Geotechnical Engineering Services Report

Coastal Pacific Food Distributors
Freezer Expansion
322 Valley Avenue NW
Puyallup, Washington

for

Coastal Pacific Food Distributers, Inc.

March 8, 2024

City of Puyallup
Development & Permitting Services
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File No. 27044-001-00

March 8, 2024

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

GeoEngineers, Inc. (GeoEngineers) is pleased to present this final report providing the results of our geotechnical engineering services for the proposed Coastal Pacific Food Distributors Freezer Expansion project. The project site is located at 322 Valley Avenue NW in Puyallup, Washington. A Vicinity Map showing the approximate project location is provided as Figure 1. We prepared a draft geotechnical report dated January 5, 2024 for this project. This final report reflects changes or additions since our draft report was prepared.

Our project understanding is based on our discussions with the project owner's representative, the project civil and structural engineers, other project team members, our previous involvement at the project site, and our review of the following:

- 30-percent plans for the expansion prepared by AWB Engineers and dated July 28, 2023 (provided).
- Project plans for the existing development prepared by HGA Architects and Engineers, LLP and dated October 1, 2004 (provided).
- As-built plans for the existing development prepared by AHBL and dated September 7, 2005 (provided).
- Preliminary design plans for a water tank, including calculations, prepared by CST Storage dated December 12, 2023.
- Geotechnical report for the existing development prepared by GeoEngineers and dated August 18, 2004 (in-house, "2004 Report"). Our 2004 Report included completion of explorations (drilled borings, cone penetrometer tests [CPTs], and test pits) to support analysis for design and construction of the existing development.

The existing facility consists of a single-story structure that consists of an office area, loading dock, and a freezer (warehouse-type) area. The structure is surrounded by paved parking and driveway areas, concrete, and landscaping. Specific development plans are still being finalized; however, we understand the proposed improvements will include expanding the existing facility by connecting to the existing structure and almost tripling the building footprint to nearly 50,000 square feet. Existing structure foundations are reported to accommodate structurally tying in and supporting the expansion; in some cases, new footings will be constructed. Other site improvements include design and construction of a water tank proposed in the northeastern-eastern portion of the site, utilities and pavements, minor grading, and stormwater management. Stormwater facilities will be designed based on criteria provided in the 2019 Washington State Department of Ecology Stormwater Management Manual for Western Washington (SMMWW) and other requirements set forth by the City of Puyallup.

2.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our geotechnical engineering services is to complete subsurface explorations, including a groundwater monitoring well, to characterize soil conditions at the project site and to provide recommendations for geotechnical design and development of the proposed improvements. Our specific scope of services is provided in our proposal dated October 2, and executed October 3, 2023. We can provide this proposal upon request.

Additional services to complete a site-specific response analysis to support seismic design of the new water tank were determined necessary during the course of our initial study. These additional services were authorized on January 31, 2024 as Contract Amendment No. 1. The results of the seismic analysis are still in progress at the time of preparing this report and will be provided in a separate report addendum.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The overall property consists of a rectangular-shaped parcel totaling approximately 5 acres. The property is bounded by existing commercial developments to the north, east and west, and by a section of the future State Route (SR) 167 extension to the south. The Puyallup River is located about 1,000 feet south of the property.

The existing building is located in the central portion of the property and is typically surrounded by landscaping and/or pavements. Vegetated stormwater ponds on the order of 5 to 6 feet deep are located on the north and south sides of the existing building. The remainder of the property generally consists of parking areas, planters, and undeveloped vegetated areas including small trees, which typically border the site.

Site topography is generally level with grade change across property around a few feet or less. Existing site elevations are around Elevation (EL) 39 feet to EL 41 feet. Elevations referenced herein are referenced to the North American Vertical Datum of 1988 (NAVD 88).

3.2. Literature Review

3.2.1. Geology

We reviewed the "Geologic Map of the Tacoma 1:100,000-scale Quadrangle, Washington" (Schuster et al. 2015). According to the map, the site is underlain by Holocene-aged alluvium (Qa). Alluvium is typically described as silt, sand and gravel with occasional estuarine deposits that can be relatively siltier. Alluvium is a water-deposited unit that has not been glacially consolidated and is typically loose to medium dense.

3.2.2. Natural Resources Conservation Service (NRCS) Description

According to the NRCS Web Soil Survey (accessed December 5, 2023) the proposed development area is generally underlain by two soil types: Puyallup fine sandy loam (Unit 31A) and Sultan silt loam (Unit 42A). The Puyallup fine sandy loam is the primary mapped unit and covers the majority of the site; the sultan silt loam is only mapped in the southwest corner of the site.

Puyallup fine sandy loam is described as well drained with a high capacity of the most limiting layer to transmit water and classified in Hydrologic Soil Group A. Sultan silt loam is described as moderately well drained with a moderately high capacity of the most limiting layer to transmit water. Sultan silt loam is classified in Hydrologic Soil Group C/D.

3.2.3. 2004 Report Explorations

Our 2004 Report was completed to support design and construction of the existing development. In preparation of our 2004 Report, we completed 2 drilled borings (B-1 and B-2), 3 CPTs (CPT-1 through CPT-3), and 5 test pits (TP-1 through TP-5) at the site. Approximate locations of the 2004 Report explorations are shown on the Site Plan, Figure 2 and the exploration logs are included in this report in Appendix B. Borings were advanced to depths between about 30 and 50 feet below the existing ground surface (bgs), CPTs extended to about 30 feet bgs, and test pits were excavated to between 8 and 12 feet bgs. Based on our review of the 2004 Report explorations, we interpret two general soil units at the project site: (1) fill and (2) alluvium. A brief description of each unit is provided below.

Fill material was documented in some, but not all, of the 2004 Report explorations (B-1, TP-1, and CPT-1). Where encountered, fill material consisted of loose to dense sand and gravel with variable silt content. Fill extended to depths between about 1½ and 4 feet bgs.

Below the fill in B-1, TP-1, and CPT-1 and beginning near the ground surface of the other explorations, soils we interpret to be alluvium were observed. Alluvium typically consisted of loose to medium dense sand with variable silt content and intermittent approximately 2- to 5-foot-thick layers of medium stiff to very stiff silt. Organic matter consisting of wood fragments was occasionally noted within the alluvium. All the 2004 Report explorations were completed in alluvium at depths between about 12 and 50 feet bgs.

Groundwater was noted in the 2004 Report borings at depths between about 16 and 16½ feet bgs.

3.3. Subsurface Conditions

We explored subsurface conditions at the site for this study by completing five drilled borings (GEI-1 through GEI-5) on November 15, 2023. We completed an additional exploration consisting of a Cone Penetrometer Test (CPT) with shear wave velocity measurements (CPT-01-24) on February 26, 2024 to support seismic design of the proposed water tank. Approximate locations of the explorations completed for this study are shown in Figure 2. Boring GEI-1 was completed as a monitoring well. A pressure transducer was installed in the well to measure groundwater pressures at regular intervals. Details regarding the subsurface exploration program, including summary logs of the explorations, are provided in Appendix A.

Selected samples from our explorations were tested to evaluate engineering properties and to confirm or modify field classifications. Our testing program consisted of grain-size distribution analyses, hydrometer analyses, moisture content determinations, and fines content determinations. Details and the results of our laboratory testing program are provided in Appendix A.

3.3.1. Soil Conditions

At the surface of the explorations, we observed surficial material to consist of either asphalt concrete pavement or sod. Asphalt was observed in borings GEI-1 and GEI-5 and in CPT-01-24 with thickness on the order of 5 inches. Sod was observed in the remaining explorations (GEI-2 through GEI-4) and was typically around 8 inches thick. Below the surficial material, we observed what we interpret to be native alluvium soils. Although not directly observed in the explorations completed for this study, we expect that a few feet of fill material (likely consisting of reworked alluvium soils) could be present across the site as noted in our 2004 Report.

Alluvium varied in composition but typically consisted of interbedded layers of very loose to medium dense sand with variable silt content, soft to medium stiff silt with variable sand content, and isolated pockets of medium dense gravel with variable silt and sand content. Organic matter consisting of wood fragments was occasionally observed within the alluvium. All our borings were terminated within alluvium at depths between about 11½ and 31½ feet bgs. CPT-01-24 was also completed within alluvium at a depth of about 192¼ feet bgs.

3.3.2. Groundwater Conditions

Groundwater was observed during drilling of GEI-1 at depth of about 16¾ feet bgs. Direct groundwater measurement was not possible during advancement of CPT-01-24; however, a porewater pressure dissipation test was completed in the CPT and indicated groundwater around 12 feet bgs.

On February 14, 2024 (91 days after well installation) we measured groundwater levels in GEI-1 to be about 13 feet bgs. During the monitored interval, the pressure transducer in the well indicated that groundwater levels in the well generally fluctuated but had an overall increasing trend. A groundwater hydrograph indicating water levels in the monitoring well at GEI-1, as well as precipitation data from a nearby weather station, are presented in the Groundwater Hydrograph and Precipitation Data, Figure 3. This figure will be updated as additional groundwater data is collected.

Groundwater was not observed during drilling of the remaining borings, GEI-2 through GEI-5, which were terminated at about 11½ to 13 feet bgs, generally above the water levels recorded. We did observe occasional mottling and iron-oxide staining, as noted on the boring logs, which are indications of intermittent presence of groundwater and/or seepage.

Based on our observations during drilling and our understanding of site geology, in our opinion groundwater observed at the site is likely representative of the regional groundwater table and present year-round. Based on current groundwater information we recommend assuming a design groundwater depth of about 10 feet bgs (approximate EL 30 feet).

There is also the potential for perched groundwater to accumulate at relatively shallower depths. It is common for perched groundwater to be present near contacts where soil that is more permeable overlies soil that is less permeable (i.e., sand over silt). Site grading, especially utility cuts into low permeability soils (i.e., silt) that are backfilled with more permeable imported sands and gravels, can also affect the quantity and location of perched groundwater. Perched groundwater is expected to be intermittent and discontinuous at the project site.

The presence of groundwater and groundwater seepage can fluctuate depending on soil conditions, rainfall amounts, irrigation activities and other factors. We anticipate groundwater levels, and the presence of perched groundwater, will generally be highest during the wet season, typically October through May.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. General

Based on our understanding of the project, observations during our explorations and our experience with similar projects, it is our opinion that the proposed improvements can be constructed generally as envisioned with regards to geotechnical considerations. A summary of the primary geotechnical considerations for the project is provided below and is followed by our detailed recommendations.

- We identified potentially liquefiable soils in our explorations. We provide additional discussion on liquefaction and resulting settlement below. Total and differential settlement as a result of liquefaction should be anticipated without deeper improvement to site soils (i.e., ground improvement installation). We understand that management of estimated liquefaction-induced settlements has been considered in the overall design.

- Due to the presence of potentially liquefiable soils, the site is Site Class F per American Society of Civil Engineers (ASCE) 7-16 and the 2018/2021 International Building Code (IBC), and a site-specific seismic evaluation could be required to determine seismic design parameters. ASCE 7-16 allows alternative code applications to be considered if the fundamental period of vibration of the structure is less than or equal to 0.5 seconds.
  - We understand that the building expansion design will fall within this limit and therefore we provide seismic parameters per the ASCE 7-16 alternative code application.
  - We understand that the fundamental period of vibration of the proposed water tank structure will exceed 0.5 seconds. We will provide site-specific seismic design parameters for the tank in a separate report addendum.
  - Site-specific seismic design parameters to be provided separately for the water tank could also be considered for the expansion building structure in lieu of the parameters provided in this report.
- New structures can be adequately supported using conventional shallow foundations and slabs-on-grade. Shallow foundations should be supported on a minimum 2-foot-thick bearing pad of imported crushed rock structural fill. Additional bearing pad thickness may be required if unsuitable soils (i.e., organic-rich soil and/or soft silt) are present.
- Most of the site soils encountered, especially in the upper few feet of our explorations, contain a significant percentage of fines (material passing the U.S. No. 200 sieve). Soil with a higher fines content is more sensitive to small changes in moisture content and may be difficult, if not impossible, to work and compact during wet weather conditions. At this time, we recommend that on-site soils not be considered for reuse as structural fill.
- Groundwater at the site is expected to fluctuate depending on weather and time of year. At this time, we recommend assuming a static design depth to ground water of 10 feet bgs (Elevation 30 feet) for design. Additional groundwater elevation data at the site is being collected at the time of this study. This design groundwater level may be modified as additional groundwater data at the site is collected. Shallow groundwater seepage should be expected to be encountered during almost any time of the year.
- As previously discussed, most of the soils observed in our explorations contain a significant percentage of fines. We also noted intermittent and alternating layers of silt and sands. Based on our experience and laboratory testing of selected samples, the infiltration rate of the silt alluvium would be considered very low or practically infeasible. Site soil layering observed indicates variable and lack of consistent permeable conditions. As such, we do not recommend infiltration as a primary design solution for stormwater management control.

4.2. Seismic Design Considerations

4.2.1. Seismic Design Parameters

We performed an evaluation of seismic design parameters per ASCE 7-16. Based on our understanding of subsurface conditions at the site, we expect that potentially liquefiable soils are present. As a result, the site is Site Class F per ASCE 7-16, and a site-specific seismic evaluation is typically required to determine the seismic design parameters. ASCE 7-16 Section 20.3.1 indicates that for structures with a fundamental period of vibration less than or equal to 0.5 seconds, a site-specific seismic evaluation is not required, and a Site Class is permitted to be determined In accordance with Section 20.3. Provided this period criterion can be satisfied, ASCE 7-16 permits the use of seismic design parameters derived for Site Class D, as shown in Table 1. It should be noted that provisions for Site Class F are still required for other elements of the code not directly related to the development of seismic design parameters (i.e., structural foundation design). If the fundamental period of vibration for the proposed structure exceeds 0.5 seconds, we should be notified to provide updated recommendations.

Further, per ASCE 7-16 Section 11.4.8, ground motion hazard analysis is required for structures on Site Class D with S_1 greater than or equal to 0.2. S_1 is greater than 0.2 for this site; therefore, this provision applies. Alternatively, per ASCE 7-16 Supplement 3 Section 11.4.8, a ground motion hazard analysis is not required where the value of S_{M1} is increased by 50 percent for all applications of S_{M1} and the resulting value of S_{D1} is used for all applications of S_{D1}. This exception was incorporated in the seismic design parameters provided in Table 1 below; however, we can perform ground motion hazard analysis if preferred by the design team.

TABLE 1. SEISMIC DESIGN CRITERIA

2018 IBC Parameters[1]	Value
Code Defined Site Class	F
Site Class for Seismic Design Parameters	D
Mapped MCE_R Spectral Response Acceleration at Short Period, S_s (g)	1.28
Mapped MCE_R Spectral Response Acceleration at 1-second period, S_1 (g)	0.44
Site Modified Peak Ground Acceleration, PGA_M	0.55
Short Period Site Coefficient, F_a	1.00
Long Period Site Coefficient, F_v	1.86
Design Spectral Acceleration at 0.2-second period, S_{DS} (g)	0.85
Design Spectral Acceleration at 1.0-second period, S_{D1} (g)	0.82[2]
Site Modified Earthquake Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.28
Site Modified Considered Earthquake Spectral Response Acceleration at 1-Second Periods, S_{M1} (g)	1.23[2]

Notes:

[1] Parameters developed based on latitude 47.206508 and longitude -122.298852 using the Applied Technology Council (ATC) Hazards online tool (https://hazards.atcouncil.org/).

[2] Per ASCE 7-16 Supplement 3 Section 11.4.8 Item 1, parameter has been increased 50 percent or has increased by 50 percent as a result of the adjusted S_{M1} value.

We understand that the fundamental period of the water tank structure (convective period) exceeds 0.5 seconds and therefore, a ground motion hazard analysis is required to determine seismic design parameters for this structure. Site-specific seismic design parameters for the water tank structure will be provided in a separate report addendum. If desired, these parameters could also be considered for design of the building expansion structure in lieu of those provided in Table 1.

4.2.2. Liquefaction

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in loose, saturated soils and subsequent loss of strength in the deposit of soil affected. In general, soils that are susceptible to liquefaction include loose to medium dense sands to silty sands that are below the water table.

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 soil's resistance to liquefaction. Estimation of the CSR and the CRR were completed using empirical methods (Youd et al. 2001; Idriss and Boulanger 2014). Estimated ground settlement resulting from earthquake induced liquefaction was analyzed using empirical procedures based on standard penetration tests (from borings) and cone penetration tests (Cetin et al. 2009; Idriss and Boulanger 2014). We considered a 2,475-year return period design earthquake event as per IBC 2018 design guidelines. The following design earthquake event parameters were used in our analyses:

- Peak ground acceleration (PGA_m) of 0.55g, per IBC 2018 Site Class D design response spectra determined for the site.
- Mean magnitude earthquake of 7.11 for the 2,475-year return event determined for the site and obtained from the online interactive United States Geological Survey (USGS) Unified Hazard Tool.

The results of our liquefaction analysis indicate significant free field ground settlement may occur due to liquefaction for the design earthquake event considered. Potentially liquefiable soils were observed beginning around 15 feet bgs and intermittently extending to the full depths explored (up to about 50 feet bgs considering our 2004 Report explorations). We estimate the total liquefaction-induced (uniform) settlements at the site could be on the order of about 2 to 5 inches. Differential settlements over a distance of about 100 feet are estimated to be up to about 2.5 inches.

Based on discussions with the design team, we understand that provisions for managing liquefaction settlement estimates has been considered in the design. Reducing these settlements would require modifications to the subsurface, such as ground improvement consisting of stone columns or potentially rigid inclusions. We can provide additional considerations and recommendations for ground improvement design, if desired.

4.2.3. Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on our understanding of site topography, and proposed improvements, it is our opinion that the risk of lateral spreading is low.

4.2.4. Surface Rupture Potential

According to the USGS Interactive Fault Map (accessed December 12, 2023), there are no mapped faults or other seismogenic features within about 4 miles of the site. Based on the proximity of the site to the nearest mapped seismogenic feature and our understanding of local geology (bedrock below the project site overlain by hundreds of feet of soil) it is our opinion the risk for surface rupture at this site is low.

4.3. Site Development and Earthwork

4.3.1. General

We anticipate site development and earthwork activities on site will include: clearing and stripping vegetated areas; site grading; establishing subgrades for driveways, parking areas, and building foundations; and placing and compacting fill and backfill materials. We expect site grading and earthwork can be accomplished with conventional earthmoving equipment.

4.3.2. Clearing and Stripping

Existing surfaces within proposed building areas should be cleared and stripped of all vegetation and organics prior to site development. Minimum estimated stripping depths at the site will likely be on the order of 8 inches. However, greater stripping depths could be required to remove localized zones of loose or organic-rich soil, especially in areas of the site currently vegetated with trees. During clearing and stripping, stumps and primary root systems of shrubs and trees should be completely removed. Voids caused by removal of stumps and/or root systems should be backfilled with compacted structural fill.

Based on our explorations we anticipate soils exposed after stripping could have a high fines content and, thus, be susceptible to disturbance when wet. Care should be taken to avoid allowing these soils to become saturated and disturbed. We provide recommendations for subgrade protection in the "Subgrade Preparation and Protection" section below.

Although not observed in our explorations, cobbles and/or boulders should be removed from structural areas, if encountered. Boulders may be removed from the site or used in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

Structural elements of the existing buildings and pavements should be demolished and removed from within the footprint of the new improvements. During demolition, excessive disturbance of surficial soils may occur, especially if left exposed and/or conducted in wet conditions. Disturbed soils may require additional remediation during construction and grading.

4.3.3. Temporary Erosion and Sedimentation Control

Erosion and sedimentation rates and quantities can be influenced by construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an erosion and sedimentation control plan will reduce the project impact on erosion-prone areas. The plan should be designed in accordance with applicable city, county, and state standards. The plan should incorporate basic planning principles, including:

- Scheduling grading and construction to reduce soil exposure;
- Re-vegetating or mulching denuded areas;
- Directing runoff away from exposed soils;
- Reducing the length and steepness of slopes with exposed soils;
- Decreasing runoff velocities;
- Preparing drainage ways and outlets to handle concentrated or increased runoff;

- Confining sediment to the project site;
- Inspecting and maintaining control measures frequently.

Some sloughing and raveling of exposed or disturbed soil on slopes should be expected. We recommend disturbed soil be restored promptly so surface runoff does not become channeled.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by paving, structure construction or landscape planting.

Until permanent erosion protection is established, and the site is stabilized, site monitoring may be required by qualified personnel to evaluate the effectiveness of the temporary erosion control measures and to repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan.

4.3.4. Temporary Excavations and Cut Slopes

Based on our explorations, it is likely shallow excavations could experience minor caving. Excavations deeper than 4 feet should be shored or laid back at a stable slope if workers are required to enter. Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." Shoring, trench boxes or sloped sidewalls will be required under the Washington Industrial Safety and Health Act (WISHA), regardless of the soil type encountered in the excavation. We recommend contract documents specify that the contractor performing the work is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures.

We recommend for general planning purposes all temporary cut slopes be inclined no steeper than about 1.5H:1V (horizontal:vertical). This guideline assumes all surface loads remain a minimum distance of at least one-half the depth of the cut away from the top of the slope and seepage is not present on the slope face. Flatter cut slopes will be necessary where seepage occurs or if surface surcharge loads are anticipated. Temporary covering with heavy plastic sheeting should be used to protect these slopes during periods of wet weather.

4.3.5. Permanent Cut and Fill Slopes

We recommend permanent slopes be constructed at a maximum inclination of 2H:1V to manage erosion. Where 2H:1V permanent slopes are not feasible, protective facings and/or retaining structures should be considered.

To achieve uniform compaction of fill slopes, we recommend fill slopes be overbuilt and subsequently cut back to expose well-compacted fill. Fill placement on existing slopes steeper than 5H:1V should be benched into the slope face. The configuration of benches depends on the equipment being used and the inclination of the existing slope. Bench excavations should be level and extend into the slope face at least half the width of the compaction equipment used.

Exposed areas should be re-vegetated as soon as practical to reduce surface erosion and sloughing. Temporary protection should be used until permanent protection is established.

4.3.6. Temporary Groundwater Handling Considerations

As previously discussed, we expect that static groundwater at the site is present around 15 feet bgs (EL 25 feet). Perched groundwater could be encountered above this depth, although perched groundwater is expected to be intermittent and discontinuous. We anticipate groundwater levels will vary throughout the year and will generally be highest during the wet season, typically October through May. We expect perched groundwater could be encountered depending on the time of year of construction, likely near contacts where soil that is more permeable overlies soil that is less permeable (i.e., sand over silt).

Groundwater handling needs will typically be lower during the late summer and early fall months. We anticipate shallow perched groundwater can typically be handled adequately with sumps, pumps, and/or diversion ditches, as necessary. Perched groundwater at relatively shallow depths is typically surface water that has recently infiltrated the ground surface. Proactive handling of surface water (i.e., grading to reduce ponding) can reduce groundwater handling needs. If excavations extend more than a few below the static groundwater level (about 15 feet bgs), a more robust dewatering system such as well points will be necessary to maintain a dry excavation. Ultimately, we recommend the contractor performing the work be made responsible for controlling and collecting groundwater encountered.

4.3.7. Surface Drainage

Surface water from roofs, driveways and landscape areas should be collected and controlled. Curbs or other appropriate measures such as sloping pavements, sidewalks and landscape areas should be used to direct surface flow away from buildings, erosion sensitive areas and from behind retaining structures. Roof and catchment drains should not be connected to wall or foundation drains.

4.3.8. Subgrade Preparation and Protection

Subgrades that will support structures or paving should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping and demolition, prior to placing structural fill. We recommend subgrades for structures and pavement 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, where practical.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation which cannot be compacted to a stable and uniformly firm condition, we recommend: (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.

Near surface site soils encountered in our explorations contain a significant amount of fines and will be susceptible to disturbance during periods of wet weather. Soil with high fines content is very sensitive to small changes in moisture and is susceptible to disturbance from construction traffic when wet or if earthwork is performed during wet weather. 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. In our opinion, earthwork will be most efficient during the summer months or during periods of extended dry weather. If wet weather earthwork is unavoidable, we offer the following recommendations:

- 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 compacting with a smooth-drum roller or other appropriate compaction equipment 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 from construction traffic, if needed. Typically, minimum gravel thicknesses on the order of 24 inches are necessary to provide adequate subgrade protection. Maintaining the existing asphalt surfacing is also an adequate method of protection; however, asphalt could become distressed and may need repairs depending on the amount of heavy truck traffic.

