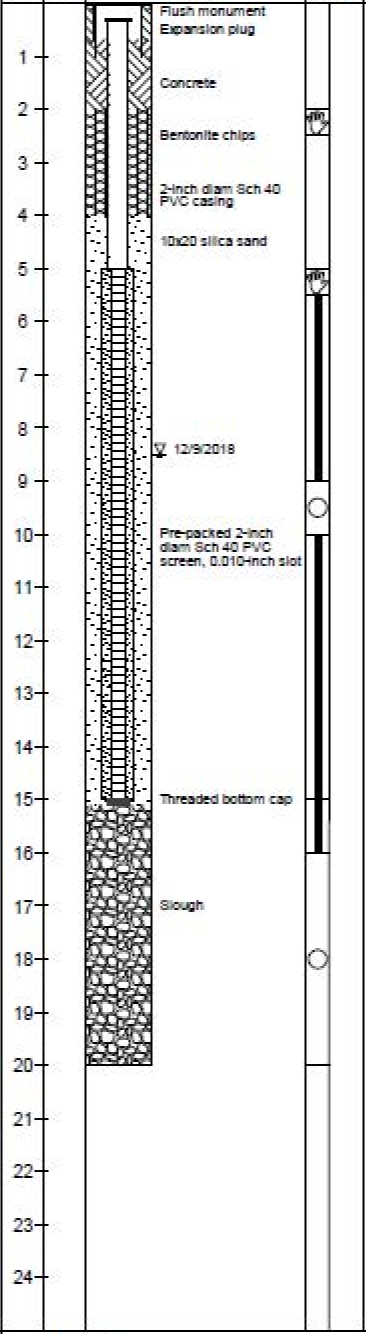
Sheet 1 of 1

NEW STANDARD EXPLORATION LOG TEMPLATE P:\GINT\PROJECTS\EVERGREEN CLINIC 180043.GPJ January 17, 2019

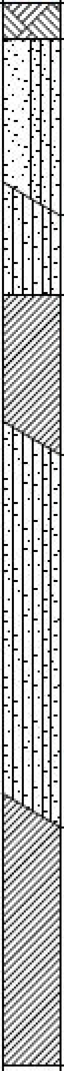
Evergreen Treatment Services - 180043

Environmental Exploration Log

Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
1700 Airport Way S, Seattle, WA, Parking lane			47.58726, -122.32128 (est)	**AMW-3**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface (GS) Elev.*	Ecology Well Tag No. BKF907
Cascade Drilling	Geoprobe 7822 DT	Percussion hammer		
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Tim	Direct push	12/9/2018	NA (est)	8.5' (ATD)



Depth (feet)	Analytical Sample Number & Lab Test(s)	Field Tests
2		Sheen=None PID=0
5	AMW-3-5.0 NWTPH-Dx, NWTPH-Gx, Metals, cPAHs	Sheen=None PID=0
6		PID=0 Sheen=None
8.5		PID=0 Sheen=None
10	AMW-3-10.0	
11		PID=0 Sheen=None
15	AMW-3-15.0	PID=0 Sheen=None
16		PID=0 Sheen=None



Depth (ft)	Description
0	CONCRETE; 8 inches
0.7	SAND WITH SILT AND GRAVEL (SP-SM); moist, gray; medium sand; fine to coarse gravel; no odor
3	Vacuum-excavated to 5 ft
4	SILTY SAND (SM); moist, gray; no odor
5.5	SANDY CLAY (CL); moist, gray; medium to high plasticity; fine sand; interbedded with 1' beds SAND WITH GRAVEL (SW); wet, gray; no odor
8.5	SILTY SAND (SM); wet, gray; medium to coarse sand; trace fine gravel; no odor
14	Becomes fine sand
15.5	CLAY (CL); wet, gray; medium to high plasticity; no odor
20	Bottom of exploration at 20 ft. bgs.

Legend

Sample Type
- ☐ No Soil Sample Recovery
- ▨ Grab sample
- ◫ Continuous core 1.85" ID

Water Level
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: KB
Approved by: DC, 1/15/2019

