Exhibit "8"

RED

Chick-fil-A Restaurant #05839

Geotechnical Engineering Report

May 13, 2024 | Terracon Project No. 81245040

Prepared for:

Chick-fil-A, Inc.
5200 Buffington Road
Atlanta, Georgia 30349

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May 13, 2024

Chick-fil-A, Inc.
5200 Buffington Road
Atlanta, Georgia 30349

Attn: Ms. Jessika Guerrero
P: (562) 441-3631
E: jessika.guerrero@cfacorp.com

Re: Geotechnical Engineering Report
Chick-fil-A Restaurant #05839
680 Highway 20 and 808 Nevitt Rd
Burlington, Washington
Terracon Project No. 81245040

Dear Ms. Guerrero:

We have completed the scope of Geotechnical Engineering services for the referenced project in general accordance with Terracon's Task Oder #05839 dated March 21, 2024 and the Master Services Agreement dated March 31, 2005. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, floor slabs, and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultant, Inc.

Baylee A. Sergent, G.I.T.
Staff Geologist

David A. Baska, Ph.D., P.E., C.E.G.
Senior Engineering Consultant

National Account Manager: Joshua J. Schilling

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Attachments

Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Report Summary

Topic [1]	Overview Statement [2]
Project Description	The project involves the construction of a one-story Chick-fil-A restaurant with drive-thru lanes, canopies, a pylon sign, and a parking lot on an approximately 2.37-acre square foot parcel. The proposed building is an approximately 5,970 square feet structure with maximum: ▪ Column loads: 50 kips ▪ Wall loads: 2 kips per lineal foot (klf) ▪ Floor slab load: 125 pounds per square foot (psf) We have designed the pavement section for an estimated 18-kip equivalent single axel load (ESALs) of 50,000. If the traffic loading is expected to exceed this amount please notify Terracon so we can revisit our pavement design parameters.
Geotechnical Characterization	The motel parking area appears to be comprised of asphalt overlying concrete pavement. The historic gas station area in the northwestern corner of the site includes asphalt pavement and sparsely vegetated areas. Approximately 4 to 6 inches of topsoil exists within the undeveloped, vegetated areas of the site. A summary of subsurface findings at our exploration locations are presented below: ▪ **Fill:** Existing undocumented fill comprised of loose silty sand and poorly graded gravel with silt and sand was encountered to a depth of 1 to 3 feet at exploration locations B-04 and MW-01. ▪ **Upper Alluvium A**: Soft to very stiff, nonplastic, silt with variable sand content. This unit extends to about 3½ to 11½ feet. ▪ **Upper Alluvium B**: Very loose to medium dense sand with variable silt and gravel content. This unit is anticipated to extend to depths of approximately 31 to 34 feet. ▪ **Upper Alluvium C**: Soft to very stiff clay and silt with interbedded sand. This unit underlies Upper Alluvium B and extends to depths between approximately 34 to 55 feet. ▪ **Lower Alluvium**: Medium dense to dense sand with variable silt and gravel content. This unit underlies Upper Alluvium C and extends to the maximum exploration depth of approximately 77 feet.

Topic [1]	Overview Statement [2]
	Upper Alluvium C and Lower Alluvium are inferred based on the cone penetration testing (CPT). Groundwater was observed to depths between 8 and 12 feet below ground surface elevation (bgs).
Seismic Considerations	The Seismic Site Class is F due to the site-wide liquefaction hazard. Site Class D was used to develop the seismic design parameters, assuming the fundamental period of vibration for site structures would be equal to or less than 0.5 seconds. The total seismic-induced settlement is estimated to be about 13 and 15 inches at CPT-01 and sCPT-01, respectively, for the upper 60 feet. Post-liquefaction differential settlements are estimated to be a maximum of 6 inches over a distance of 100 feet. In addition, a bearing capacity failure is feasible. We computed lateral spread displacements of 24 and 115 inches at CPT-01 and sCPT-01, respectively, assuming a ground slope of 0.5 percent.
Liquefaction Mitigation	We recommend that the lateral spread and differential settlement hazards be mitigated with a mat foundation or grade beams. Ground improvement may be required to mitigate a post-liquefaction bearing failure. We recommend that the structural engineer and Terracon discuss seismic bearing pressures, tolerable post-liquefaction displacements and mitigation strategies.
Earthwork	Remove the upper 6 inches of topsoil, tree roots, and existing fill where encountered. Allow observation by the Geotechnical Engineer to help identify if additional stripping and site preparation are needed. Near-surface soils contain a high silt content and are moisture sensitive. Subgrades may become unstable when exposed to excessive moisture. Utility trench stability may be impacted by the presence of very loose/soft on-site soils and relatively shallow groundwater. The utility subcontractor should be prepared to contend with these conditions.
Shallow Foundations	Shallow foundations with seismic ties are feasible with ground improvement. Preliminary allowable bearing pressure = 3,000 to 4,000 psf (spread and wall footings) Expected settlements: < 1-inch total, < 2/3-inch differential Detect and remove zones of fill as noted in **Earthwork**.