Foundation bearing surface protection should also be considered. We provide additional recommendations in the “Shallow Foundations” section of this report.

4.3.9. Cement Treatment

Cement treatment could be considered to stabilize and strengthen subgrade soils, allow on-site materials to become more manageable, and could be considered as an alternative to overexcavation and replacement. In general, 6 to 8 percent (percent cement relative to the moist unit weight of subgrade soils) is typically required to stabilize similar silty and fine-grained soils encountered at shallow depths at this site. For budgeting purposes, we recommend 6 percent as an average value for consideration. More cement is commonly needed for treatment of soils with natural moisture contents above the optimum moisture content (OMC). Excessively wet or saturated materials or mixing of cement in inclement weather will require a higher cement percentage and may not always perform as intended. The cement treatment percentage is typically optimized in the field based on the natural moisture content of the subgrade soil to be treated, condition of the subgrade soil, and by visual observation.

Minimum cement treatment mixing depth should be 18 inches. If there are isolated areas of deep, soft, and pumping soils, it might be more cost effective to remediate these areas with overexcavation and replacement prior to cement treatment. Prior to mixing, the area to be treated should be graded to the design subgrade elevation. The cement should be thoroughly blended and uniformly mixed throughout the soil matrix during placement. The use of rotary and tilling mixing equipment is preferred. After mixing, the cement treated soil should be compacted in place as soon as possible to reduce moisture penetration and maintain firm and unyielding conditions. The presence of granular materials imbedded within the mixture can reduce disturbance and softness.

The treated surface along with finer grained materials encountered at this site will still tend to become disturbed from construction traffic. Excessive turning and repeated loading of the surface with equipment and traffic may also weaken the treated surface, particularly if is exposed to surface water and rain. Treated areas should be closed to traffic and construction equipment after compaction until the surface has become sufficiently stable to resist permanent deformations, typically on the order of 12 to 48 hours. In future paved areas, it may be necessary to protect the treated layer with a 6-inch layer of crushed surfacing base course, especially if traffic will be allowed to operate on the surface for an extended period. Additionally, this can be helpful to reduce the effects of slick surfaces that can develop on the surface of treated materials and provide some additional resiliency. For haul roads, areas of repeated construction traffic, and/or heavier loading conditions, mixing of angular granular materials and/or quarry spalls within the cement treated matrix should be considered.

Cement treatment used to replace overexcavation below structure foundations should be evaluated on a case-by-case basis but likely can be used as a one-to-one replacement where otherwise, overexcavation and replacement with structural fill is required. Additional mixing depths, or use of surfacing granular materials may be necessary to achieve the full 24 inches below foundations, as recommended in this report.

Additional discussion on pavement sections over areas that have cement treated base are provided in the "Pavement Design" section below. We have considered a reduction in the subbase thickness for subgrade areas that have been cement treated and prepared as recommended.

4.4. Fill Materials

4.4.1. Structural Fill

The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. Material used for general structural fill should be free of debris, organic contaminants, and rock fragments larger than 6 inches. For most applications, we recommend that structural fill consist of material similar to "Select Borrow" or "Gravel Borrow" as described in Section 9-03.14 of the Washington State Department of Transportation (WSDOT) Standard Specifications. Ultimately, time of year (wet weather vs. dry weather) of attainment and use of materials should be one of the considerations to determine imported material types.

4.4.2. Select/Wet Weather Granular Structural Fill

Weather, material use, schedule, duration exposed, and site conditions should be considered when determining the type of import fill materials purchased and brought to the site for use as structural fill. If imported fill material is required during periods of wet weather, it should consist of select granular structural fill. Select granular structural fill should consist of material similar to WSDOT Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing) or 9-03.10 (Aggregate for Gravel Base) with the exception that the fines content be less than 5 percent, based on the minus ¾-inch fraction, and the maximum particle size is 6 inches. Gravel backfill for walls as described in WSDOT Specification 9 03.12(2) could also be considered for use as select granular fill. Alternative materials may be considered and should be reviewed and accepted by the project civil and geotechnical engineer.

4.4.3. Crushed Rock

Crushed surfacing base course (CSBC) and crushed surfacing top course (CSTC) should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. We recommend that crushed rock used as structural fill consist of material of approximately the same quality as CSBC. For pavement sections, CSTC may be used where fine grading or grade control is desired.

4.4.4. Quarry Spalls

Quarry spalls should conform to WSDOT requirements 9-13.1(5). If necessary, some variations regarding the particle sizes of this specification, including sizes up to about 8 inches, may be appropriate.

4.4.5. Pipe Bedding

Trench backfill for the bedding and pipe zone should consist of well-graded granular material similar to "Gravel Backfill for Pipe Zone Bedding" described in Section 9-03.12(3) of the WSDOT Standard Specifications. The material must be free of roots, debris, organic matter, and other deleterious material. Other materials may be required depending on pipe manufacturer specifications and/or jurisdictional requirements and should be considered.

4.4.6. Trench Backfill

Trench backfill must be free of debris, organic material, and rock fragments larger than 6 inches. We recommend trench backfill material consist of imported material similar to "Select Borrow" or "Gravel Borrow" as described in the "Structural Fill" section above. During wet weather, select/wet weather granular fill may be required as trench backfill. Pipe manufacturer and jurisdictional requirements should also be considered when choosing trench backfill material.

4.4.7. On-site Soil as Structural Fill

Based on our explorations, near-surface soils at the site typically contain high silt content, are moisture sensitive, and will likely be generated above optimum moisture content and will be very difficult to manage and compact. Practically speaking, these soils will be nearly impossible to handle and compact during wet weather conditions. Unless special provisions are considered in project plans and specifications along with time and budget allowance for use of these materials, we recommend that they not be considered for reuse as structural fill.

4.5. Fill Placement and Compaction

To obtain proper compaction, fill material should be compacted near optimum moisture content and in uniform horizontal lifts. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Lift thickness and compaction procedures will depend on the moisture content and gradation characteristics of the soil and the type of compaction equipment used. Compaction should be achieved by mechanical means. Generally, 12-inch-thick loose lifts are appropriate for steel-drum vibratory roller compaction equipment. During fill and backfill placement, regular testing of in-place density should be conducted to check that adequate compaction is being achieved.

Fill placed to raise site grades and materials under pavements and structural areas should be placed on subgrades prepared as previously recommended. Fill material placed below structures, including slab areas and footings must be compacted to at least 95 percent of the theoretical maximum dry density (MDD) per

ASTM International (ASTM) D 1557. In paved areas, fill must be compacted to at least 95 percent of the MDD in the upper 2 feet below pavement subgrade. Fill placed below a depth of 2 feet from subgrade in paved areas must be compacted to at least 90 percent of the MDD.

Fill material placed in landscaping areas should be compacted to a firm condition that will support construction equipment, as necessary, typically around 85 to 90 percent of the MDD.

4.6. Shallow Foundations

4.6.1. General

We anticipate proposed structures can be adequately supported on shallow foundations and slabs-on-grade. We understand that portions of the existing building were designed to accommodate the expansion structure and new loads will be distributed on existing shallow foundations. Other portions of the expansion structure will be supported on newly constructed shallow foundations.

The upper soils observed in our explorations were variable and typically consisted of loose sand with variable silt content to soft to stiff sandy silt. Due to the variability in density and composition of shallow soils below foundations and the potential for excessive differential settlement, we recommend foundations for the proposed structures not bear directly on these soils. We recommend overexcavation of existing soils and replacement with a bearing pad consisting of crushed rock structural fill as discussed in the section below.

4.6.2. Footing Bearing Surface Preparation

Our specific bearing surface preparation recommendations are as follows:

- Existing soils should be overexcavated at least 2 feet below footings and replaced with compacted structural fill. Structural fill should extend at least 2 feet laterally beyond the edges of the footings.
- If soft fine-grained soils are present, these soils should be overexcavated up to 5 feet to expose underlying granular soils (i.e., alluvium sand) and replaced with structural fill. Considerations to reduce this 5-foot excavation depth may be determined in the field and substituted with geotextiles, fabrics, and/or quarry spalls. Subgrade soils upon overexcavation should be observed and confirmed by GeoEngineers. Where practical, structural fill extending to granular soils should extend laterally beyond the edge of the footings a distance equal to the thickness of the fill or 2 feet, whichever is less.
- Soils at the base of overexcavations should be proof compacted to a uniformly firm and unyielding condition prior to placement of structural fill for the bearing pad.
- Structural fill used for backfilling overexcavations should consist of crushed rock as described in the "Fill Materials" of this report. Structural fill placed below foundations should be compacted to at least 95 percent of the theoretical MDD of the material as determined per ASTM D 1557.

Footing excavations should be performed using a smooth-edged bucket to limit bearing surface disturbance. The foundation bearing surface should be prepared as discussed above and then compacted as necessary to a firm, non-yielding condition. Loose or disturbed materials present at the base of footing excavations must be removed or compacted. If soft or otherwise unsuitable areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition the following options may be considered: (1) unsuitable soils be moisture conditioned and recompacted; (2) unsuitable soils be overexcavated and replaced with compacted structural fill, as needed. Overexcavation should extend to soils that are firm enough such that backfilled structural fill can be adequately compacted. Organic debris or organic-rich soils, if encountered below foundations, should be completely removed.

Prepared foundation bearing surfaces should be evaluated by a GeoEngineers representative to confirm bearing surfaces have been prepared in accordance with our recommendations. Foundation bearing surfaces must not be exposed to standing water. If water pools in the base of the excavation, it should be removed before placing structural fill or reinforcing steel. During periods of wet weather, structural fill and concrete should be placed as soon as practical after preparation of the footing excavations. If footing excavations will be exposed to extended wet weather conditions, a lean concrete mat can be considered for subgrade protection.

4.6.3. Dimensions and Allowable Soil Bearing Pressure

Exterior footings should be established at least 18 inches below the lowest adjacent final grade. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Continuous column footings should have a minimum width of 18 inches. Isolated column footings should have a minimum width of 24 inches.

Based on our review of our 2004 Report, an allowable bearing pressure of 3,000 psf was recommended for design of the existing building and future expansion, assuming shallow foundations were constructed on a minimum 2-foot-thick bearing pad consisting of crushed rock structural fill. Provided new loads combined with loads from the existing structure do not result in an exceedance of the allowable bearing pressure, we expect the existing footings will provide adequate bearing support.

For design of new shallow foundations, we also recommend an allowable soil bearing pressure of 3,000 psf for design provided foundations are constructed as recommended above. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

4.6.4. Shallow Foundations Near Existing Improvements

We understand that there could be new shallow foundations for the expansion structure that are constructed near existing structural improvements such as shallow foundations for the existing structure.

Foundations constructed adjacent to existing foundations can induce additional loading (load influence) on soils below the foundations, which can result in bearing and settlement considerations. Additionally, excavation for shallow foundations near existing footings can result in undermining and loss of subgrade support below the existing footings. In general, ways to avoid this could include the addition of shallow drilled piers below the adjacent foundations. Other alternatives include foundation setbacks and developing a structural solution (i.e., “bridge”) to connect older and newer buildings.

We should review all new foundations (including loads, elevations, and sizes) that are planned constructed within about 10 feet or less of existing structural improvements to provide specific recommendations, as necessary.

4.6.5. Foundation Settlement

We estimate settlement of footings designed and constructed as recommended will be less than 1 inch, with differential settlements of less than ½ inch between comparably loaded isolated column footings or along 50 feet of continuous footing. Settlement is expected to occur rapidly as loads are applied. Settlements could be greater than estimated if loose/soft or disturbed soil such as soft silt is present beneath footings.

These estimates are based on footing loads proportioned using the recommended allowable bearing pressure above. We should be notified once foundation loads are available to review and revise our settlement estimate, if necessary.

4.6.6. Lateral Resistance

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs, and the passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings founded in accordance with the recommendations presented above, the allowable frictional resistance on the base of the footing may be computed using a coefficient of friction of 0.35 applied to the vertical dead-load forces.

The allowable passive resistance on the face of the footing or other embedded foundation elements may be computed using an equivalent fluid density of 230 pounds per cubic foot (pcf) for undisturbed and firm site soils or structural fill extending out from the face of the foundation element a horizontal distance at least equal to 2.5 times the depth of the element.

These values include a factor of safety of about 1.5. The passive earth pressure and friction components may be combined provided the passive earth pressure component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and groundwater remains below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with pavement or a slab-on-grade.

4.6.7. Foundation Drains

Based on the potential presence of granular structural fill overlying low-permeability silts below foundations, we expect there is the potential for accumulation of shallow perched groundwater, and as such, we recommend that perimeter foundation drains be considered. Perimeter foundation drains would help maintain bearing pressures provided, drier conditions around the structure during wetter times of the year, help reduce migration of water below the building slab, or if there was near-surface seepage from other means such as irrigation and landscaping.

We recommend perimeter footing drains be installed at the base of exterior footings and include cleanouts. Drains should be installed within a 12-inch-deep trench and consist of perforated pipe at least 4 inches in diameter placed on an approximate 3- to 4-inch-thick bed of drainage material. Perforated pipe should be surrounded by 5 to 6 inches of drainage material on the remaining sides enclosed in a non-woven geotextile fabric to prevent fine soil from migrating into the drain material. We recommend drainage pipe consist of heavy-wall solid pipe (SDR-35 polyvinyl chloride [PVC], or equal). We do not recommend using flexible tubing. The drainage pipe should be either machine-slotted or perforated over the lower 60-degree perimeter of the pipe. For slotted pipe, the slots should be a maximum of ⅛-inch wide with four slots per inch. Perforated pipe should have two rows of ½-inch diameter holes spaced 120 degrees apart and at 4 inches on center. Roof downspout and retaining wall drain lines should not be routed to footing or below-grade drain lines. The drainage material should consist of material recommended for wall and footing drains described in the "Retaining Structure Drainage" section of this report.

4.7. Slab-on-Grade Floors

Provided deleterious material is not present, the existing alluvium may remain in place below building slabs provided it can be proof compacted to a uniformly firm, dense, and unyielding condition. We recommend slab-on-grade floors be underlain by a minimum 8-inch-thick capillary break layer to provide a more uniform and consistent support across the building area and to reduce the potential for moisture migration into the slab. The capillary break material should consist of a well-graded sand and gravel, crushed rock (CSBC) or washed rock with a maximum particle size of ¾ inch and less than 5 percent fines. The material should be placed as recommended in the "Fill Placement and Compaction" section of this report.

Provided slab subgrades are prepared as recommended, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) can be used for designing building floor slabs. If slabs are supported on compacted structural fill, a higher value may be possible and should be further evaluated.

This value is for a 1-foot by 1-foot square plate. The modulus of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1}[(B+1)/2B]^2$$

Where k_s is the modulus of subgrade reaction, k_{s1} is the modulus of subgrade reaction for a 1-foot by 1-foot plate, and B is the minimum width or lateral dimension of the mat.

The exposed subgrade should be evaluated after site grading is complete. We recommend slab subgrades be evaluated by a GeoEngineers representative during construction. Disturbed areas should be compacted, if possible, or removed and replaced with compacted structural fill. In all cases, the exposed soil should be proof compacted to a firm and unyielding condition.

In our opinion, an underslab drainage system is not necessary. However, if dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to slab), a waterproof liner may be placed as a vapor barrier below the slab.

Settlement for floor slabs designed and constructed as recommended is estimated to be less than ¾ inch for a floor load of 500 psf. We estimate that differential settlement of floor slabs will be ½ inch or less over a span of 50 feet.

4.8. Retaining Walls and Below-Grade Structures

4.8.1. Design Parameters

We recommend the following lateral earth pressures be used for design of conventional retaining walls and below-grade structures. Our design pressures assume that the ground surface around the retaining structures will be level or near level. If drained design parameters are used, drainage systems must be included in the design in accordance with the recommendations presented in the "Retaining Structure Drainage" section below.

The active soil pressure condition assumes the wall is free to move laterally 0.001 H, where H is the wall height. The at-rest condition is applicable where walls are restrained from movement. The above recommended lateral soil pressures do not include the effects of sloping backfill surfaces or surcharge loads, except as described. Overcompaction of fill placed directly behind retaining walls or below-grade structures must be avoided to limit lateral pressures placed on the wall. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet of retaining walls and below-grade structures.

- Active soil pressure may be estimated using an equivalent fluid density of 38 pcf for the drained condition.
- Active total soil and hydrostatic pressure may be estimated using an equivalent fluid density of 80 pcf for the undrained condition; this value includes hydrostatic pressures.
- At-rest soil pressure may be estimated using an equivalent fluid density of 58 pcf for the drained condition.
- At-rest total soil and hydrostatic pressure may be estimated using an equivalent fluid density of 89 pcf for the undrained condition; this value includes hydrostatic pressures.
- For seismic considerations, a uniform lateral pressure of 12*H psf (where H is the height of the retaining structure or the depth of a structure below ground surface) should be added to the lateral earth pressure.
- If sloping conditions are present above or below new walls, we should be contacted to provide updated recommendations.
- A traffic surcharge can be estimated should be included if vehicles are allowed to operate within a zone equal to the height of the retaining walls. This can be estimated with a uniform horizontal load of 80 psf, applied in addition to the pressures presented above, or by assuming an additional 2 feet of fill. This is based on a uniform surface load of 250 psf; other surface loads should be considered on a case-by-case basis.

Retaining wall foundations may be designed using the recommendations presented above for building foundation design. We estimate settlement of retaining structures will be similar to the values previously presented for structure foundations.

4.8.2. Retaining Structure Drainage

If retaining walls or below-grade structures are designed using drained parameters, a drainage system behind the structure must be included to collect water and prevent the buildup of hydrostatic pressure against the structure. We recommend the drainage system include a zone of free-draining backfill against the back of the wall to within about 12 to 18 inches from the top of the wall. This drainage layer can consist of either an 18-inch-thick horizontal layer of a graded drainage material such as WSDOT Specification 9-03.12(4) (Gravel Backfill for Drains) or a 12-inch-thick layer of pea gravel with a non-woven geotextile designed for soil separation placed between the pea-gravel and backfill. Drain boards or other prefabricated drainage systems can be used provided they can be adequately connected to an appropriate collection and discharge pipe system.

A perforated, rigid, smooth-walled drainpipe with a minimum diameter of 4 inches should be placed along the base of the structure within the free-draining backfill and extend for the entire wall length. The drainpipe should be metal or rigid PVC pipe and be sloped to drain by gravity. Discharge should be routed to appropriate discharge areas and to reduce erosion potential. Cleanouts should be provided to allow routine maintenance. Roof downspouts or other types of drainage systems must not be connected to retaining wall drain systems.

4.9. Infiltration Feasibility Assessment

4.9.1. General

We understand plans to manage stormwater at the site could include on-site infiltration, if feasible. On-site stormwater facilities will be designed in general accordance with the 2019 Ecology SWMMWW.

Per the SWMMWW, design infiltration rates can be determined using the Soil Grain Size Analysis Method only for sites underlain soils that are not glacially consolidated. Native near surface soils observed in our explorations consisted of alluvium, which are not glacially consolidated and therefore we expect that the Soil Grain Size Analysis Method is acceptable for infiltration rate determination at the project site.

We observed alluvium at the site to typically include interbedded layers of sand and silt at various depths and thicknesses. Based on our experience in similar soils and our laboratory test results, fine-grained soils such as the silts observed at the site will have a very low infiltration rate. We estimate design rates on the order of practically zero to 0.35 inches per hour. In addition, the silts present at the site would generally be considered a limiting or impermeable layer in regard to infiltration capacity. Due to the presence of intermittent layers of low-permeability silts observed in our explorations (particularly within the upper 10 feet of the ground surface), the variation in thicknesses, and the interbedding, it is our opinion that on-site infiltration would not be considered practical for the project. If absolutely necessary to infiltrate, we should be consulted to discuss depths, locations, further in-situ and location-specific testing, and risks involved. It is likely that results from additional testing would not be much better than presented above.

4.10. Pavement Design

4.10.1. General Design Criteria

We understand asphalt concrete (AC) and/or Portland cement concrete (PCC) pavements may be used for the proposed improvements. We anticipate that pavements for the proposed improvements will include new parking areas, driveways, and sidewalks. Our recommended pavement sections provided below are based on our explorations and experience in the area.

The recommended pavement sections below may not be adequate for heavy construction traffic loads such as those imposed by concrete transit mixers or dump trucks. Additional pavement thickness may be necessary to prevent pavement damage during construction. The recommended sections assume final improvements surrounding the pavement areas will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not accumulate below the pavement section or pond on pavement surfaces. If pavements in parking areas slope inward (toward the center of the parking area) full depth curbs or other measures should be used to prevent water from entering and ponding on the subgrade and within the base section.

4.10.2. Pavement Construction Considerations

Existing pavements, hardscaping, or other structural elements, if present, should be removed prior to placement of new pavement sections. Pavement subgrade should be prepared to a uniformly firm, dense, and unyielding condition as previously described. CSBC and subbase should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent of the MDD (ASTM D 1577).

CSBC should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. Subbase should conform to applicable sections of 4-02 “Gravel Base” and 9-03.10 “Aggregate Gravel for Base” of the WSDOT Standard Specifications. Hot mix asphalt should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications. PCC mix design should conform with Section 5-05.3(1) of the WSDOT Standard Specifications. Aggregates for PCC should conform to applicable sections of 9-03.1 of the WSDOT Standard Specifications.

Some areas of pavement may exhibit fatigue cracking over time. Cracks in the pavement will allow water to infiltrate to the underlying base course, which could increase the amount of pavement damage caused by traffic loads. To prolong the effective life of the pavement, cracks should be sealed as soon as possible.

4.10.3. Asphalt Concrete Pavement Design

We provide recommended conventional AC pavement sections below.

4.10.3.1. Standard-Duty AC Pavement – Automobile Driveways and Parking Areas

- 2 inches of hot mix asphalt, class ½-inch, PG 58-22
- 4 inches of CSBC
- 6 inches of subbase consisting of select granular fill as previously described, to provide a uniform grading surface, to provide pavement support, to maintain drainage and to provide separation from finer grained subgrade soils
- Subgrade proof-compacted to a firm and unyielding condition or structural fill prepared in accordance with the “Subgrade Preparation and Protection” section of this report

4.10.3.2. Heavy-Duty AC Pavement –Areas Subject to Occasional Heavy Truck Traffic

- 3 inches of hot mix asphalt, class-½ inch, PG 58-22
- 6 inches of CSBC
- 12 inches of subbase consisting of select granular fill, previously described, to provide a uniform grading surface, to provide pavement support, to maintain drainage and to provide separation from subgrade soils
- Subgrade proof-compacted to a firm and unyielding condition or structural fill prepared in accordance with the “Subgrade Preparation and Protection” section of this report

4.10.4. Portland Cement Concrete

We understand PCC pavements will likely be used for sidewalk areas at the site and may also be considered for driveway and parking areas. We recommend concrete pavements in vehicular areas be jointed and that dowel bars be included at the joints to assist in load transfer. Dowels should not be included between exterior pavement slabs and interior pavement slabs to reduce the risk of cracking occurring due to differential settlements.

4.10.4.1. Sidewalk PCC Pavement – Pedestrian Areas Not Subjected to Vehicle Loading

- 4 inches (minimum) of PCC with a minimum 14-day flexural strength of 650 pounds per square inch (psi)
- 2 inches (minimum) of compacted CSTC
- Subgrade proof-compacted to a firm and unyielding condition or structural fill prepared in accordance with the "Subgrade Preparation and Protection" section of this report

4.10.4.2. Light Duty PCC Pavement– Automobile Driveways and Parking Areas

- 6 inches (minimum) of PCC with a minimum 14-day flexural strength of 650 psi
- 4 inches (minimum) of compacted CSBC
- 4 inches of subbase consisting of select granular fill to provide a uniform grading surface and pavement support, to maintain drainage and to provide separation from subgrade soils
- Subgrade proof-compacted to a firm and unyielding condition or structural fill prepared in accordance with the "Subgrade Preparation and Protection" section of this report

4.10.4.3. Heavy Duty PCC Pavement–Areas Subject to Heavy Truck Traffic

- 9 inches (minimum) of PCC with a minimum 14-day flexural strength of 650 psi
- 4 inches (minimum) of compacted CSBC
- 6 inches of subbase consisting of select granular fill to provide a uniform grading surface and pavement support, to maintain drainage and to provide separation from subgrade soils
- Subgrade proof-compacted to a firm and unyielding condition or structural fill prepared in accordance with the "Subgrade Preparation and Protection" section of this report

4.10.5. Pavement Areas with Cement Treated Subgrade

Where cement treatment is completed in paved areas, the above section recommendations still apply with the exception that the subbase section can be omitted. We recommend that the pavement (asphalt or PCC) thickness remain the same and that the CSBC base course still be considered to facilitate drainage below the overlying hardscaping.