Exploration Log AMW-3

Sheet 1 of 1

ASPECT CONSULTING

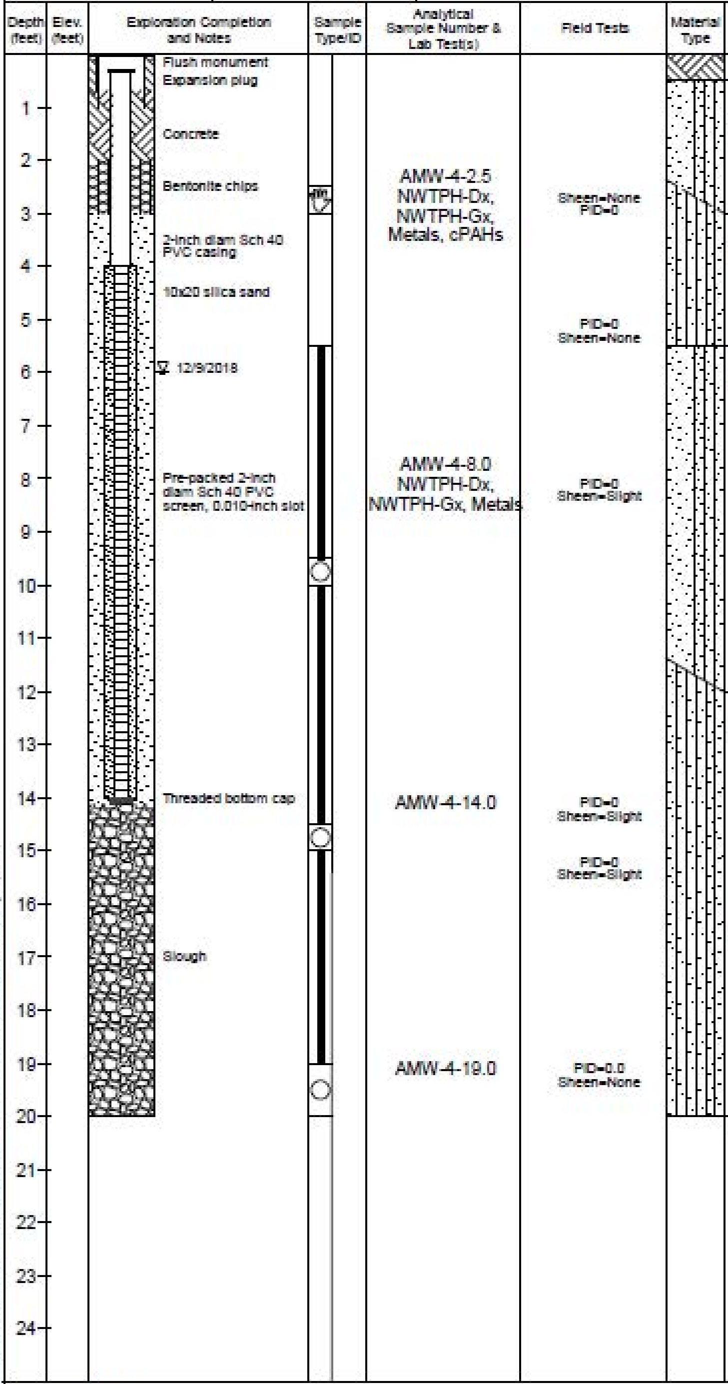
Evergreen Treatment Services - 180043

Environmental Exploration Log

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
1700 Airport Way S, Seattle, WA, Back parking lot	47.58755, -122.32088 (est)	**AMW-4**

Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev.	Ecology Well Tag No.
Cascade Drilling	Geoprobe 7822 DT	Percussion hammer		BKF908

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Tim	Direct push	12/9/2018 to 12/10/2018	NA (est)	6' (ATD)



Depth (feet)	Exploration Completion and Notes	Analytical Sample Number & Lab Test(s)	Field Tests	Description
0	Flush monument Expansion plug			CONCRETE; 6 inches
0.5				SAND WITH SILT AND GRAVEL (SP-SM); moist, brown; medium to coarse sand; fine to coarse gravel; no odor
1.5	Concrete			
2.5	Bentonite chips	AMW-4-2.5 NWTPH-Dx, NWTPH-Gx, Metals, cPAHs	Sheen=None PID=0	
3				SILTY SAND (SM); moist, brown; medium to coarse sand; no odor
3.5	2-inch diam Sch 40 PVC casing			
4.5	10x20 silica sand			
5			PID=0 Sheen=None	Vacuum excavated to 5 ft.
5.5				SAND WITH SILT (SP-SM); wet, brown; fine to medium sand, trace coarse sand; no odor
6	∇ 12/9/2018			
8	Pre-packed 2-inch diam Sch 40 PVC screen, 0.010-inch slot	AMW-4-8.0 NWTPH-Dx, NWTPH-Gx, Metals	PID=0 Sheen=Slight	
12				SILTY SAND (SM); wet, black; coarse sand; 50% coal fragments, brick fragments, seashell fragments; no odor
14	Threaded bottom cap	AMW-4-14.0	PID=0 Sheen=Slight	
15			PID=0 Sheen=Slight	
16				Becomes gray, fine to coarse sand; trace fine gravel; no odor
17	Slough			
19		AMW-4-19.0	PID=0.0 Sheen=None	
20				Bottom of exploration at 20 ft. bgs.

Legend

Sample Type
- ☒ No Soil Sample Recovery
- ▣ Grab sample
- ◫ Continuous core 1.85" ID

Water Level
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: KB
Approved by: DC, 1/15/2019

Exploration Log AMW-4

Sheet 1 of 1

NEW STANDARD EXPLORATION LOG TEMPLATE P:\GINT\PROJECTS\EVERGREEN CLINIC 180043.GPJ January 17, 2019

APPENDIX C

Shoring Monitoring Program

APPENDIX C
SHORING MONITORING PROGRAM

Shoring Monitoring

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the temporary shoring walls and existing retaining walls on the eastern edge of the site and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a preconstruction survey of adjacent improvements, such as streets, utilities, and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction as presented in the following table.

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized.	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized.	Three times per week
After excavation is complete and wall movements have stabilized, and before the floors of the building reach the top of the excavation.	Twice monthly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning excavation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established along the top of the shoring walls. The survey points should be located on every other shoring soldier pile along the wall face and on adjacent buildings (at a spacing of 25 feet on center). If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

For the existing retaining wall, survey points should be located at the top of the retaining wall at a spacing of 50 feet on center.

APPENDIX D

Seattle Landslide Inventory – 1934 Slide Map

AIRPORT WAY & HOLGATE ST.

Notes by G.W.R. & E.T.McN. - Jan. 9, 1934

Sketch by J.J.C. - Jan. 29, 1934

Scale 1" = 200'

Area = 7.6 Acres

APPENDIX E
Report Limitations and Guidelines for Use

APPENDIX E
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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 Evergreen Treatment Services and other project team members 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 Evergreen Treatment Services dated March 3, 2023 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 Seattle Clinic Renovation project in Seattle, 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.

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

- The function of the proposed structure;

---

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

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

- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

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.

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.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.