5.0 LIMITATIONS

We have prepared this report for the Coastal Pacific Food Distributors Freezer Expansion project in Puyallup, Washington. Coastal Pacific Food Distributors may distribute copies of this report to authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices for geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment, and experience. No warranty, express or implied, applies to the services or this report.

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

6.0 REFERENCES

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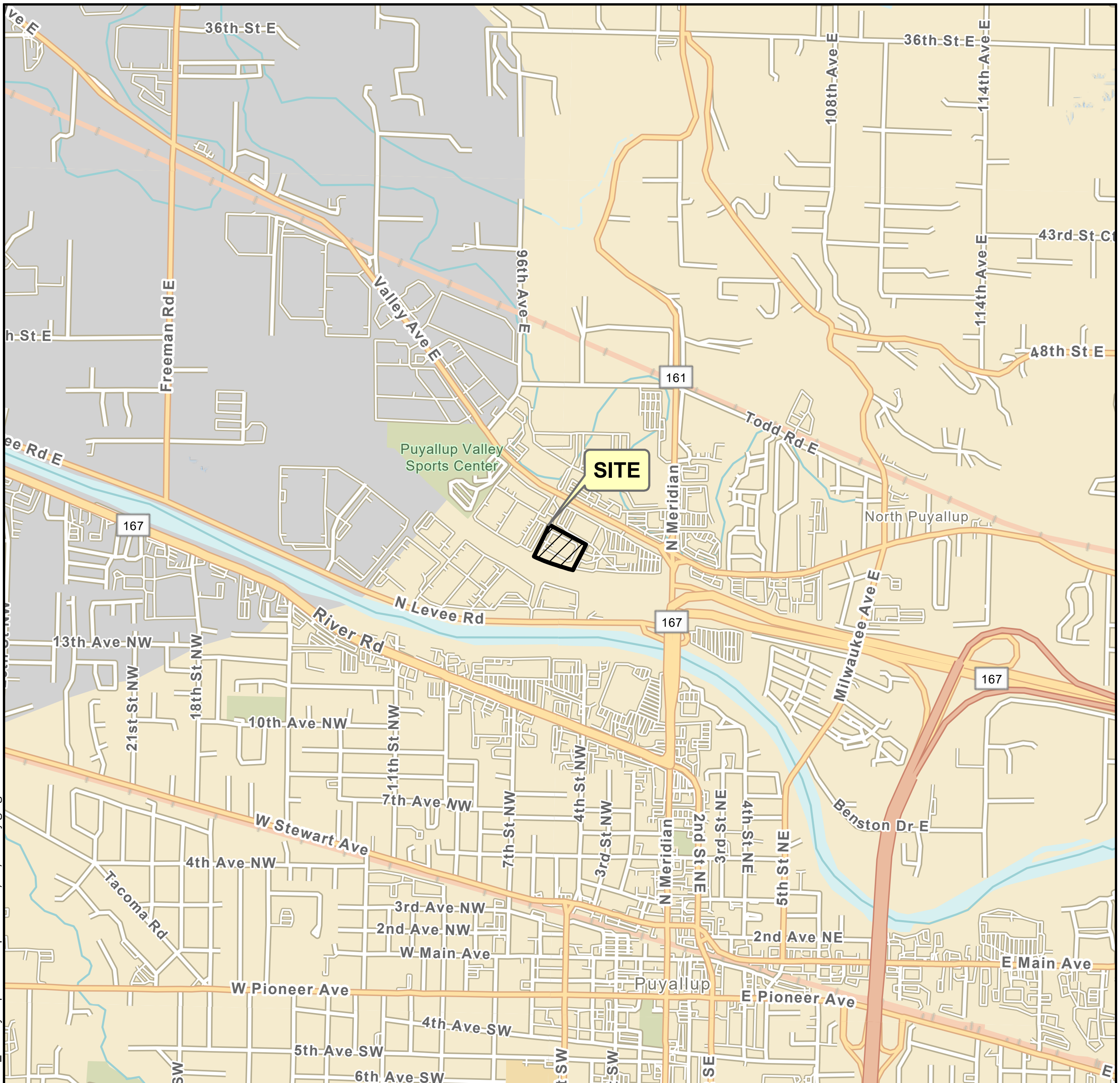
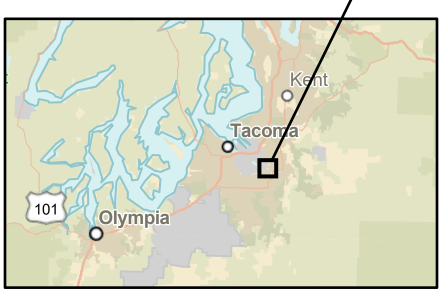
FIGURES





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Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 StatePlane Washington South FIPS 4602 Feet

Vicinity Map

Coastal Pacific Food Distributors - Freezer Expansion
Puyallup, Washington

GEOENGINEERS

Figure 1

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Legend

Site Boundary

CPT-01-24 Cone Penetration Test with Shear Wave Velocity Measurements by GeoEngineers, 2024

GEI-1 Boring by GeoEngineers, Inc., 2023

B-1 Boring by GeoEngineers, Inc., 2004

C-1 Cone Penetration Test by GeoEngineers, 2004

TP-1 Test Pit by GeoEngineers, Inc., 2004

Source(s):

- Aerial from Microsoft Bing Images

Projection: WA State Plane, South Zone, NAD83, US Foot

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

N

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Feet

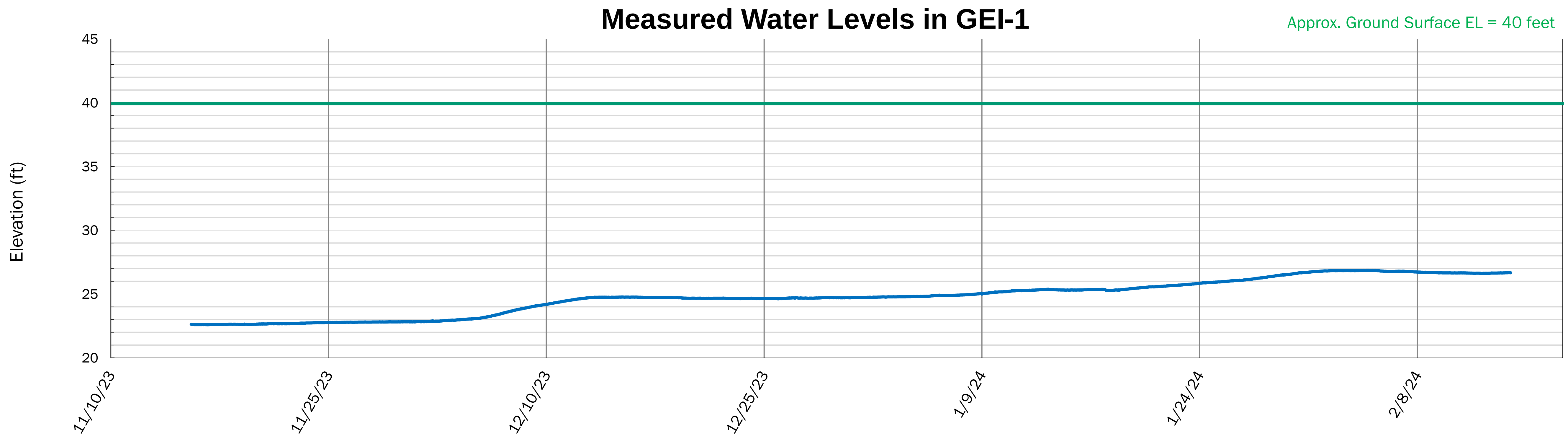
Site Plan

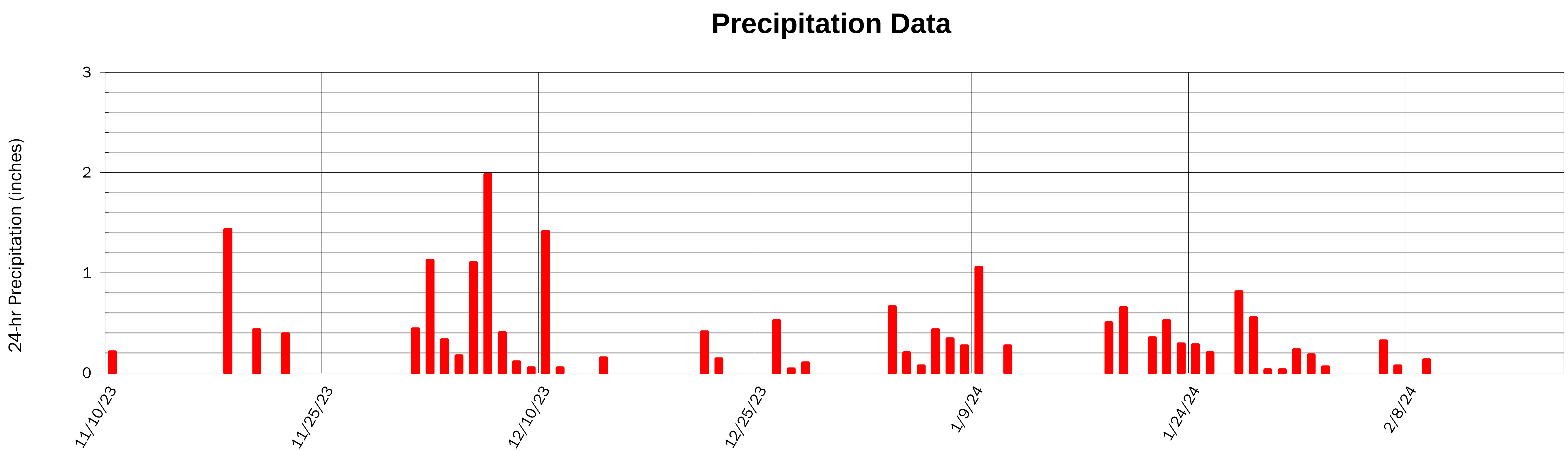
Coastal Pacific Food Distributors - Freezer Expansion
Puyallup, Washington

GEOENGINEERS

Figure 2

P:\27\27044001\CAD\00\Geotech Report\2704400100_F02_Site Plan.dwg 2 Date Exported:3/7/2024 2:18 PM - by Gabby Register





Notes:

1. 24-hour precipitation data obtained from weather station Puyallup 2.1 ESE in Puyallup, Washington.
2. Elevations are referenced to the North American Vertical Datum of 1988 (NAVD88) and should be considered approximate.
3. 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.

GEI-1 Groundwater Hydrograph and Precipitation Data

Coastal Pacific Food Distributors – Freezer Expansion
Puyallup, Washington

GEOENGINEERS

Figure 3

27044-001-00 Date Exported 02/15/2024

APPENDICES

GeoEngineers

APPENDIX A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

Drilled Borings

Soil and groundwater conditions at the site were explored by advancing five drilled borings on November 15, 2023. Locations of the borings were determined via an electronic tablet with global positioning system (GPS) software and are shown in the Site Plan, Figure 2. Locations were selected to target the proposed development area but were constrained to some degree by existing site infrastructure, grading, vegetation, and underground utilities. The locations and elevations of the explorations should be considered approximate.

The borings were performed using truck-mounted drilling equipment provided and operated by Holocene Drilling, Inc. under subcontract to GeoEngineers. Borings were advanced using hollow-stem auger drilling methods and advanced to depths between approximately 11½ and 31½ feet below existing site grade (bgs). Boring GEI-1 was completed as a monitoring well after drilling. The other borings were backfilled by the driller in accordance with Washington State Department of Ecology requirements. Soil cuttings generated from the borings were placed in metal barrels and hauled off by the driller for off-site disposal.

During the exploration program our field representative continuously monitored the borings, obtained representative soil samples, classified the soils, maintained a detailed log of each exploration, and observed groundwater conditions. Soil samples were obtained from the borings using a 1.4-inch-inside-diameter split-barrel sampler driven into the soil using a 140-pound hammer free-falling a distance of 30 inches. The number of blows required to drive the sampler the last 12 inches or other indicated distance is recorded on the logs as the blow count. Our field representative made sample attempts at 2½- to 5-foot-depth intervals. Samples were retained in sealed plastic bags to prevent moisture loss. The soils were classified visually in general accordance with ASTM International (ASTM) D 2488 and Figure A-1, which includes a Key to the Exploration Logs. Summary logs of the explorations are included as Figures A-2 through A-6.

CPT Sounding

We advanced one cone penetrometer test (CPT) sounding to 192¼ feet bgs on February 26, 2024. Prior to cone advancement, the upper approximately 1¾ feet were “punched out” which consisted of driving a casing through the pavement and upper soils.

The CPT sounding and pavement core were advanced and completed using equipment and operators under subcontract to GeoEngineers. The CPT sounding involves pushing an instrumented steel probe into the ground and continuously recording soil friction, tip resistance and dynamic pore pressure using electronic methods. No soil samples are obtained during CPT soundings. Soil types and equivalent SPT “N” values are interpreted based on empirical relationships between measured CPT parameters described above.

Our representative assisted in coordination of the CPT and located the exploration in the field using an electronic tablet with GPS software. The exploration location should be considered approximate and is indicated in the Site Plan, Figure 2.

Laboratory Test Results

Soil samples obtained from the explorations were retained in sealed plastic bags to prevent moisture loss and transported to the GeoEngineers laboratory. Representative soil samples were selected for laboratory tests to evaluate the pertinent geotechnical engineering characteristics of the soils and to confirm our field classification. The following paragraphs provide a description of the tests performed.

Particle Size Gradation – Sieve Analysis (SA)

Sieve analyses were performed on selected samples in general accordance with ASTM D6913. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes larger than 75 micrometers (µm) is determined by sieving. The results of the tests were used to verify field soil classifications and determine pertinent engineering characteristics. Figure A-7 presents the results of our sieve analyses.

Particle Size Gradation – Hydrometer Analysis (HA)

Hydrometer analyses were performed on selected samples in general accordance with ASTM Test Method D 7928. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes smaller than 75 µm is determined by a sedimentation process using a hydrometer. The hydrometer analysis alone determines the distribution of particle sizes smaller than 2 millimeters (mm). Figure A-7 includes the results of our hydrometer analyses.

Percent Fines (%F)

Selected samples were “washed” through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve (fines). Tests were conducted in general accordance with ASTM D 1140. Test results are presented on the exploration logs at the respective sample depths.

Moisture Content (MC)

The moisture content of selected samples was determined in general accordance with ASTM Test Method D 2216. The test results are used to aid in soil classification and correlation with other pertinent engineering soil properties. The results are presented on the test pit logs at the depth tested.

SOIL CLASSIFICATION CHART

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

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

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel / Dames & Moore (D&M)
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

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

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure A-1

Drilled	Start 11/15/2023	End 11/15/2023	Total Depth (ft)	31.5	Logged By AvD, Checked By CJL	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo	DOE Well I.D.: BPQ 645 A 2-in well was installed on 11/15/2023 to a depth of 25 ft.
Surface Elevation (ft) Vertical Datum	40 NAVD88	Top of Casing Elevation (ft)		
Easting (X) Northing (Y)	1192850 688140	Horizontal Datum	WA State Plane South NAD83 (feet)	

Groundwater

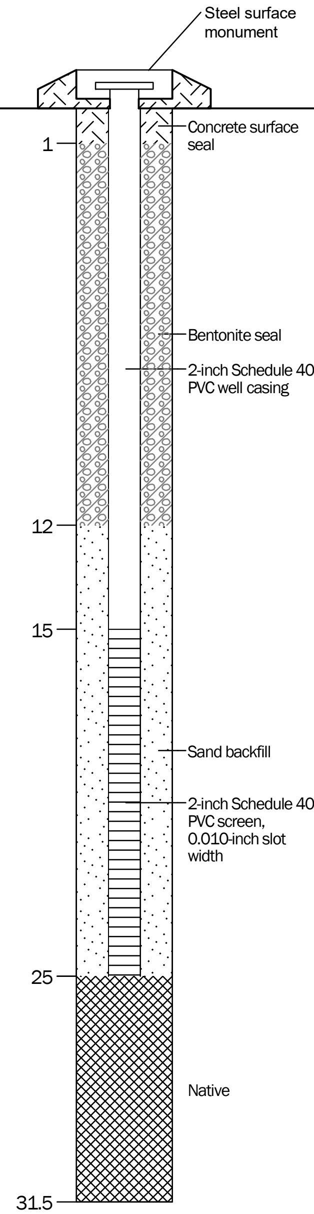
Date Measured	Depth to Water (ft)	Elevation (ft)
11/15/2023	16.70	23.30

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
40	0					AC	Approximately 5 inches of asphalt concrete pavement		
						SM	Dark gray with occasional iron oxide staining silty fine sand (loose, moist) (alluvium)		
		12	8	1A 1B			Grades to brown		
35	5	18	9	2 %F				6	16
		18	9	3					
30	10	18	3	4 SA/HA		ML	Gray-brown with iron oxide staining silt with occasional sand (soft, moist)	32	90
25	15	18	17	5 MC	▼	SP-SM	Gray with iron oxide staining fine sand with silt and organic matter (wood fragments) (medium dense, wet)	28	
20	20	18	16	6			Grades to dark gray, without organic matter		
15	25	18	22	7A 7B		GP	Dark gray fine gravel with sand and organic matter (wood fragments) (medium dense, wet)		
						SP-SM	Dark gray fine to medium sand with silt (medium dense, wet)		
10	30	18	10	8					

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on DEM.

Log of Monitoring Well GEI-1

GEOENGINEERS

Project: Coastal Pacific Food Distributors Freezer Expansion
Project Location: Puyallup, Washington
Project Number: 27044-001-00

Figure A-2
Sheet 1 of 1

Drilled	Start 11/15/2023	End 11/15/2023	Total Depth (ft) 11.5	Logged By AvD, Checked By CJL	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 40; Vertical Datum NAVD88	Hammer Data: Autohammer, 140 (lbs) / 30 (in) Drop	Drilling Equipment: Diedrich D-50 Turbo
Easting (X) 1193010; Northing (Y) 688030	System Datum: WA State Plane South, NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0					SOD	Approximately 8 inches of sod			
						ML	Brown sandy silt (medium stiff, moist) (alluvium)			
		18	6	1 / MC				23		
35	5	18	6	2			Grades with mottling and iron oxide staining			Interbedded silt and sand to 10 feet
		18	13	3 / SA/HA			Grades to stiff	10	54	
30	10	18	8	4			Grades to medium stiff			Increased moisture content

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on DEM.

Log of Boring GEI-2

GEOENGINEERS

Project: Coastal Pacific Food Distributors Freezer Expansion
Project Location: Puyallup, Washington
Project Number: 27044-001-00

Drilled	Start 11/15/2023	End 11/15/2023	Total Depth (ft)	11.5	Logged By AvD	Checked By CJL	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	41 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo
Easting (X) Northing (Y)	1193250 688000	System Datum	WA State Plane South NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Approximately 8 inches of sod			
40								SM	Brown-gray with iron oxide staining silty fine to coarse sand (medium dense, moist)			
			18	10		1 / MC				21		
35	5		18	10		2						Interbedded layers of silt and sand to 10 feet
			18	3		3 / %F		ML	Brown-gray silt with sand (soft, moist)	25	78	
30	10		18	6		4			Grades to medium stiff			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on DEM.

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Log of Boring GEI-3

GEOENGINEERS

Project: Coastal Pacific Food Distributors Freezer Expansion
Project Location: Puyallup, Washington
Project Number: 27044-001-00

Drilled	Start 11/15/2023	End 11/15/2023	Total Depth (ft)	11.5	Logged By AvD	Checked By CJL	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 42 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment Diedrich D-50 Turbo	
Easting (X) 1193320 Northing (Y) 688110	System Datum WA State Plane South NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Approximately 8 inches of sod			
40								ML	Orange-brown with iron oxide staining sandy silt with occasional organic matter (fine roots) (stiff, moist) (alluvium)			
			18	10		1 / MC				20		
	5		18	8		2 / %F		SM	Brown silty fine sand with occasional organic matter (fine roots) (loose, moist)	12	48	
35												
			18	6		3						
	10		18	1		4			Grades to very loose			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on DEM.

Log of Boring GEI-4

GEOENGINEERS

Project: Coastal Pacific Food Distributors Freezer Expansion
Project Location: Puyallup, Washington
Project Number: 27044-001-00

Drilled	Start 11/15/2023	End 11/15/2023	Total Depth (ft) 13	Logged By AvD Checked By CJL	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 41 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment Diedrich D-50 Turbo
Easting (X) 1193160 Northing (Y) 688280	System Datum WA State Plane South NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

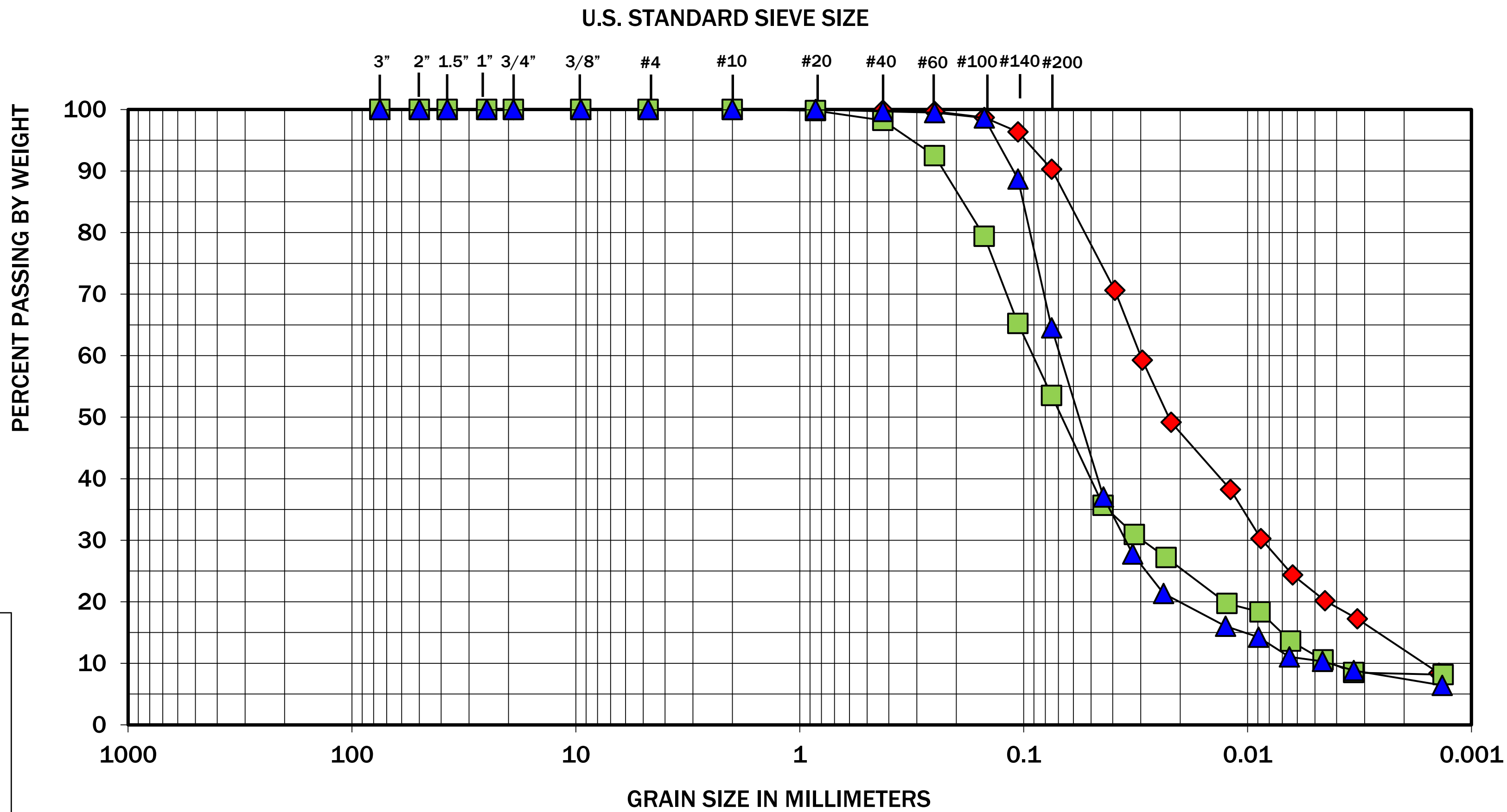
Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
40	0							AC	Approximately 5 inches of asphalt concrete pavement			
								ML	Brown-gray with iron oxide staining silt with sand (stiff, moist) (alluvium)			
			18	11		1A 1B		SM	Brown-gray with mottling and iron oxide staining, silty fine sand (medium dense, moist)			
35	5		18	9		2 SA/HA		ML	Brown-gray sandy silt (stiff, moist)	15	65	
				8		3			Grades to medium stiff			Interbedded silt and sand to 10 feet
30	10		18	15		4		SM	Brown-gray silty fine sand (medium dense, moist)			
			18	21		5 %F				24	48	

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on DEM.

Log of Boring GEI-5

GeoEngineers

Project: Coastal Pacific Food Distributors Freezer Expansion
Project Location: Puyallup, Washington
Project Number: 27044-001-00



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-1	10	32	Silt (ML)
■	B-2	7.5	10	Sandy silt (ML)
▲	B-5	5	15	Sandy silt (ML)

AASHTO ACCREDITED

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

GEOENGINEERS

Coastal Pacific Food Distributors – Freezer Expansion
Puyallup, Washington

Sieve Analysis Results

Figure A-7

APPENDIX B
2004 Report Explorations

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

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

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

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

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

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Date(s) Drilled	08/06/04	Logged By	TCK	Checked By	TAD
Drilling Contractor	Holt Drilling	Drilling Method	Hollow-Stem Auger (HSA)	Sampling Methods	2.4-inch I.D. Split Spoon Ring Sampler
Auger/Bit Data	4-inch I.D. Continuous Flight	Hammer Data	300 (lb) hammer/ 30 (in) drop	Drilling Equipment	B-59 Foremost Mobile Drill Rig
Total Depth (ft)	30	Surface Elevation (ft)	39	Groundwater Elevation	22.5
Vertical Datum		Datum/ System	N prop corner = Elevation 39.3	Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0							SP	Brown fine to medium sand with occasional gravel (medium dense, moist) (fill)			
							SP	Brown fine to medium sand (medium dense, moist)			
		18	21	1			SP	Gray fine to medium sand, trace silt (medium dense, moist)	8	91	
5											
		14	9	2			SM	Gray silty sand (medium dense, moist)			
10											
		12	12	3			ML	Gray sandy silt (medium stiff, moist to wet)	27		GS; %F=53
15					▼						
		14	15	4			SW-SM	Black-gray fine to coarse sand with silt, occasional wood chips and gravel (medium dense, wet)			
20											
		14	17	5			SP	Gray fine to medium sand, trace silt (medium dense, wet)			
25											
		6	24	6							
30											
35											

Note: See Figure A-1 for explanation of symbols.

V6 GTBORING P:\11\11624001\00\FINALS\1162400100TP B.GPJ GEIV6 1.GDT 8/18/04

LOG OF BORING B-1

GeoEngineers

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-2
Sheet 1 of 1

Date(s) Drilled	08/06/04	Logged By	TCK	Checked By	TAD
Drilling Contractor	Holt Drilling	Drilling Method	Hollow-Stem Auger (HSA)	Sampling Methods	2.4-inch I.D. Split Spoon Ring Sampler
Auger/Bit Data	4-inch I.D. Continuous Flight	Hammer Data	300 (lb) hammer/ 30 (in) drop	Drilling Equipment	B-59 Foremost Mobile Drill Rig
Total Depth (ft)	50	Surface Elevation (ft)	38.4	Groundwater Elevation	22.4
Vertical Datum		Datum/ System	N prop corner = Elevation 39.3	Easting(x): Northing(y):	

Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0					SP-SM	Brown fine to medium sand with silt (loose, dry to moist)			
					ML	Brown-red mottled silt (medium stiff, moist)			
	18	10	1		SM	Brown-red mottled silty sand (loose, moist)	10	88	
5									
	18	13	2		SP	Black-gray fine to medium sand (medium dense, moist)	4		GS; %F=2
10									
	12	13	3		SW	Red-brown fine to coarse sand, trace silt, occasional gravel (medium dense, moist to wet)			
15				▼					
	6	25	4						
20									
	12	29	5		SP	Gray-black fine to medium sand, trace silt (medium dense, wet)			
25									
	14	23	6						
30									
	18	21	7		SM	Gray silty sand (medium dense, wet)			
35									

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-2

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-3
Sheet 1 of 2

Depth feet	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
35												
								ML	Gray silt (very stiff, wet)			
		16	26		8			SM	Gray silty sand (medium dense, wet)			
40												
		18	7		9			SM	Gray silty fine to coarse sand (loose, wet)	20		%F=26
45												
		14	4		10							
50												

55

60

65

70

75

LOG OF BORING B-2 (continued)

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-3
Sheet 2 of 2

V6_GTBORING P:\11\11624001\00\FINALS\1162400100TP_B.GPJ GEIV6_1.GDT 8/18/04

Date Excavated: 08/06/04

Equipment: Case 580 Super L

Logged by: TAD

Surface Elevation (ft): 38

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				GM	Gray silty gravel with sand (medium dense, dry) (fill)		
					ML	Brown silt (medium stiff, moist)		
35					SM	Gray silty fine sand (loose to medium dense, moist)		
	5							
			1				7	GS; %F=16
30								

Test pit completed at 8 feet on 08/06/04
No groundwater seepage observed
Moderate caving observed at 4 to 6 feet
Disturbed soil sample obtained at 6 feet

Depth scale: 0, 5, 10, 15, 20, 25 feet. Elevation scale: 35, 30, 25, 20, 15.

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

LOG OF TEST PIT TP-1

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-4
Sheet 1 of 1

V6_GTTPIT P:\11\11624001\00\FINALS\1162400100TP_B.GPJ GEIV6_1.GDT 8/18/04

Date Excavated: 08/06/04

Equipment: Case 580 Super L

Logged by: TAD

Surface Elevation (ft): 37

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				ML	Brown silt (medium stiff, dry)		
					ML	Gray-brown silt with fine sand (medium stiff, moist)		
35			1				22	OC=1.1%; %F=64
					SM	Gray-brown silty fine sand (medium dense, moist)		
	5							
30								
					ML	Gray-brown silt with wood fragments (medium stiff, moist)		
			2					pH
	10							
					SM	Gray silty fine sand with wood fragments (medium dense, moist)		
25						Test pit completed at 12 feet on 08/06/04 No groundwater seepage observed No caving observed Disturbed soil samples obtained at 3 and 9 feet		
	15							
20								
	20							
15								
	25							

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

LOG OF TEST PIT TP-2

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-5
Sheet 1 of 1

Date Excavated: 08/06/04

Equipment: Case 580 Super L

Logged by: TAD

Surface Elevation (ft): 37.9

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				ML	Brown silt (medium stiff, dry to moist)		
			1				37	OC=3%; pH
35								
					ML	Brown-gray silt with sand (medium stiff, moist)		
	5		2		SP	Reddish-brown fine sand (medium dense, moist)	26	AL; resistivity
					SP	Gray fine sand (medium dense, moist)		
30								
	10							
			3					

Test pit completed at 12 feet on 08/06/04
No groundwater seepage observed
No caving observed
Disturbed soil samples obtained at 2, 5 and 11 feet

Elevation scale: 25, 20, 15; Depth scale: 15, 20, 25

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

LOG OF TEST PIT TP-3

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-6
Sheet 1 of 1

Date Excavated: 08/06/04

Equipment: Case 580 Super L

Logged by: TAD

Surface Elevation (ft): 38.6

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				ML	Brown silt (medium stiff, dry to moist)		
						Occasional wood fragments at 2 feet		
35								
	5		1		SP	Gray fine sand (loose, moist)	4	Resistivity; %F=3.5
30								

Test pit completed at 9 feet on 08/06/04
No groundwater seepage observed
Moderate caving observed at 4.5 and 7 feet
Disturbed soil sample obtained at 5 feet

Elevation scale: 25, 20, 15. Depth scale: 10, 15, 20, 25.

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

LOG OF TEST PIT TP-4

GEOENGINEERS

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-7
Sheet 1 of 1

Date Excavated: 08/06/04

Equipment: Case 580 Super L

Logged by: TAD

Surface Elevation (ft): 38.6

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				ML	Brown sandy silt (medium stiff, dry to moist)		
35			1				24	%F=89
	5				SP	Brown to gray-brown fine sand (medium dense, moist)		
			2			Grades to reddish-brown		
30					SP	Gray fine to medium sand (medium dense, moist)		
	10				SP	Gray fine sand (medium dense, moist)		

Test pit completed at 10.5 feet on 08/06/04
No groundwater seepage observed
No caving observed
Disturbed soil samples obtained at 4 and 6.5 feet

Elevation scale: 25, 20, 15 — Depth scale: 15, 20, 25

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

V6_GTTPIT P:\11\11624001\00\FINALS\1162400100TP_B.GPJ GEIV6_1.GDT 8/18/04

LOG OF TEST PIT TP-5

GeoEngineers

Project: Schwan Food Company
Project Location: Puyallup, Washington
Project Number: 11624-001-00

Figure: A-8
Sheet 1 of 1

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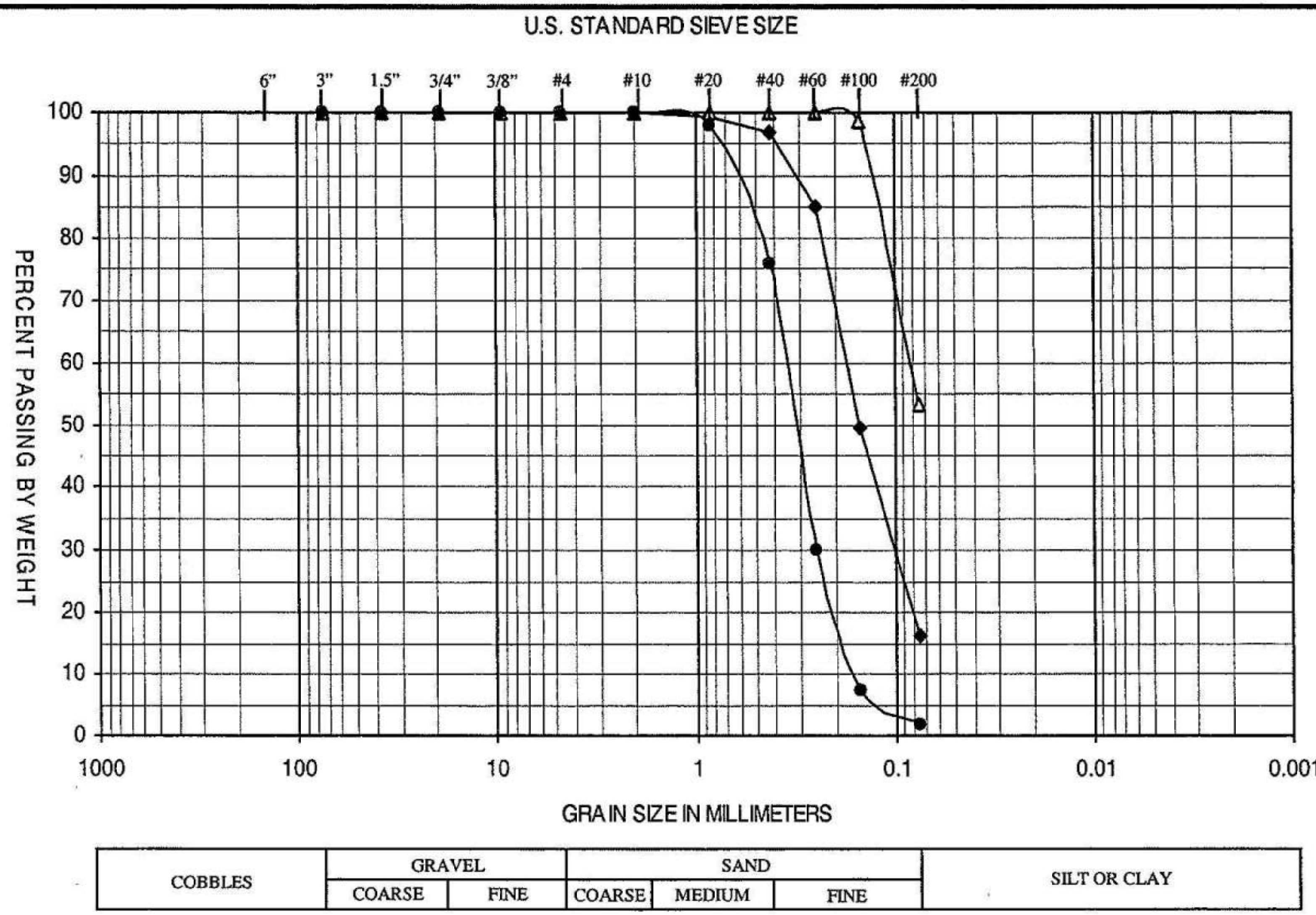


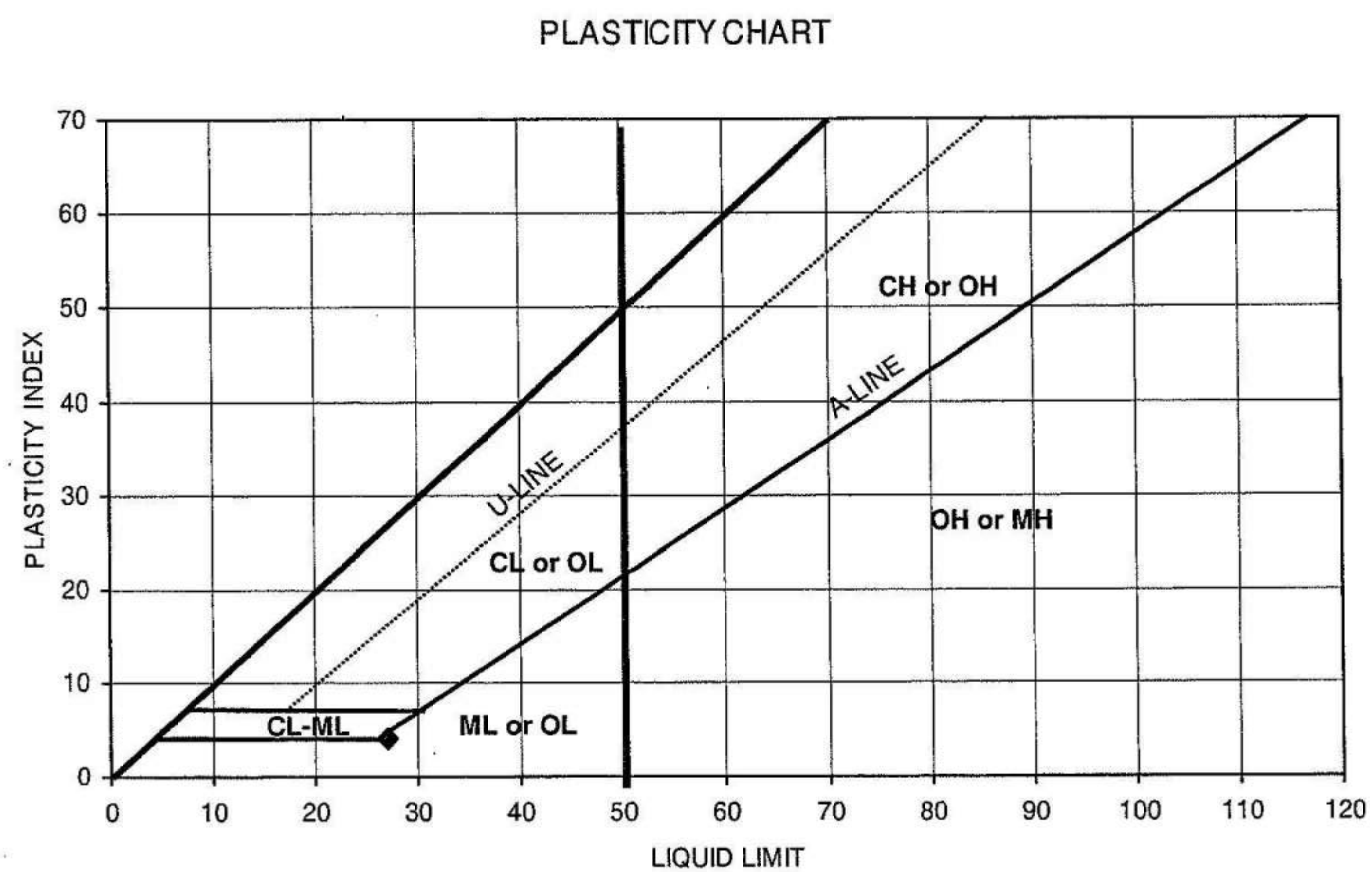
SYMBOL	BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION
◆	TP-1	6.0	Silty sand (SM)
△	B-1	14.5	Sandy silt (ML)
●	B-2	9.5	Fine to medium sand (SP)

GEOENGINEERS

SIEVE ANALYSIS RESULTS

FIGURE A-9

TACO:11\11624001\00\Finals\1162400100FigureA10.ppt TAD:CTS :cts 081804



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	TP-3	5.0'	26	27	4	Brown silt w/sand (ML)

GeoEngineers

ATTERBERG LIMITS TEST RESULTS

FIGURE A-10

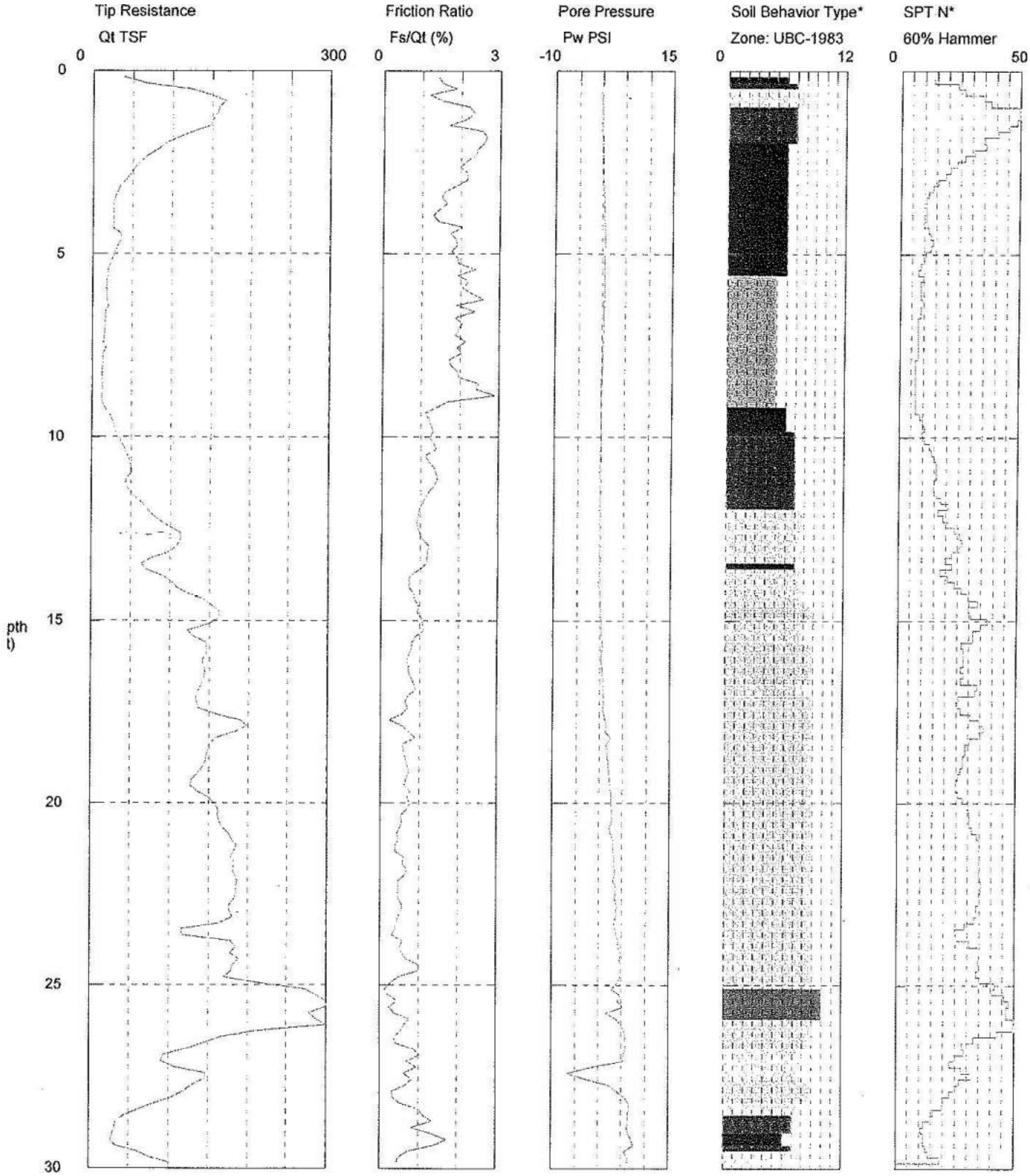
ATTACHMENT A

CPT LOGS BY NORTHWEST CONE, INC

Geo-Engineers

Operator: Nowak
Sounding: CPT-1
Cone Used: DSG0880

CPT Date/Time: 8/6/2004 8:41:51 AM
Location: Schwan Food Company
Job Number: 1162400100



Maximum Depth = 30.18 feet

Depth Increment = 0.164 feet

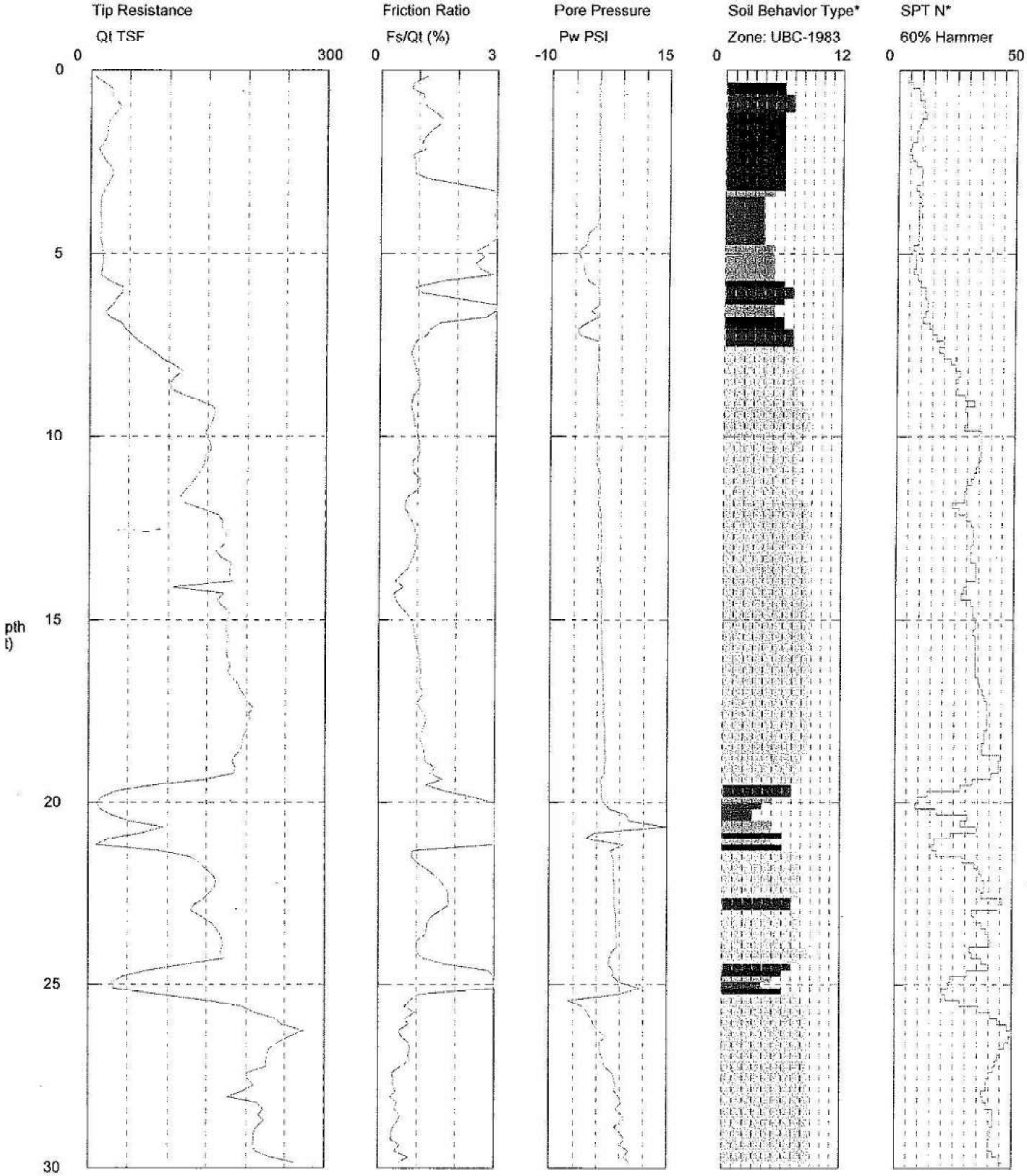
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

behavior type and SPT based on data from UBC-1983

Geo-Engineers

Operator: Nowak
Sounding: CPT-2
Cone Used: DSG0880

CPT Date/Time: 8/6/2004 9:23:41 AM
Location: Schwan Food Company
Job Number: 1162400100



Maximum Depth = 30.51 feet

Depth Increment = 0.164 feet

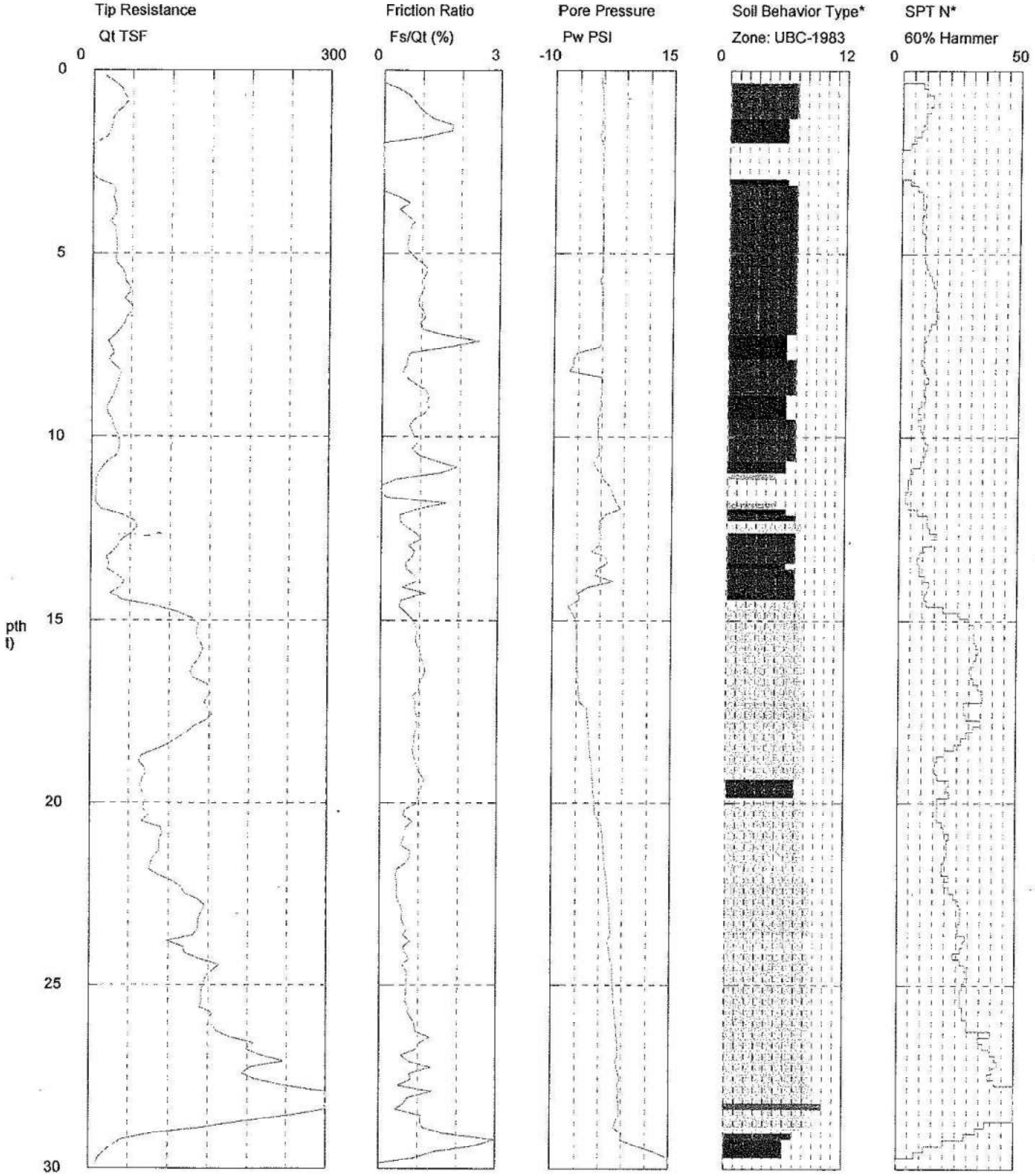
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

behavior type and SPT based on data from UBC-1983

Geo-Engineers

Operator: Nowak
Sounding: CPT-3
Cone Used: DSG0880

CPT Date/Time: 8/6/2004 10:01:27 AM
Location: Schwan Food Company
Job Number: 1162400100



Maximum Depth = 30.02 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

behavior type and SPT based on data from UBC-1983

APPENDIX C
2024 ConeTec, Inc. Report

PRESENTATION OF SITE INVESTIGATION RESULTS

CPFD-Freezer Expansion

Prepared for:

GeoEngineers Inc.

ConeTec Job No: 24-59-27281

Project Start Date: 2024-02-26
Project End Date: 2024-02-26
Release Date: 2024-03-05

Report Prepared by:

ConeTec, Inc.

1237 S Director Street, Seattle, WA 98108

Tel: (253) 397-4861

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

CONETEC

ABOUT THIS REPORT

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

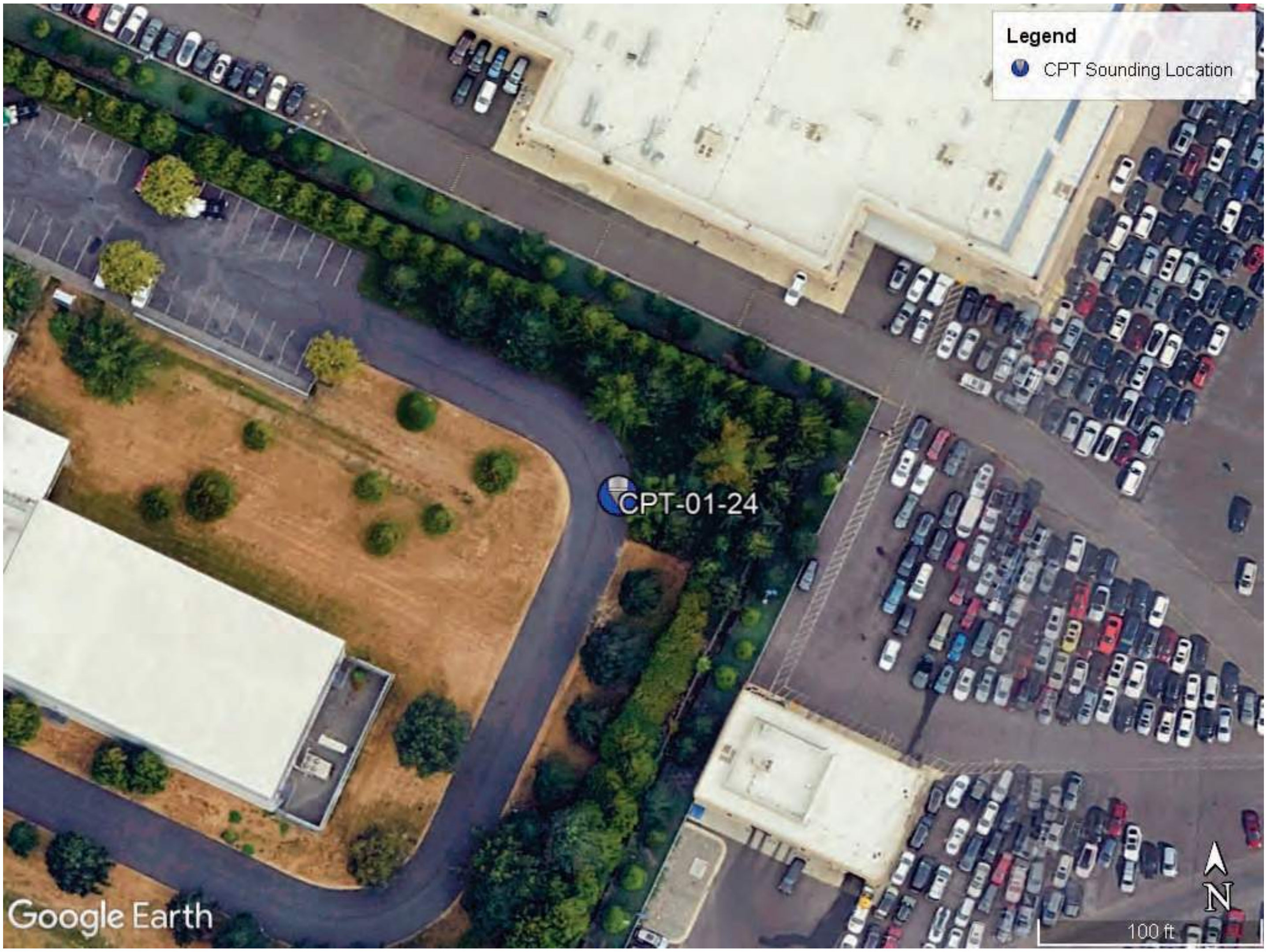
Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report. Please refer to the list of attached documents following the text of this report. A site map, test summaries, and test plots are all included in the body of this report.

Project	
Client	GeoEngineers Inc.
Project	CPFD-Freezer Expansion
ConeTec Project Number	24-59-27281
Test Types	SCPTu
Additional Comments	None

Contents

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

SITE MAP



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

ConeTec Job Number: 24-59-27281

Client: GeoEngineers Inc.

Project: CPFD-Freezer Expansion

Release Date: 2024-03-05

CONETEC

LIMITATIONS

3rd Party Disclaimer

The “Report” refers to this report titled: CPFD-Freezer Expansion

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.

The “Report” refers to this report titled: CPFD-Freezer Expansion

ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

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

Closure

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

Report prepared by: Alex Leibold
Report Reviewed by: Jesse Martinez

PROJECT INFORMATION

Rigs		
Description	**Deployment System**	**Test Type**
C02-020 CPT Truck Rig	Twin mounted cylinders	SCPTu

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

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC859:T1500F15U35	859	15	225	1500	15	35

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.

Calculated Geotechnical Parameters	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Methodology Statements and Data File Formats

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

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

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

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

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

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

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

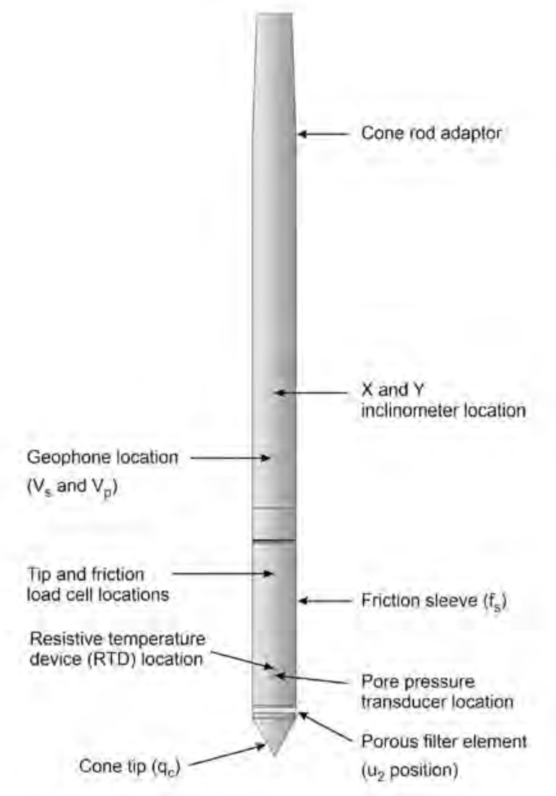


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

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

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

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

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

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

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

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

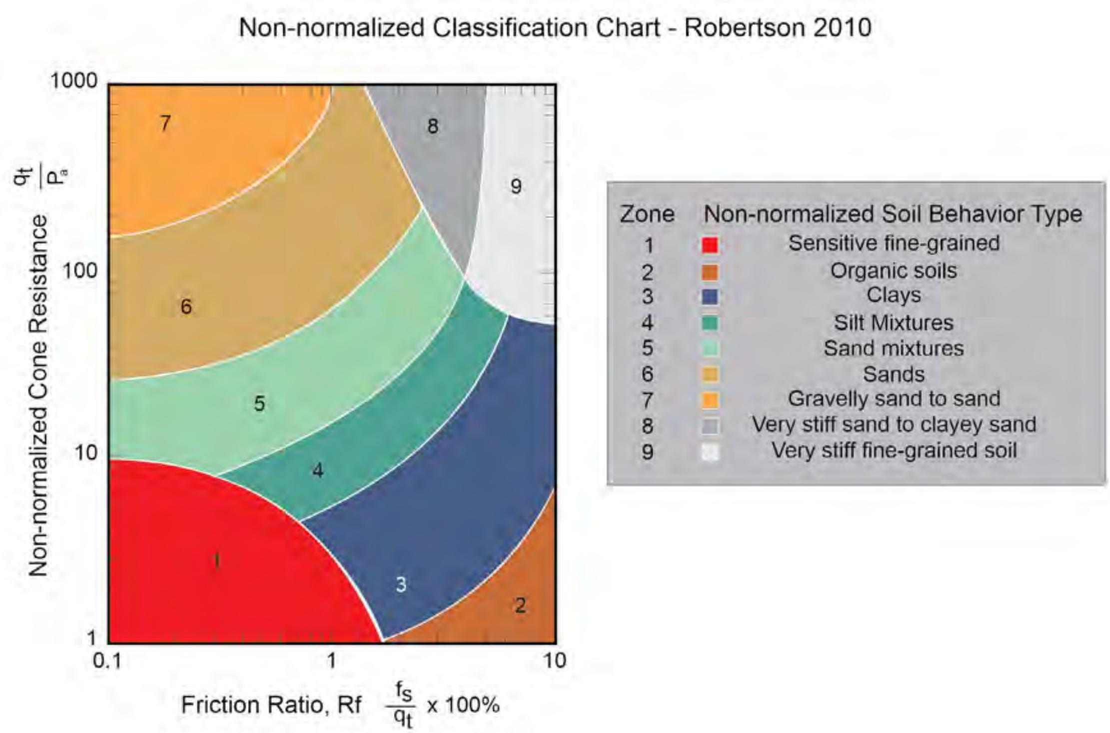


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

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

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

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

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

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

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

REFERENCES

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

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

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

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

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

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

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

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

PORE PRESSURE DISSIPATION TEST

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

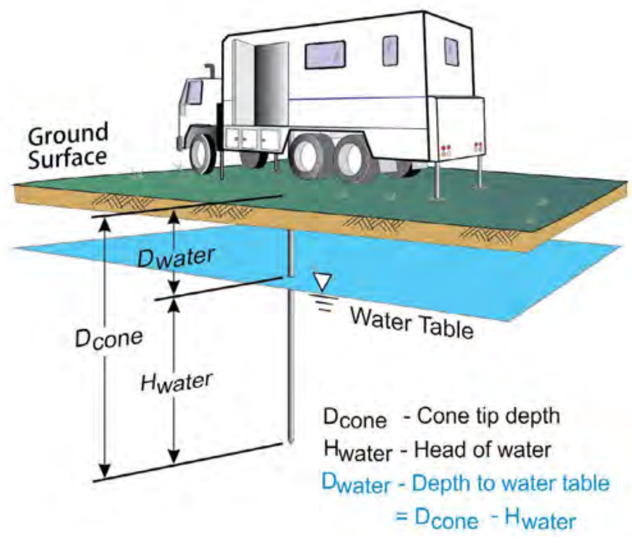


Figure PPD-1. Pore pressure dissipation test setup

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

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

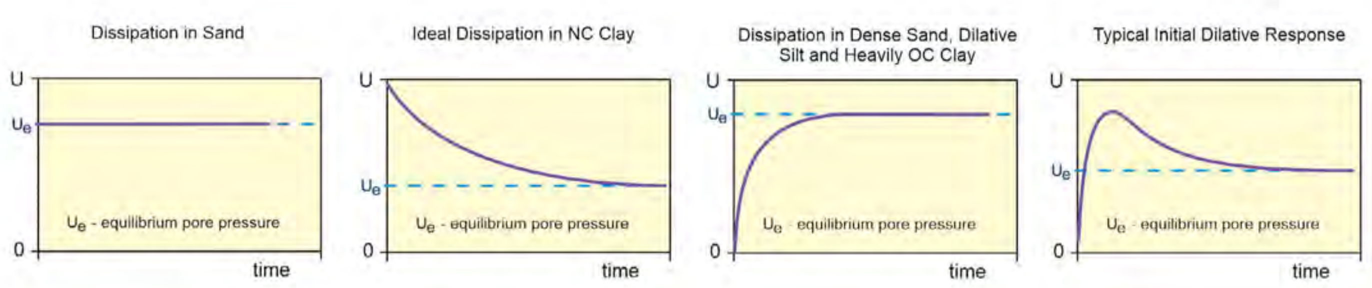


Figure PPD-2. Pore pressure dissipation curve examples

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

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

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

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

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

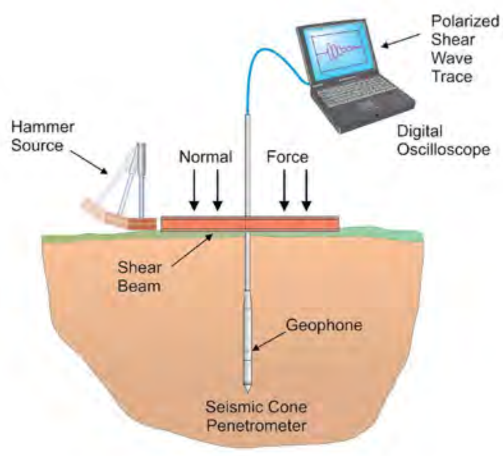


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

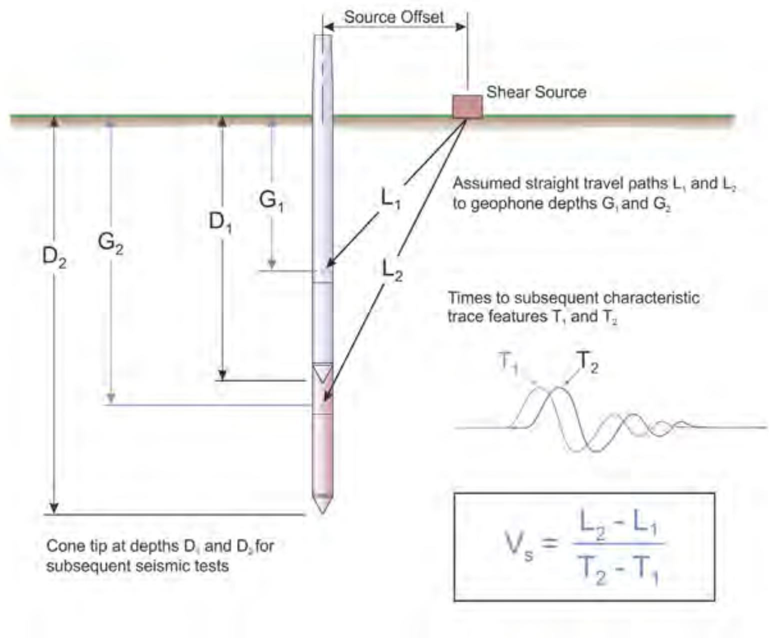


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

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

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

Average shear wave velocity, $\bar{v}_S$ is also referenced to V_S100 or V_S30.

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

REFERENCES

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

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

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

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

CPT Data Files (COR Extension)

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

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

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

Header Lines

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

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

Data Records

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

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

End of Data Marker

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

Units Information

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

CPT Data Files (XLS Extension)

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

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

CPT Dissipation Files (XLS Extension)

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

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

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

Data Records

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

Cone Type Designations

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

refers to the Cone ID number

**Outer Cylindrical Area

REPORT APPENDICES

The appendices listed below are included in the report:

- **Cone Penetration Test (CPTu) Summary and Standard CPTu Plots**
- **Normalized Cone Penetration Test Plots**
- **Advanced Cone Penetration Test Plots**
- **Soil Behavior Type (SBT) Scatter Plots**
- **Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots**
- **Seismic Cone Penetration Test (SCPTu) Tabular Results**
- **SCPTu Plots**
- **SCPTu Velocity Wave Traces**
- **Description of Methods for Calculated CPTu Geotechnical Parameters**
- **Piezocone Calibration Sheets**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No:	24-59-27281
Client:	GeoEngineers Inc.
Project:	CPFD-Freezer Expansion
Start Date:	2024-02-26
End Date:	2024-02-26

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Seismic Intervals	Latitude[2]	Longitude[2]	Refer to Notation Number
CPT-01-24	24-59-27281_SP01	2024-02-26	859:T1500F15U35	15	12.3	192.25	58	47.20650	-122.29888	
Totals	1 Sounding					192.25 ft	58			

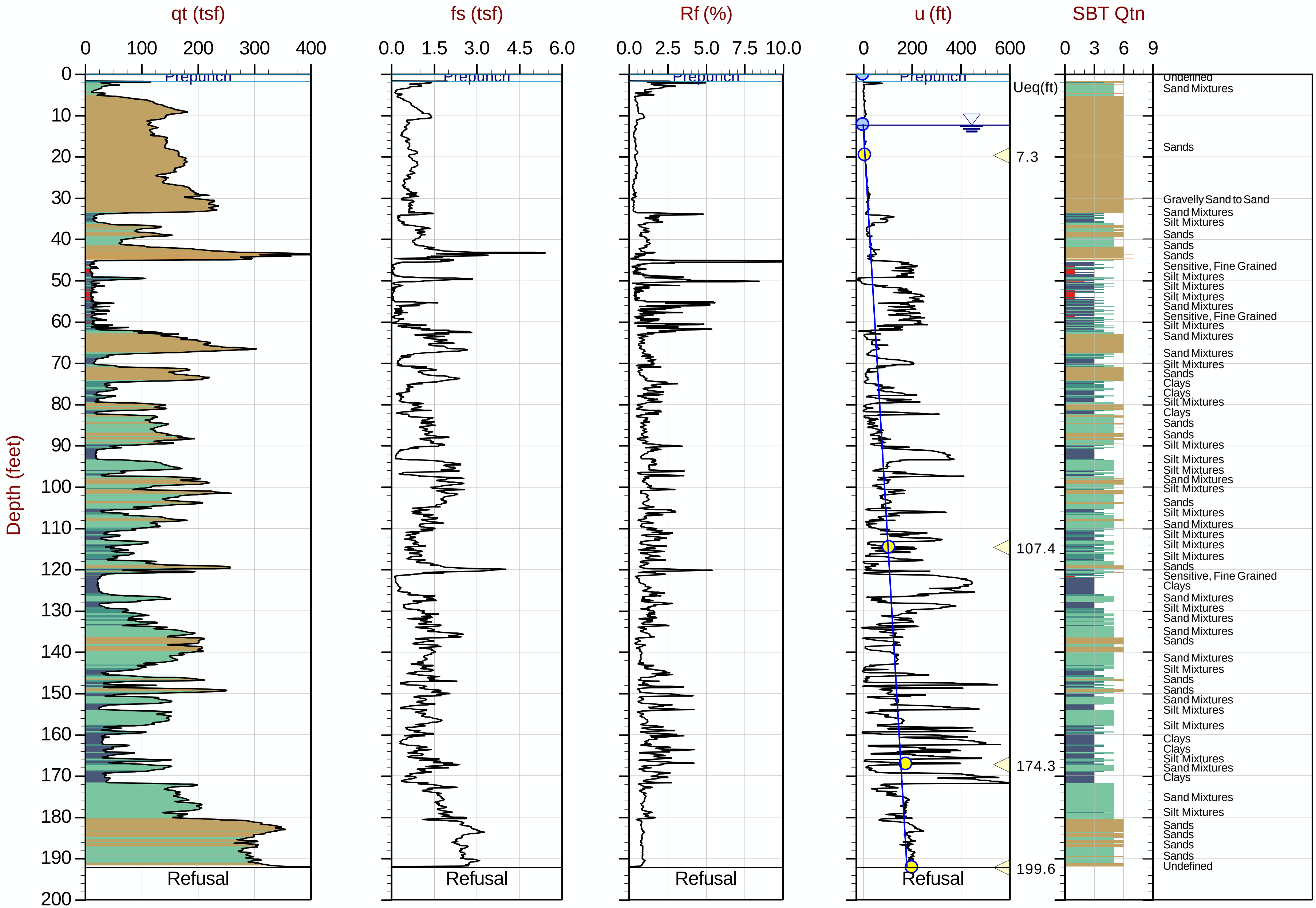
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).

CONETEC

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35



Max Depth: 58.600 m / 192.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27281_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.20650 Long: -122.29888

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

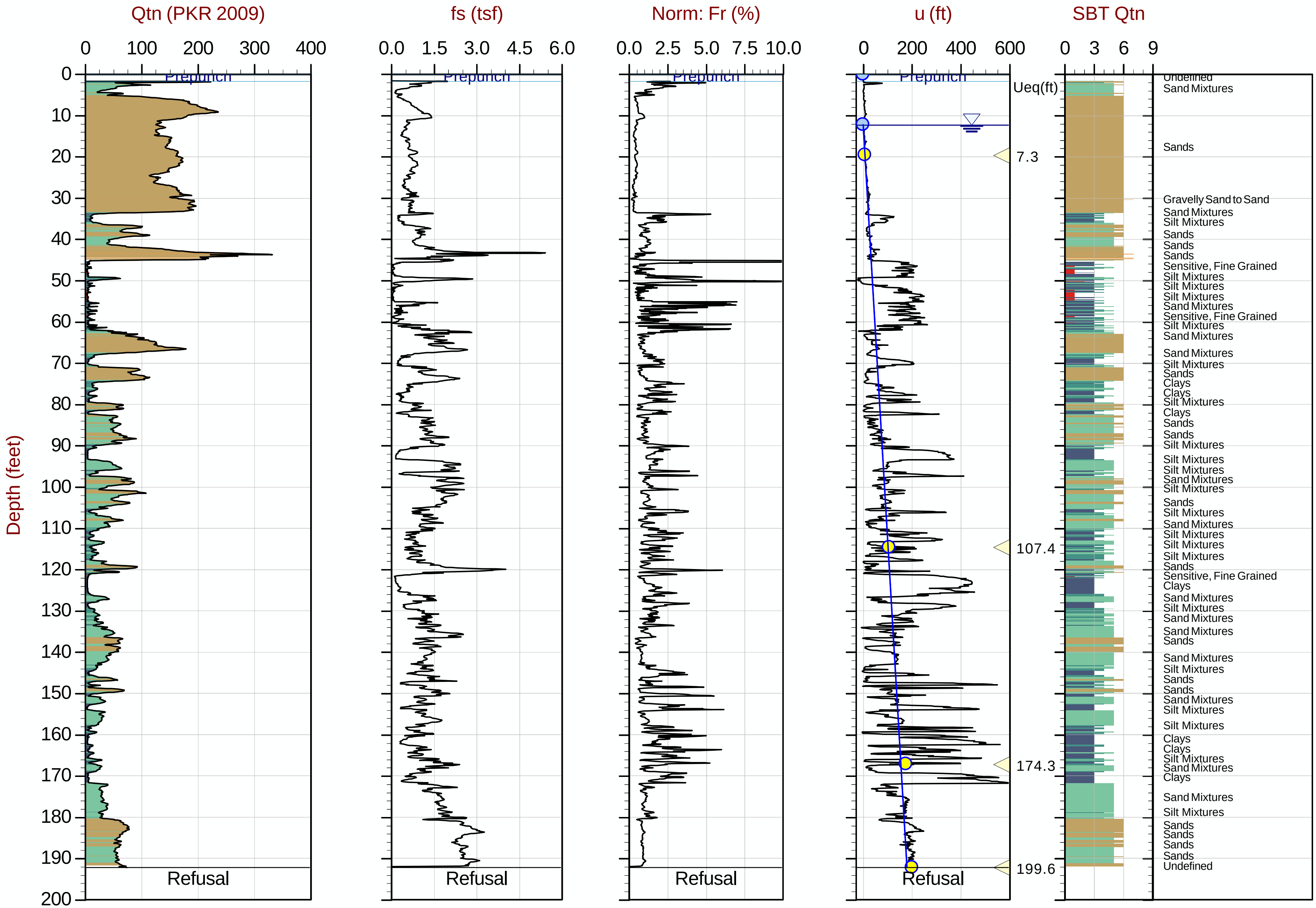
Normalized Cone Penetration Test Plots

CONETEC

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35



Max Depth: 58.600 m / 192.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27281_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.20650 Long: -122.29888

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Hydrostatic Line

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

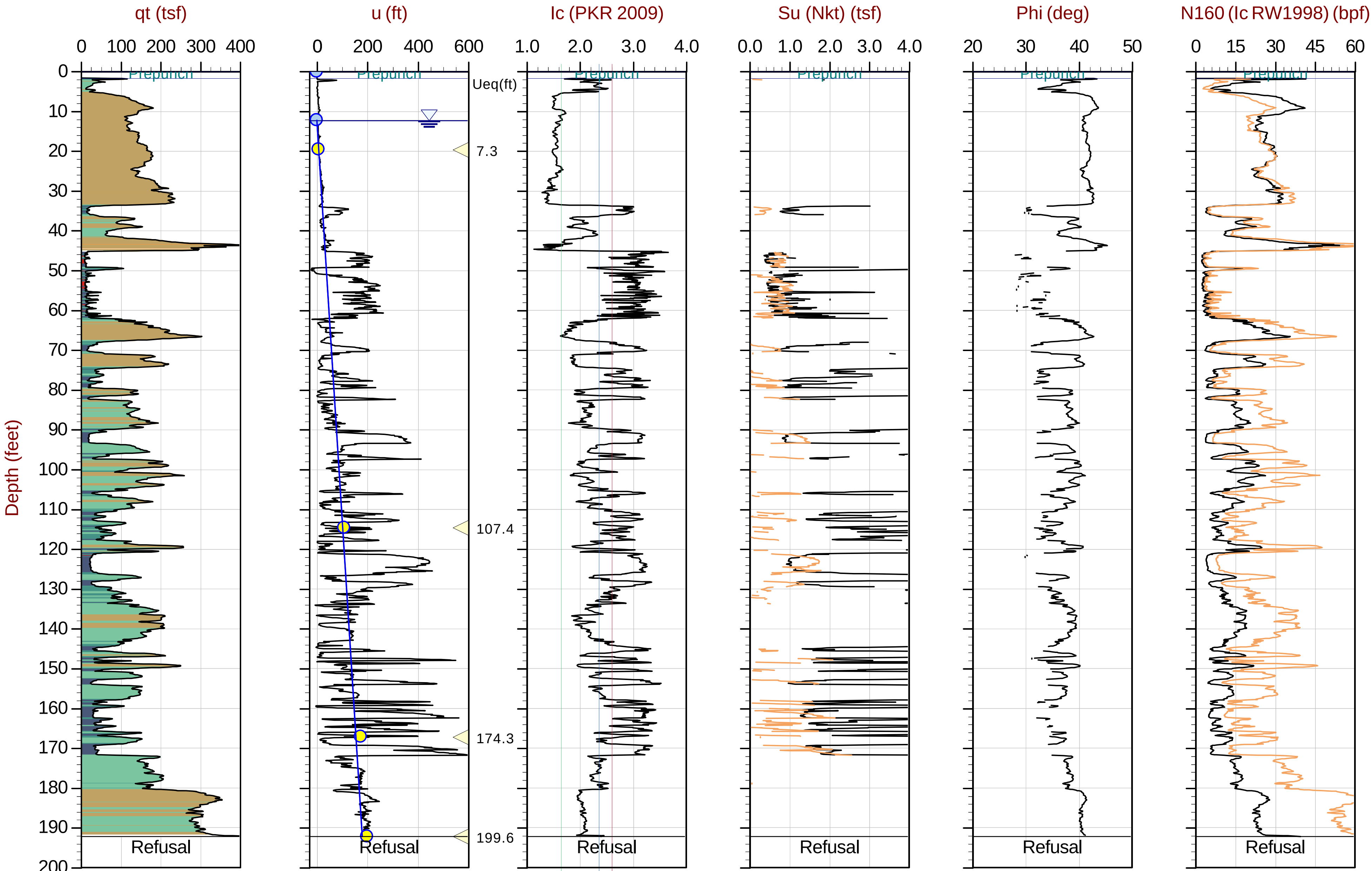
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

CONETEC

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35



Max Depth: 58.600 m / 192.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27281_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.20650 Long: -122.29888

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

Soil Behavior Type (SBT) Scatter Plots

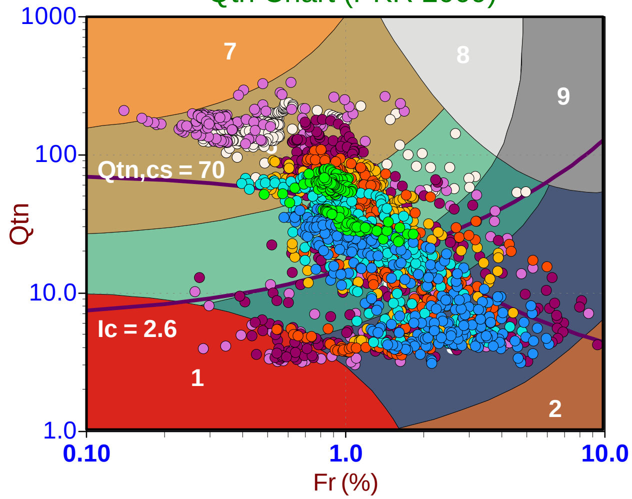
CONETEC

GeoEngineers

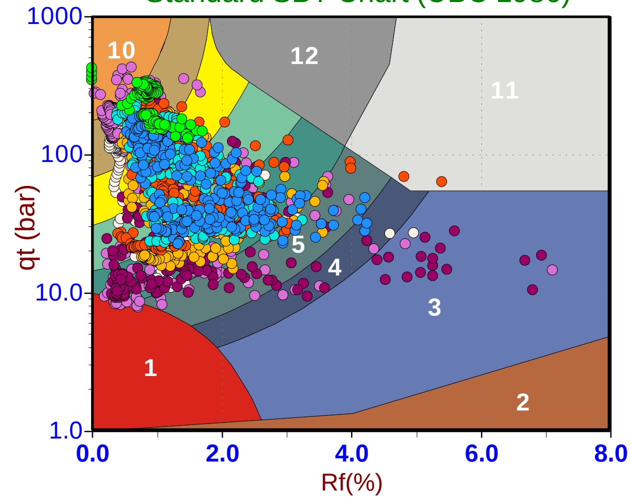
Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35

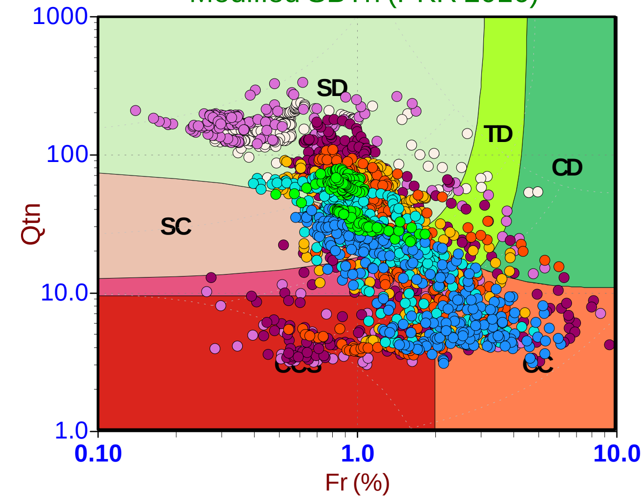
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 25.0 ft
- >25.0 to 50.0 ft
- >50.0 to 75.0 ft
- >75.0 to 100.0 ft
- >100.0 to 125.0 ft
- >125.0 to 150.0 ft
- >150.0 to 175.0 ft
- >175.0 to 200.0 ft
- >200.0 to 225.0 ft
- >225.0 to 250.0 ft
- >250.0 ft

Legend

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

Legend

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

Legend

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

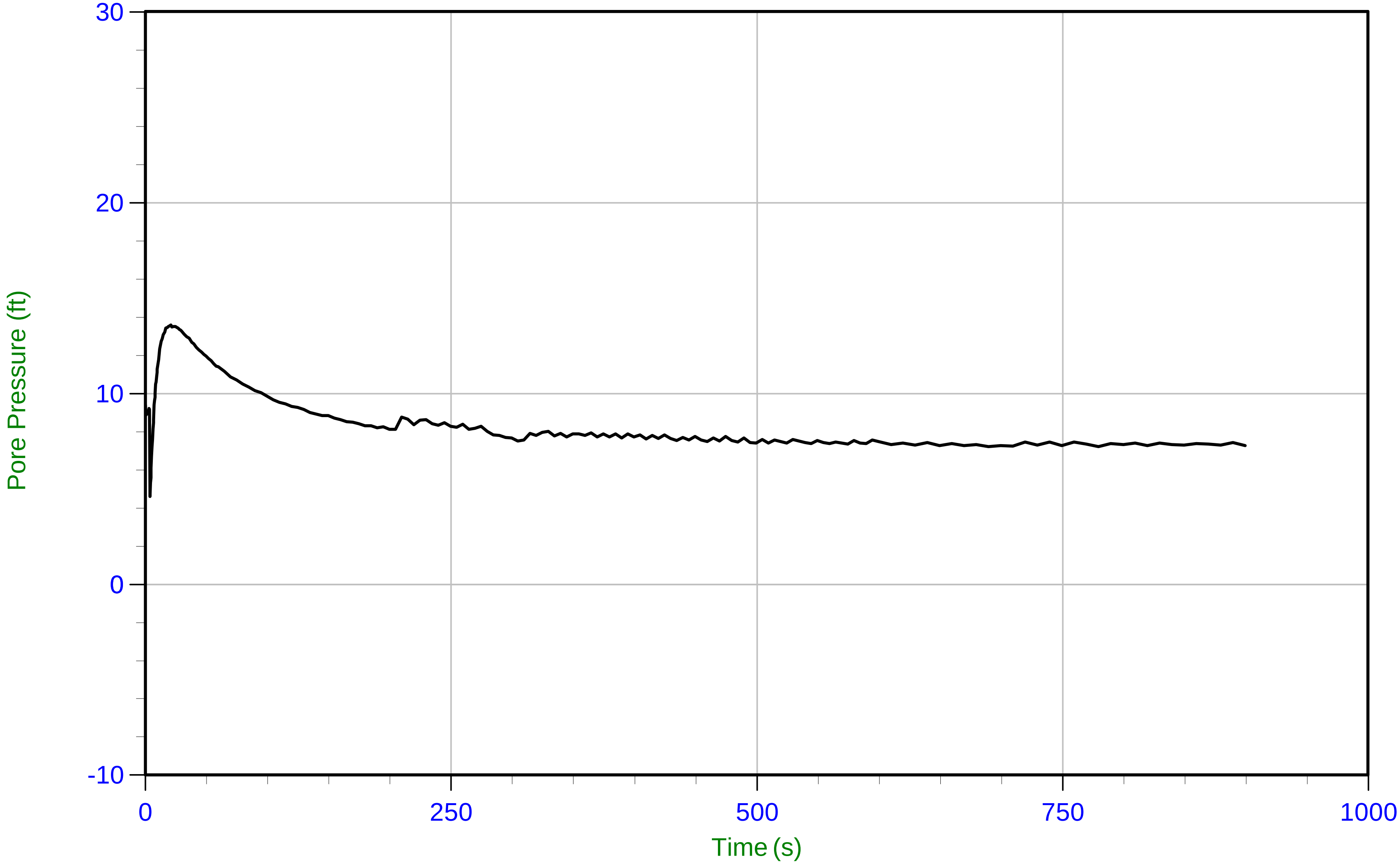
Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots

Job No: 24-59-27281
Client: GeoEngineers Inc.
Project: CPFD-Freezer Expansion
Start Date: 2024-02-26
End Date: 2024-02-26

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)	Refer to Notation Number
CPT-01-24	24-59-27281_SP01	15	900	19.60	7.3	12.3	
CPT-01-24	24-59-27281_SP01	15	1160	114.66	107.5	7.2	
CPT-01-24	24-59-27281_SP01	15	585	167.24	174.4	-7.2	
CPT-01-24	24-59-27281_SP01	15	480	192.25	199.7	-7.4	
Totals			52 min				

GeoEngineers	Job No: 24-59-27281	Sounding: CPT-01-24	
	Date: 2024-02-26 08:58	Cone: 859:T1500F15U35	Area=15 cm²
	Site: CPFD-Freezer Expansion		



Trace Summary:

Filename: 24-59-27281_SP01.PPR2
Depth: 5.975 m / 19.603 ft
Duration: 900.0 s

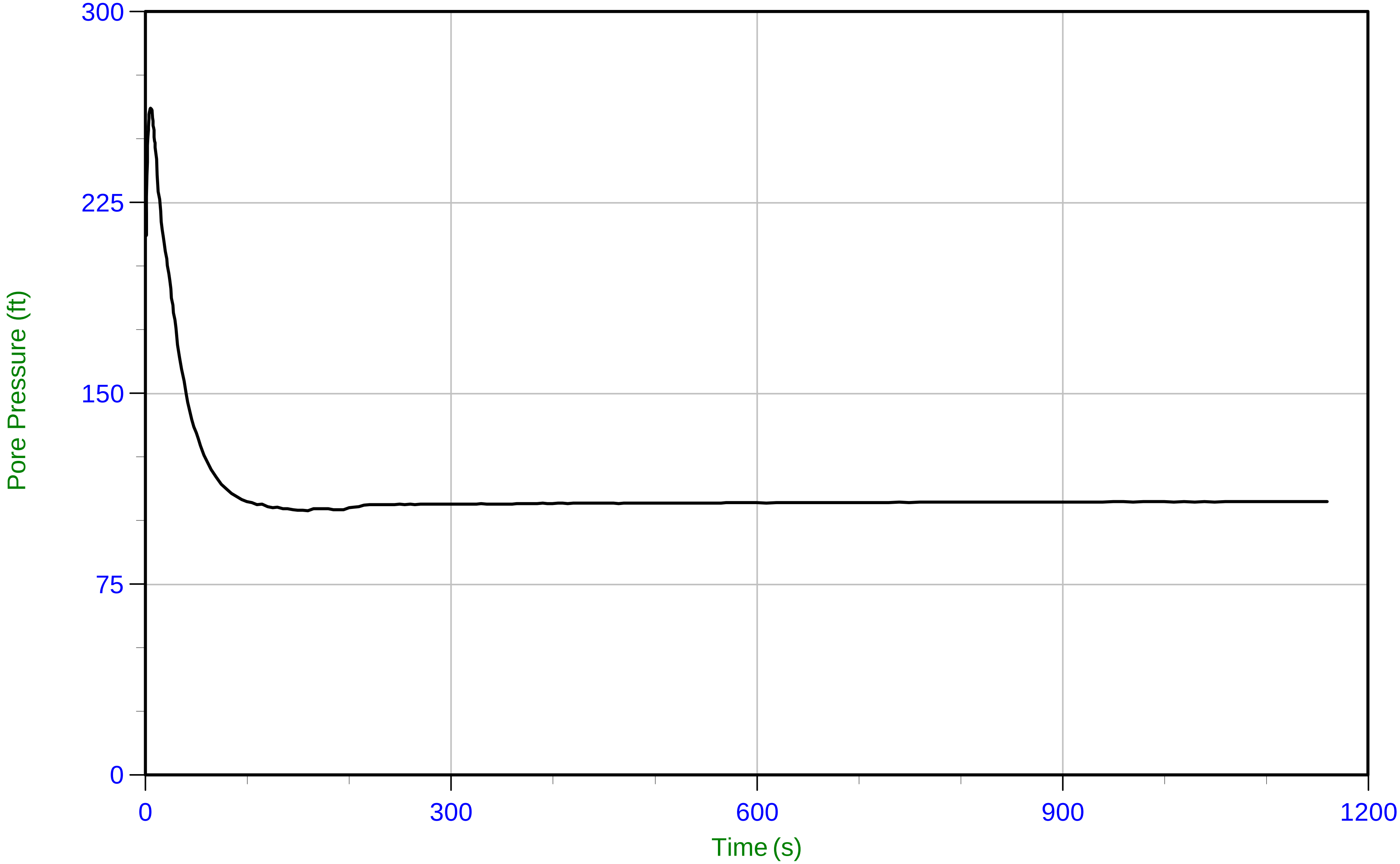
u Min: 4.6 ft
u Max: 13.6 ft
u Final: 7.3 ft

WT: 3.749 m / 12.300 ft
Ueq: 7.3 ft

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27281_SP01.PPR2
Depth: 34.950 m / 114.664 ft
Duration: 1160.0 s

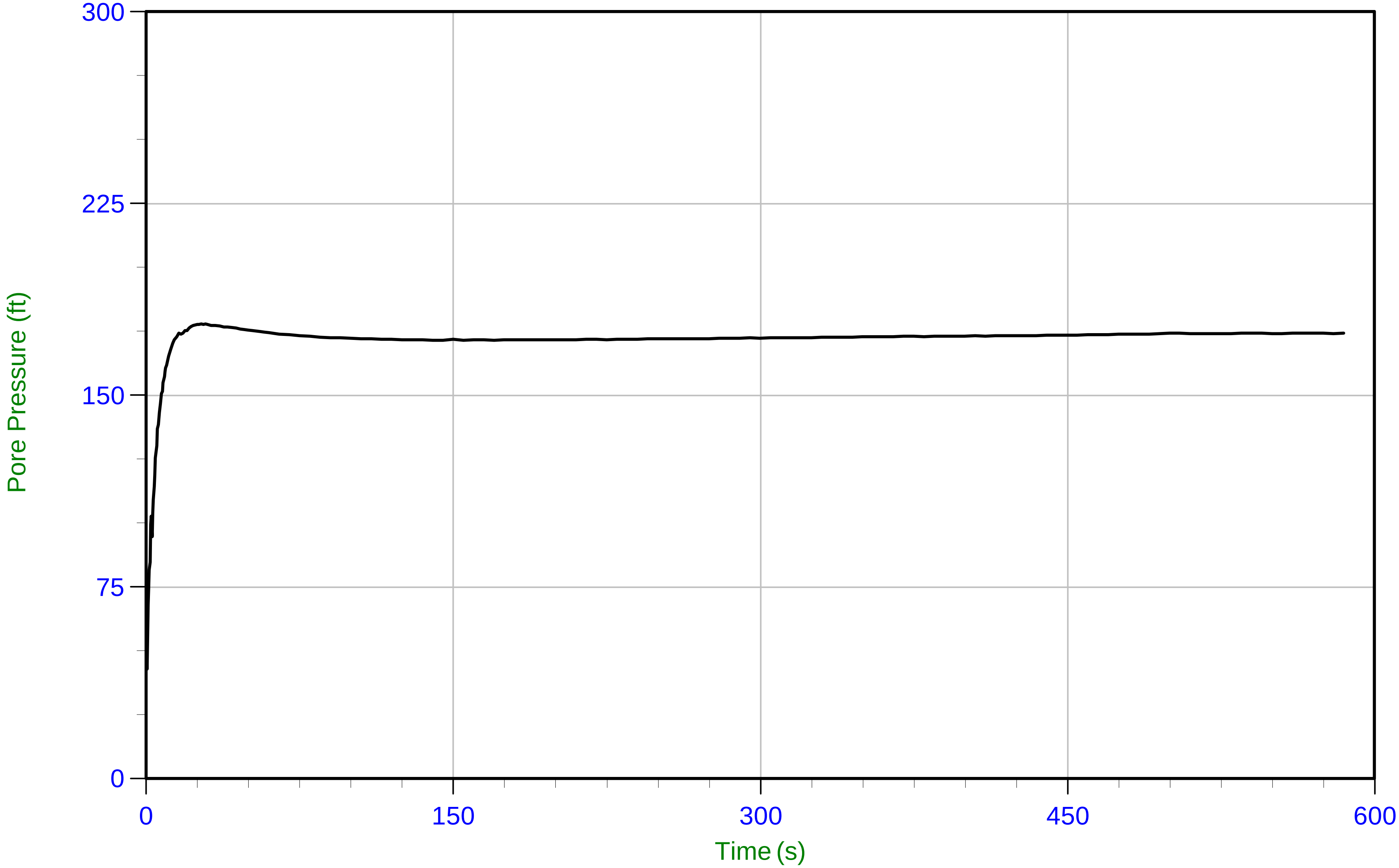
u Min: 104.0 ft
u Max: 262.0 ft
u Final: 107.4 ft

WT: 2.192 m / 7.192 ft
Ueq: 107.5 ft

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27281_SP01.PPR2
Depth: 50.975 m / 167.239 ft
Duration: 585.0 s

u Min: 42.9 ft
u Max: 177.9 ft
u Final: 174.3 ft

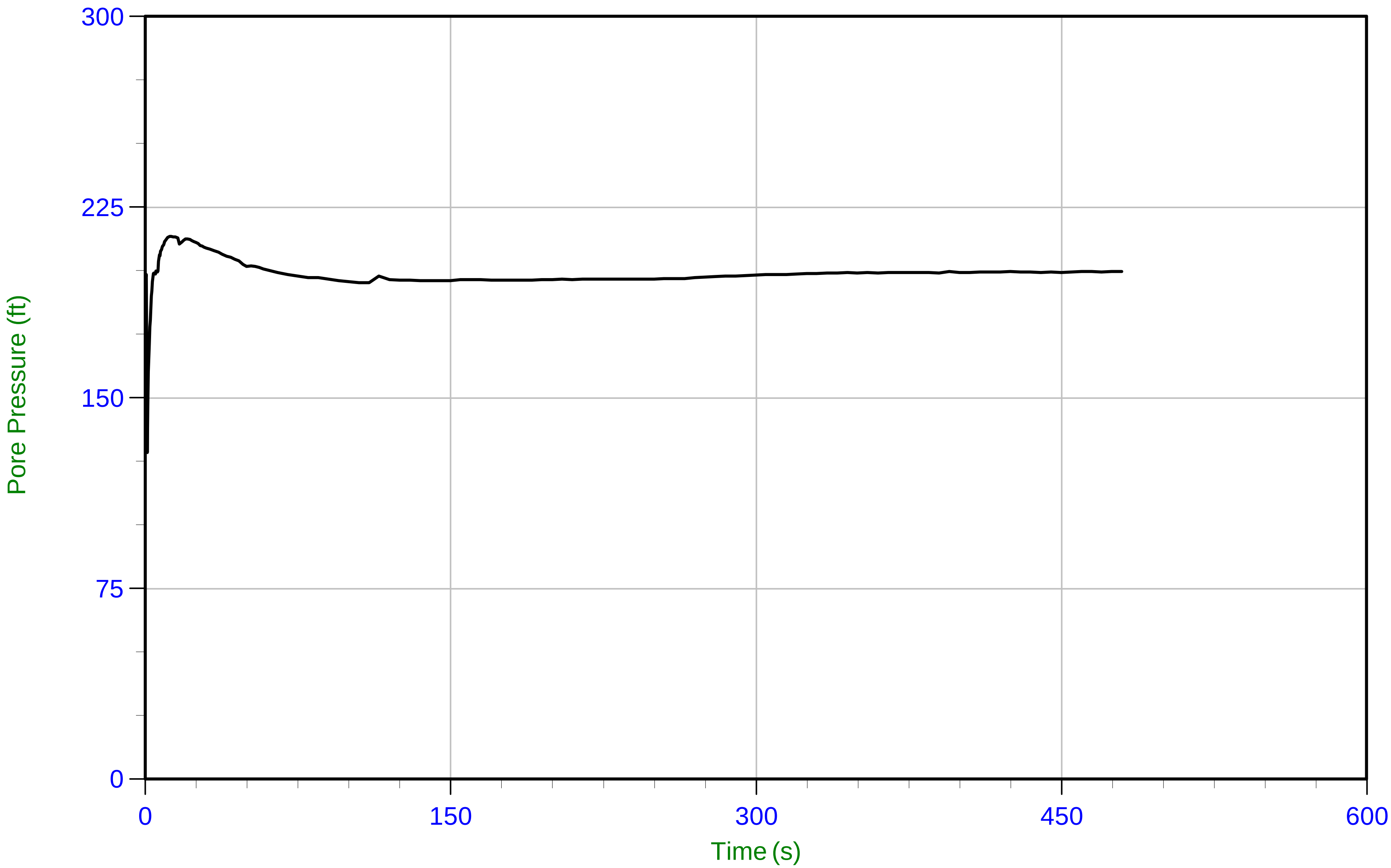
WT: -2.181 m / -7.155 ft
Ueq: 174.4 ft

CONETEC

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27281_SP01.PPR2
Depth: 58.600 m / 192.255 ft
Duration: 480.0 s

u Min: 128.6 ft
u Max: 213.5 ft
u Final: 199.6 ft

WT: -2.266 m / -7.434 ft
Ueq: 199.7 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

Job No: 24-59-27281
Client: GeoEngineers
Project: CPFD-Freezer Expansion
Sounding ID: CPT-01-24
Date: 2024-02-26

Seismic Source: Beam
Seismic Offset (ft): 8.86
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)
3.22	2.56	9.22			
6.40	5.74	10.56	1.34	3.59	372
9.58	8.92	12.58	2.02	3.66	551
12.96	12.30	15.16	2.59	5.63	460
16.24	15.58	17.93	2.77	6.35	436
19.62	18.96	20.93	3.00	5.31	566
22.80	22.15	23.85	2.92	5.92	493
26.02	25.36	26.86	3.01	5.70	528
29.36	28.71	30.04	3.18	5.58	570
32.74	32.09	33.29	3.24	5.20	624
35.93	35.27	36.37	3.08	5.75	536
39.21	38.55	39.56	3.19	5.60	570
42.39	41.73	42.66	3.11	5.54	561
45.77	45.11	45.97	3.31	4.75	698
48.95	48.29	49.10	3.13	3.74	836
52.33	51.67	52.43	3.33	3.79	877
55.45	54.79	55.50	3.08	3.60	855
58.89	58.24	58.91	3.40	3.63	938
62.17	61.52	62.15	3.25	3.61	900
65.55	64.90	65.50	3.35	4.05	827
68.73	68.08	68.65	3.15	4.56	692
72.01	71.36	71.91	3.26	4.97	655
75.30	74.64	75.16	3.26	4.78	682
78.58	77.92	78.42	3.26	5.40	603
81.86	81.20	81.68	3.26	4.88	669
85.14	84.48	84.95	3.26	4.62	707
88.42	87.76	88.21	3.26	4.88	668
91.60	90.95	91.38	3.17	4.52	701
94.98	94.32	94.74	3.36	5.05	666
98.26	97.61	98.01	3.27	4.37	747
101.54	100.89	101.27	3.27	4.35	752
104.92	104.27	104.64	3.37	4.66	723
108.10	107.45	107.81	3.17	4.68	678

Job No: 24-59-27281
Client: GeoEngineers
Project: CPFD-Freezer Expansion
Sounding ID: CPT-01-24
Date: 2024-02-26

Seismic Source: Beam
Seismic Offset (ft): 8.86
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)
111.38	110.73	111.08	3.27	4.35	752
114.67	114.01	114.35	3.27	4.94	662
117.95	117.29	117.62	3.27	4.51	725
121.23	120.57	120.90	3.27	4.48	730
124.51	123.85	124.17	3.27	5.07	645
127.79	127.13	127.44	3.27	5.10	642
131.07	130.41	130.71	3.27	4.52	724
134.45	133.79	134.09	3.37	4.50	749
137.63	136.98	137.26	3.18	4.26	745
140.81	140.16	140.44	3.18	3.96	803
144.19	143.54	143.81	3.37	4.66	724
147.47	146.82	147.09	3.28	4.67	701
150.75	150.10	150.36	3.27	4.39	745
154.04	153.38	153.64	3.28	4.15	790
157.22	156.56	156.81	3.18	3.97	801
160.60	159.94	160.19	3.38	4.24	796
163.98	163.32	163.56	3.37	4.32	781
167.26	166.60	166.84	3.28	4.37	749
170.44	169.78	170.01	3.18	3.98	798
173.72	173.06	173.29	3.28	3.99	821
177.00	176.35	176.57	3.28	4.37	750
180.18	179.53	179.75	3.18	4.01	793
183.66	183.01	183.22	3.47	3.69	941
186.84	186.19	186.40	3.18	3.55	896
190.12	189.47	189.68	3.28	3.32	988
192.26	191.60	191.81	2.13	2.01	1060

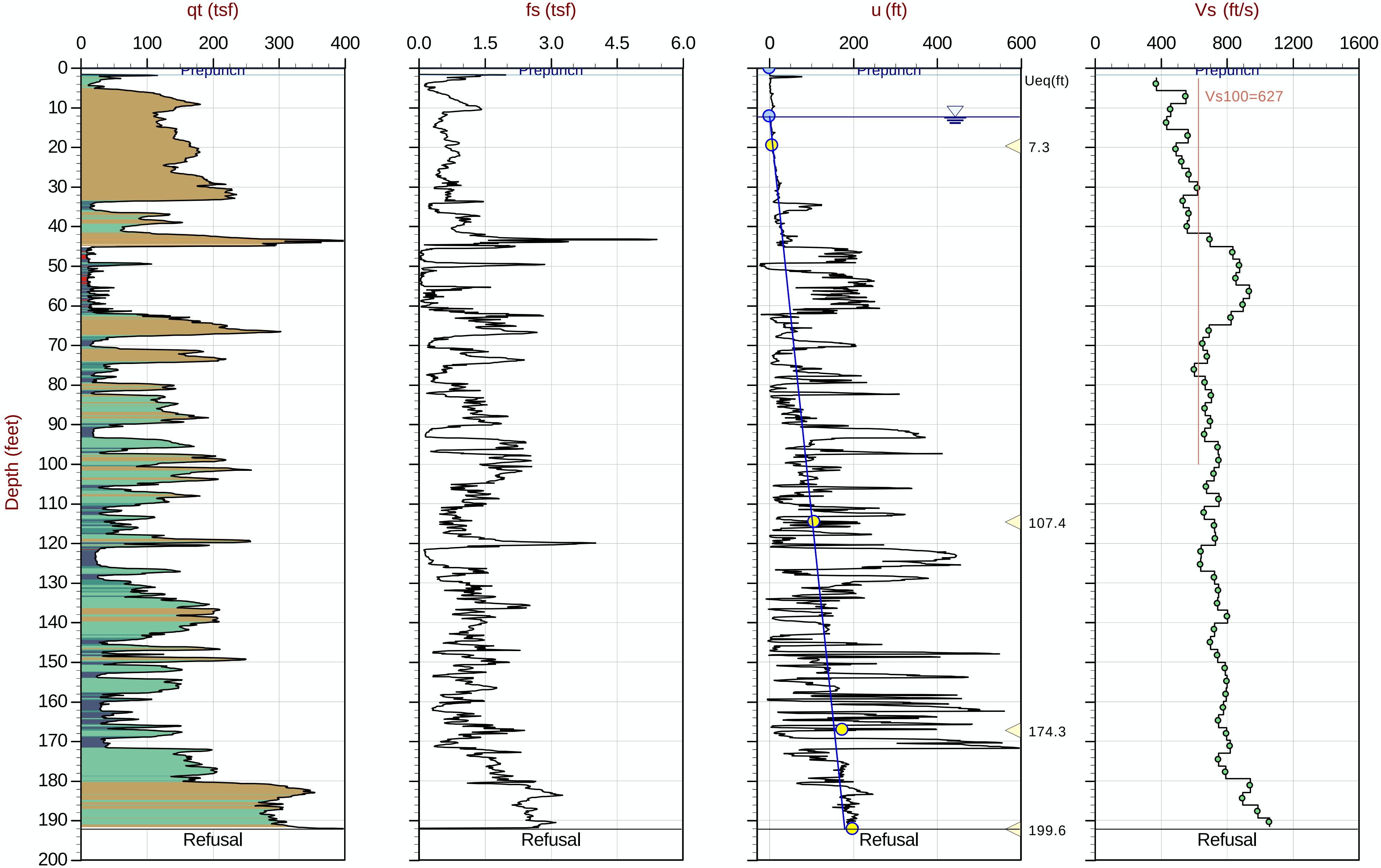
SCPTu Test Plots

ConeTec

GeoEngineers

Job No: 24-59-27281
Date: 2024-02-26 08:58
Site: CPFD-Freezer Expansion

Sounding: CPT-01-24
Cone: 859:T1500F15U35



Max Depth: 58.600 m / 192.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27281_SP01.COR
Unit Wt: SBTQtn (PKR2009)

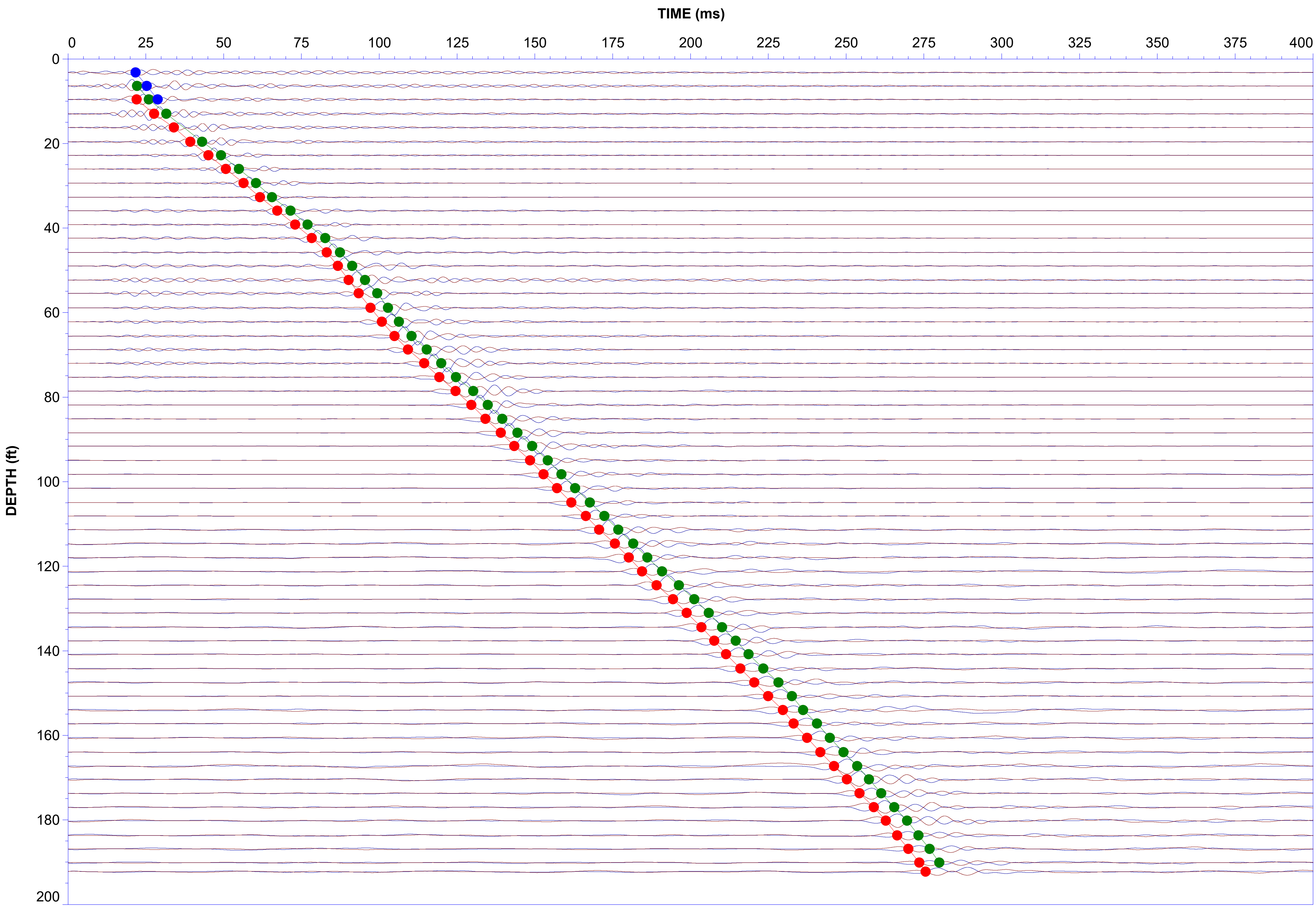
SBT: Robertson, 2009 and 2010
Coords: Lat: 47.20650 Long: -122.29888

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

SCPTu Velocity Wave Traces

Job No: 24-59-27281 | Client: GeoEngineers | Project: CPFD-Freezer Expansion | Analysis: Shear Wave | Sounding: CPT-01-24
Filter: BP 10 - 200 Hz | Date: February 26, 2024 | Cone: 859:T1500F15U35



Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023
Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

Limitations

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

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

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

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

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

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

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

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

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

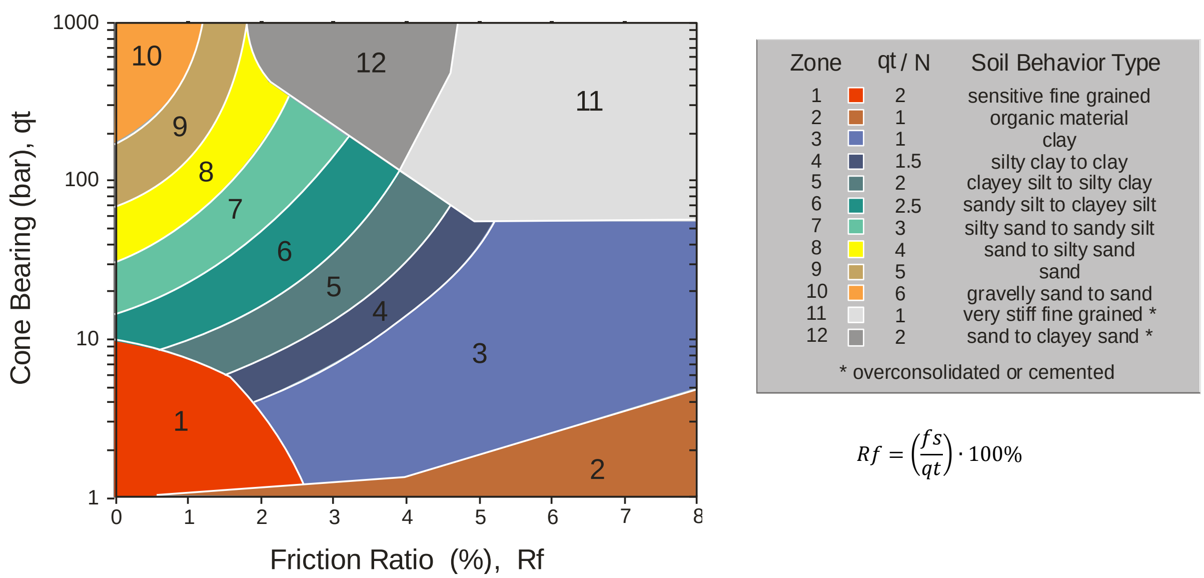
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from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

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

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

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

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



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

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

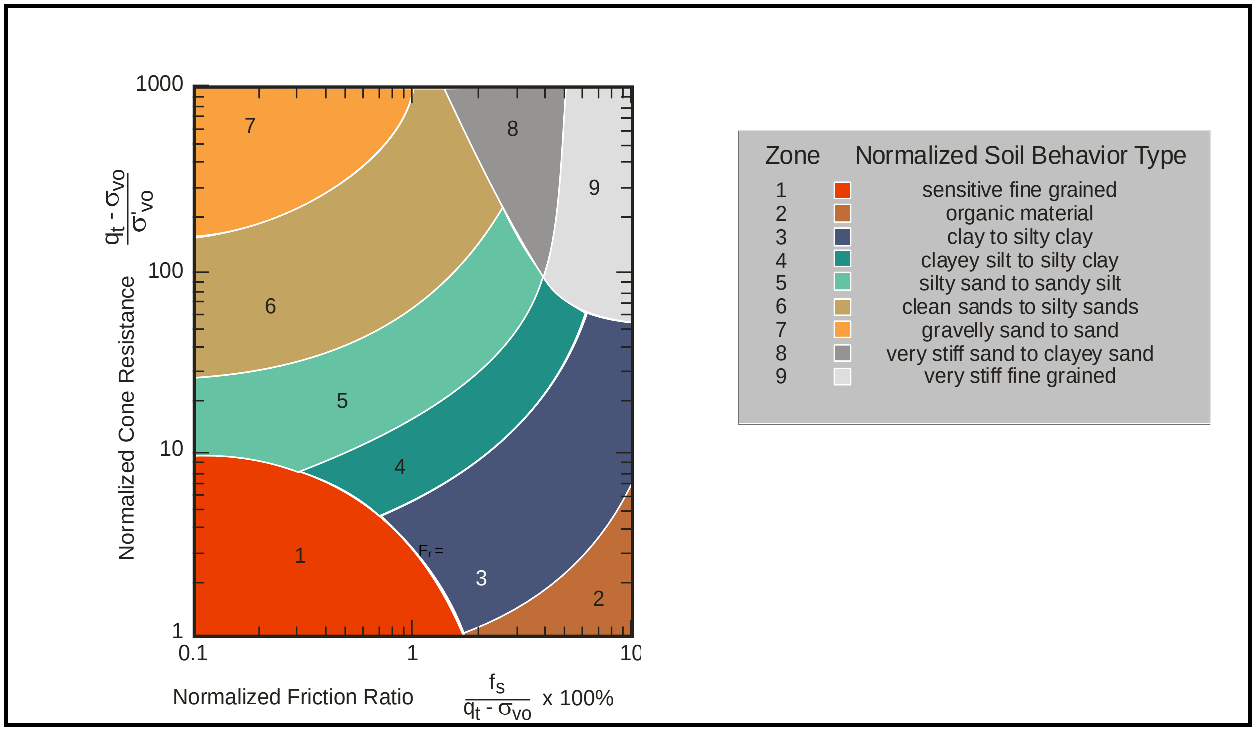


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

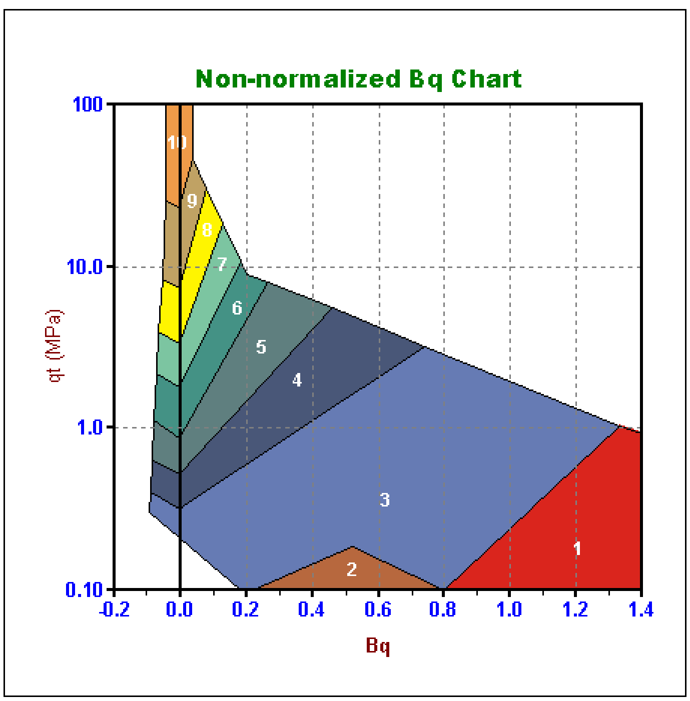


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

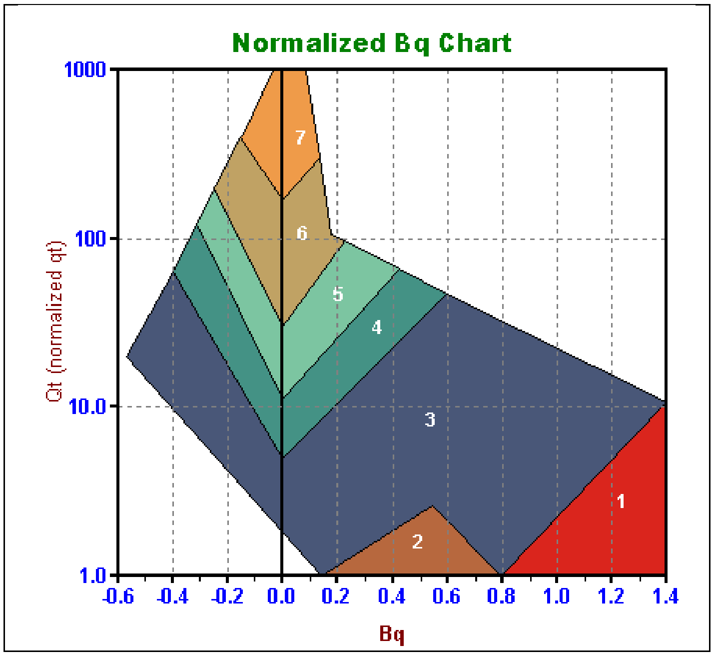


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

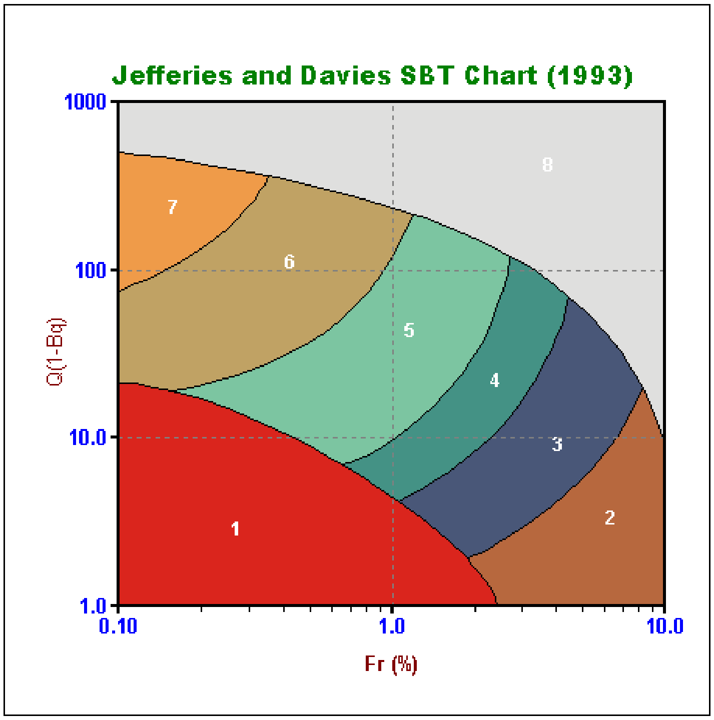


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

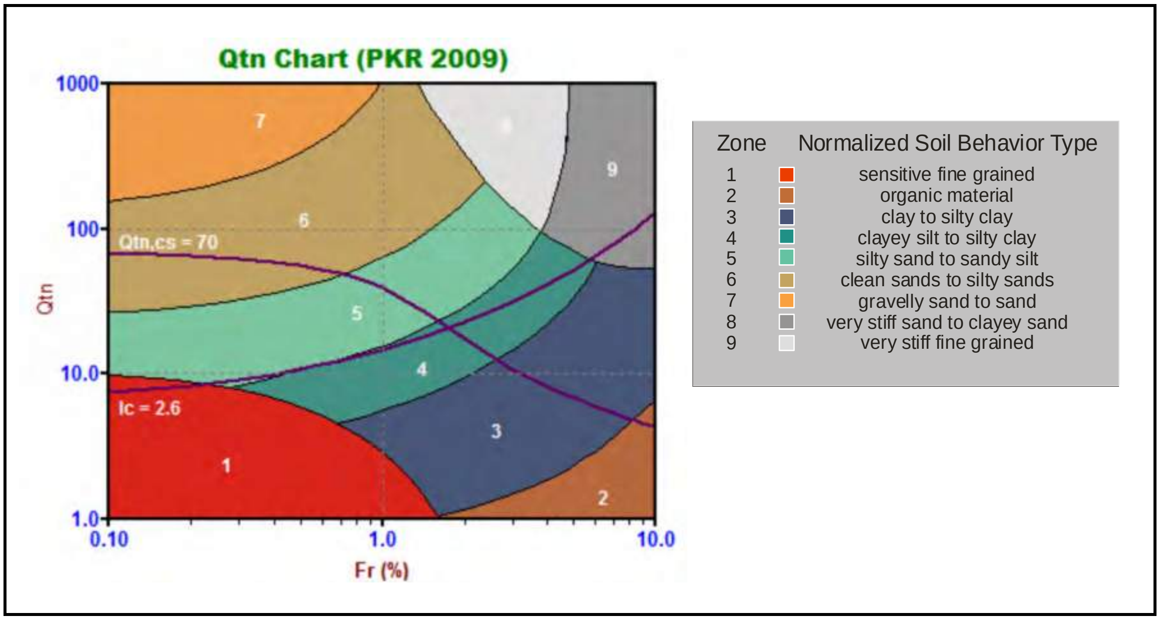


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

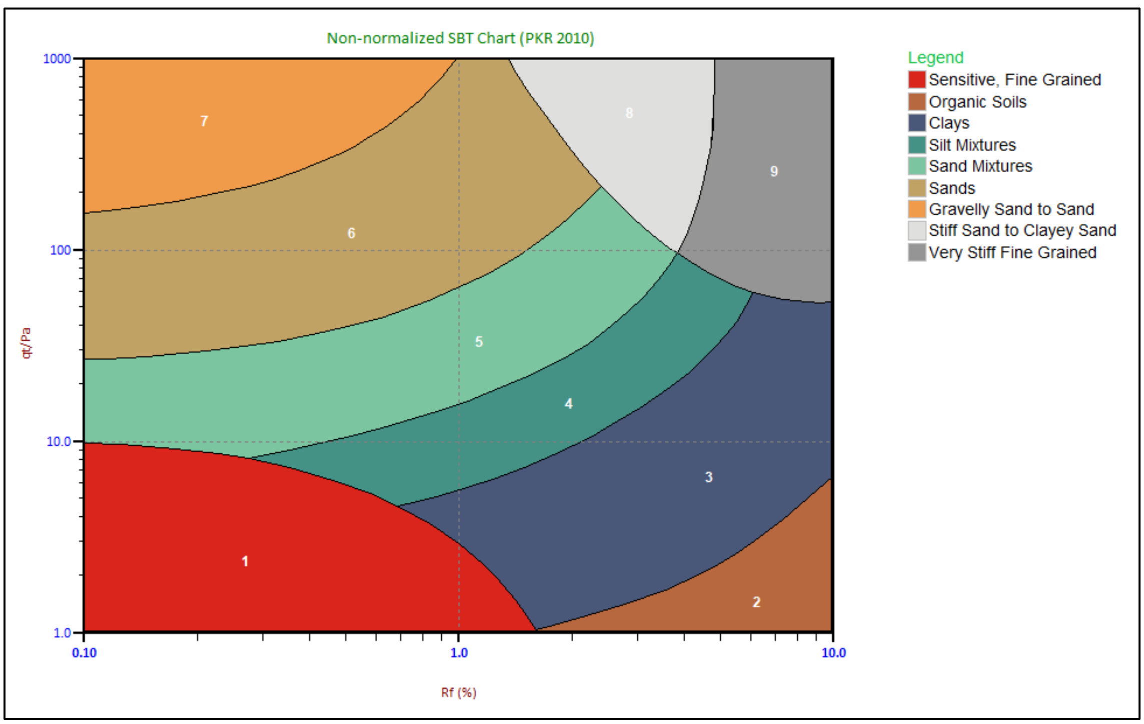


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

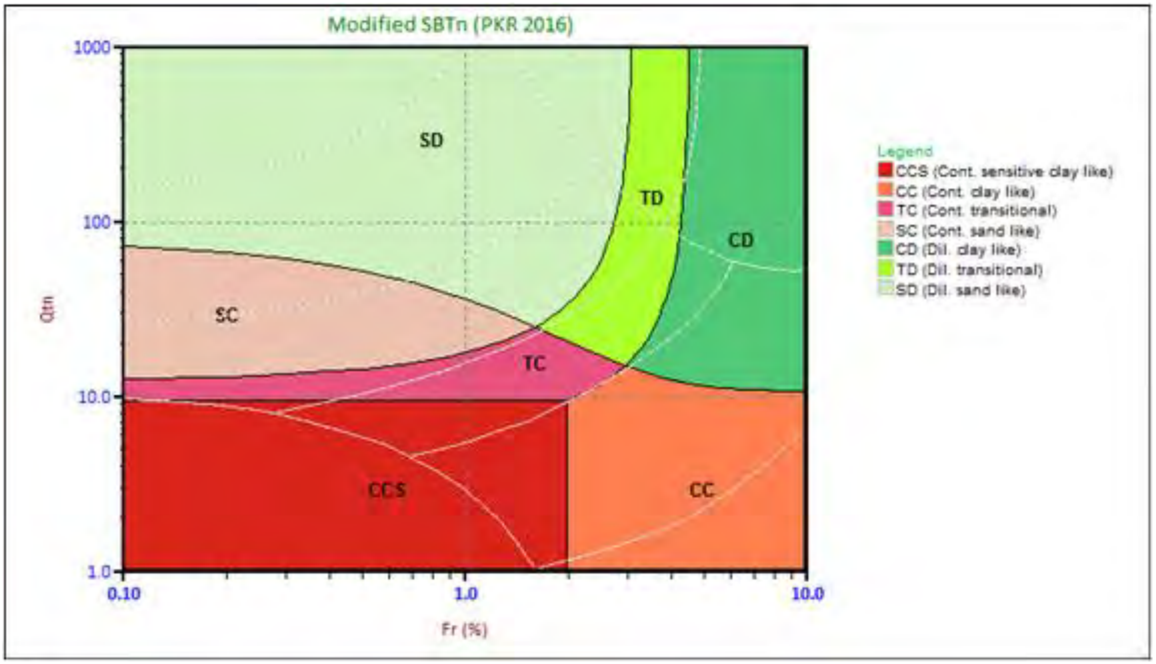


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

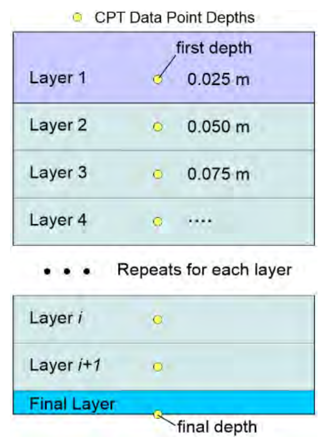
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

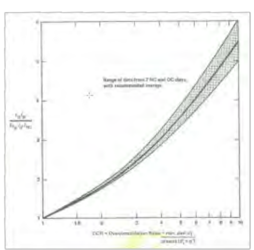
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

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

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

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

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

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

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

Table 2. References

No.	Reference
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Piezocone Calibration Sheets

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

CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC859	Model	A15 T1500 F15 U35
Date	2023-12-13		
Technician Performing Calibration	Richard Chen	Signature	
Calibration Approved By	Vishrut Khunt	Signature	

Lab Condition	As Found	As Left		
Lab Temperature	N/A	24˚C		
Lab Humidity	N/A	29%	Reason for Calibration	Repair

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

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	1.072	0.481
Sleeve	bar	0.002	-0.021
Pressure	bar	1.014	1.013
X-Inclinometer	degrees	0.396	0.014
Y-Inclinometer	degrees	-0.250	0.000
Temperature	˚C	23.239	23.782

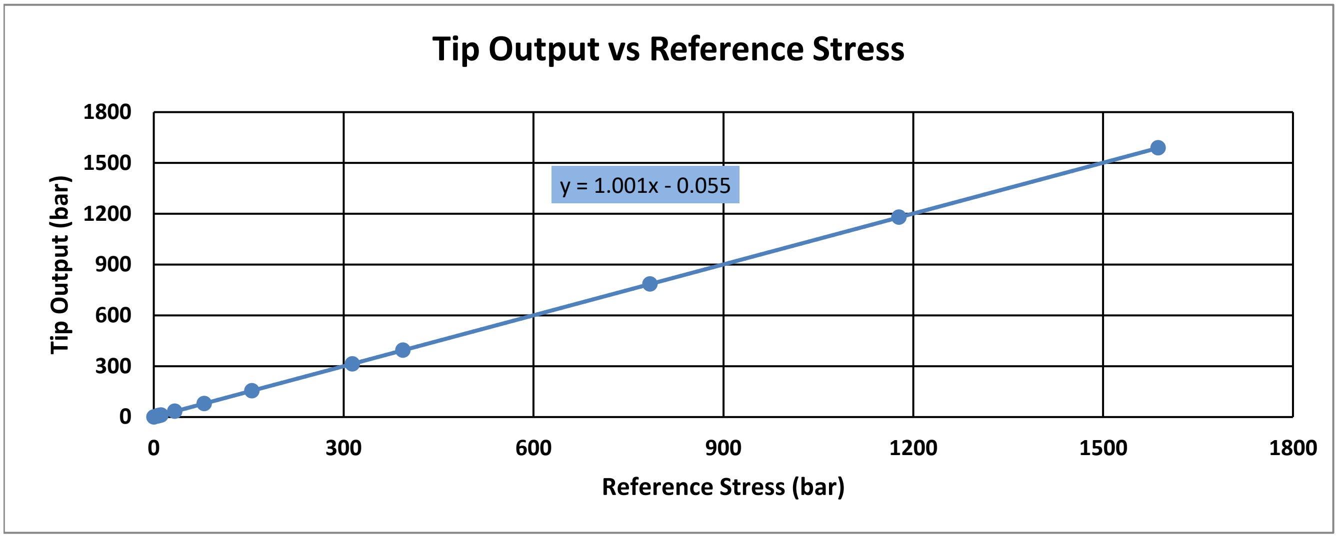
Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

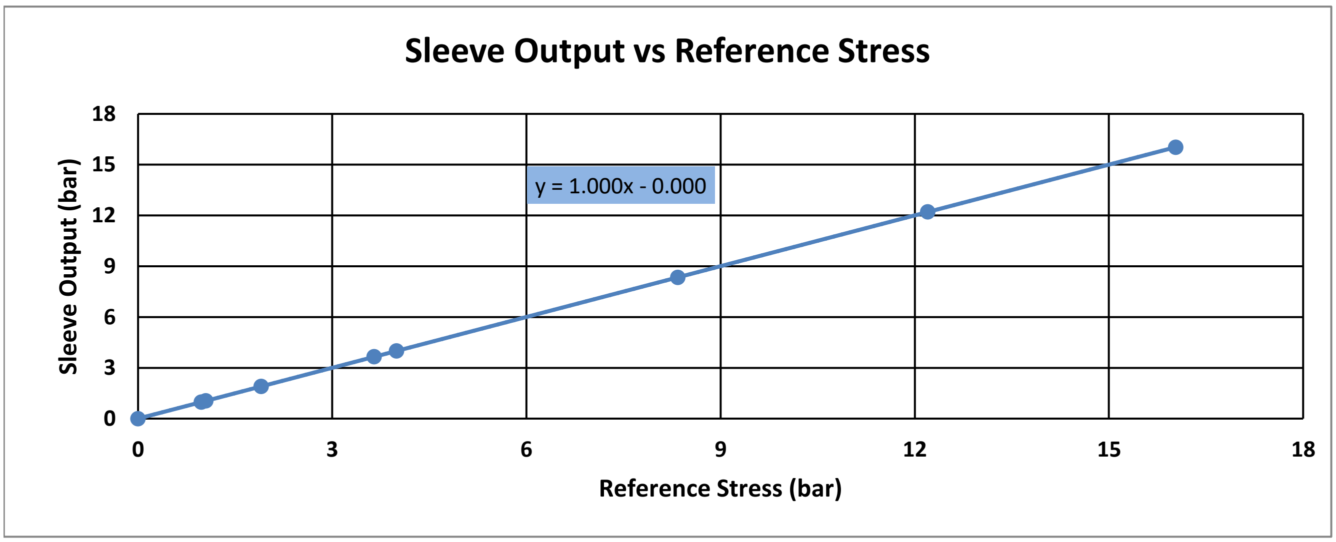
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

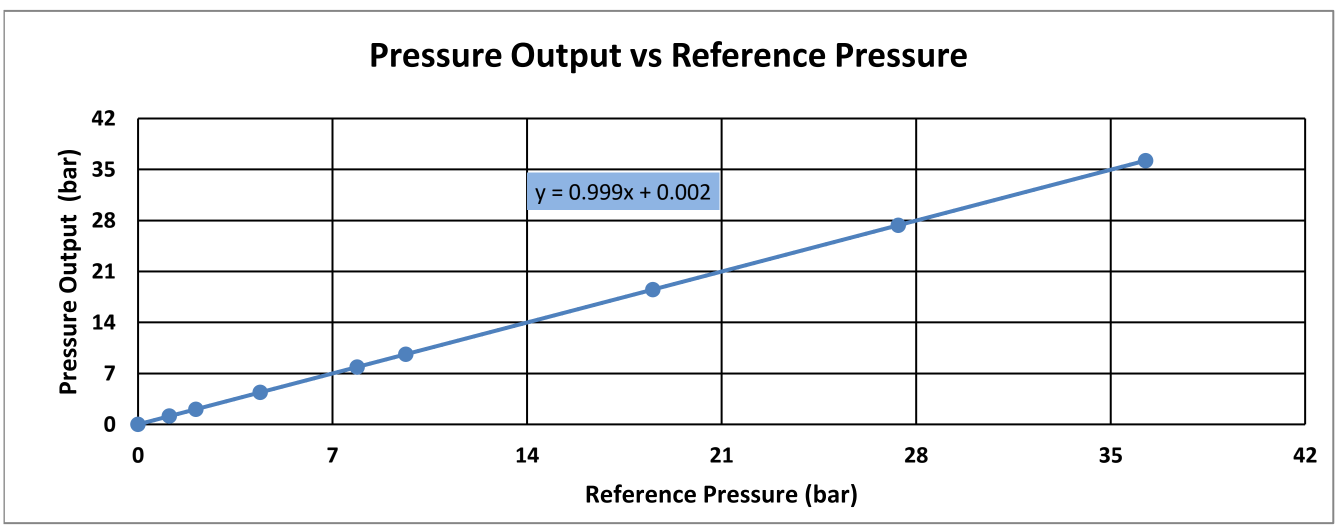
Calibrated with Adara calibration procedure EC_CPTCAL-2.1

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

Cone Output vs Reference Stress/Pressure Plots







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Calibration Results

Tip Calibration					
As Found			As Left		
Max. Non Linearity	0.07%	PASS	Max. Non Linearity	0.09%	PASS
Calibration Error	0.07%	PASS	Calibration Error	0.10%	PASS

Sleeve Calibration					
As Found			As Left		
Max. Non Linearity	0.14%	PASS	Max. Non Linearity	0.03%	PASS
Calibration Error	0.41%	PASS	Calibration Error	0.06%	PASS

Pressure Calibration					
As Found			As Left		
Max. Non Linearity	0.03%	PASS	Max. Non Linearity	0.12%	PASS
Calibration Error	0.08%	PASS	Calibration Error	0.12%	PASS

X-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.04%	PASS
Calibration Error	N/A	N/A	Calibration Error	-0.08%	PASS

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

Seismic Calibration					
As Found			As Left		
Trigger Delay Error	N/A	N/A	Trigger Delay Error	0.00%	PASS

Temperature Calibration			
Full Scale Error	0.27%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	5.3	23.3	43.3	˚C
Tip	3.954	0.275	-1.338	bar
Sleeve	0.051	-0.011	-0.016	bar
Pressure	1.074	1.061	1.052	bar
Temperature	5.340	23.106	43.189	˚C

Tip Temperature Coefficient	-0.138 bar/˚C	PASS
Sleeve Temperature Coefficient	-0.002 bar/˚C	PASS
Pressure Temperature Coefficient	-0.001 bar/˚C	PASS

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

Testing Equipment Details

Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3061	100214	2024-01-05
Tektronix Function Generator	AFG3021B	C030955	100751	2024-10-20
Thermometer	THS-222-555	D23255834	100410	2024-07-11
Thermometer	THS-222-555	D23255829	100410	2024-07-11
Thermometer	THS-222-555	D20345575	100565	2024-07-14

Adara Error Definitions

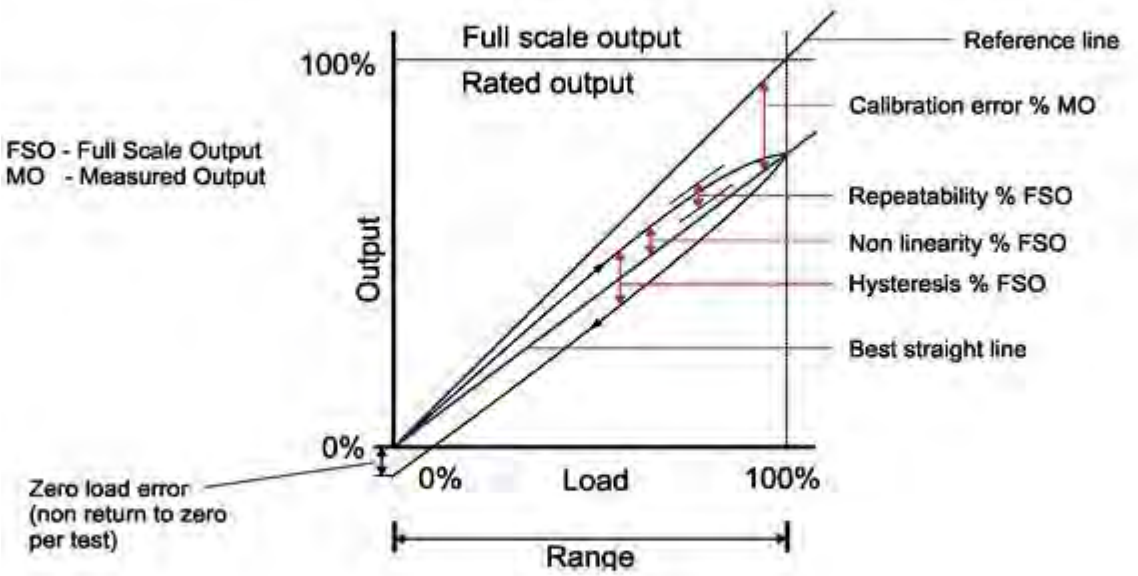

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

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

Temperature Check Passing Criteria

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

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ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

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

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

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

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

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

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

References

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

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

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

APPENDIX D
Report Limitations and Guidelines for Use

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[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 Coastal Pacific Food Distributors 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 Coastal Pacific Food Distributors executed on October 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 Coastal Pacific Food Distributors Freezer Expansion project located in Puyallup, 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.

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.

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.

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 confer with GeoEngineers and/or 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.

GeoEngineers