Topic [1]	Overview Statement [2]
Canopy and Pylon Sign Foundations	Augured cast-in-place (ACIP) piles are recommended for canopies and the pylon sign foundations. Recommendations for end bearing, side friction, and L-Pile parameters for the structural engineer's evaluation of lateral loading are provided.
Pavements	With subgrade prepared as noted in **Earthwork** we have provided pavement design parameters for the assumed traffic loading. Parking and driveway areas for passenger vehicles: ▪ 3 inches ACC over 8 inches granular base Dumpster Pad: ▪ 5 inches PCC over 4 inches granular base
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering services performed for the proposed Chick-fil-A restaurant #05839 to be located at 680 Highway 20 and 808 Nevitt Rd in Burlington, Washington. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic considerations
- Liquefaction
- Ground improvement
- Site preparation and earthwork
- Foundation design and construction
- Floor slab design and construction
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of seven (7) soil borings and two (2) cone penetration tests (CPTs) to depths ranging from approximately 10½ to 26½ and 73½ to 77 feet below existing site grades, respectively, laboratory testing, engineering analysis, and preparation of this report.

Drawings showing the site and exploration locations are shown on the **Site Location** and **Exploration Plan**, respectively. Results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs. The boring and CPT logs are shown as separate graphs in the **Exploration and Laboratory Results** section. Subcontractor reports and historical exploration results of the adjacent site to the west are included as attachments in the **Supporting Information** section.

Project Description

Our initial understanding of the project was provided in our proposal, and our current understanding of the project conditions is as follows:

Item	Description
Information Provided	▪ Email request from Chick-Fil-A on March 11, 2024 ▪ Conceptual Site Plan LAX23-0099099 Page 01, prepared by Ware Malcomb and dated November 16, 2023

Item	Description
Project Description	The project involves the construction of a one-story Chick-fil-A restaurant with drive-thru access and parking lot on approximately 2.37-acre parcel. Based on the site plan, the project also includes the demolition of the existing building on site.
Proposed Structures	Per the Conceptual Site Plan, footprint of the proposed restaurant is approximately 5,970 square feet. The project includes a pylon sign, approximately 102 parking spaces, double drive-thru lanes with two canopies, driveway entrances from Nevitt Road to the west, and driveways to other developments to the south.
Building Construction	Based on previous work with Chick-fil-A restaurants, we assume the building construction to be steel and/or wood framing over concrete foundation and floor slab.
Finished Floor Elevation	Expected to be at or near existing grades.
Maximum Loads	We have assumed the following loads based on work on previous Chick-fil-A restaurants: ▪ Column Load: 50 kips ▪ Load-Bearing Wall Loads: 2 klf ▪ Maximum Uniform Floor Slab Load: 125 psf
Grading/Slopes	A grading plan was not provided. We assume that earthwork will be limited to only excavation and removal of the existing building foundations, as well as minor cuts and fills that are necessary to level the site.
Pavements	Proposed traffic will consist of automobiles with occasional delivery and garbage trucks. Based on our expectation that the parking area will be subjected to automobile traffic only and that the drive areas will be subjected to a maximum of five delivery trucks/trash collection trucks per week, we have assumed an 18-kip equivalent single axel load (ESALs) of 50,000 for the parking lot, drive areas, and dumpster pad. A pavement design period of 20 years is also assumed.
Building Code	In 2022, the State of Washington adopted the Multi-Period Response Spectrum (MPRS) of ASCE 7-22 for determination of design ground motion values. The amendment requires use of the updated Site Class designations found in Chapter 20 of ASCE 7-22. Terracon has assumed 2021 IBC (ASCE 7-16) with ASCE 7-22 for the seismic design parameters.

Terracon should be notified if any of the above information is inconsistent with the planned construction, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	This project is located at 680 Highway 20 and 808 Nevitt Rd in Burlington, Washington. Lot size: approximately 2.37 acres Skagit County Tax Parcel No.s: P23672 and P23664 Latitude: 48.4697° North Longitude: 122.3434° West See **Site Location**
Existing Improvements	The site is currently developed with an approximate 5,422 square-foot one-story structure, which includes both a motel and detached office building. The development also includes an asphalt paved parking area underlain by concrete with a paved drive entrance from Highway 20 to the north and a gravel access road connecting Nevitt Road to the west with the motel parking area. The paved motel parking area continues south to the asphalt paved McDonalds Restaurant parking lot. In the northwest portion of the site exists an approximate 2-foot high concrete retaining wall that boarders the fractured remnant asphalt paved parking area of the historical gasoline station. The site is bordered to the north by Highway 20 followed by railroad tracks and Burlington Terrace Apartments (650 Peterson Place), to the east by I-5 right-of-way followed by I-5, to the south by McDonalds Restaurant (876 Nevitt Road), and to the west by Nevitt Road followed by other commercial one-story buildings.
Current Ground Cover	The site ground cover consists of asphalt and concrete paved parking, grassed land, and a gravel access road. Mature trees exist within the proposed building footprint in the western portion of the site.
Existing Topography	The site gently slopes from an elevation of approximately 28 feet at the southernmost boundary to 30 feet in the northeastern corner of the site according to **Google Earth Pro.**

Item	Description
Site History	From review of historical aerial photographs, it appears the existing motel has been on site since at least 1998. In 2009, the southern portion of the motel and a gasoline station that existed in the northwest portion of the site were demolished and the site has operated as the Mark II Motel to the present day. Additional site history can be found in the Phase 1 assessment Terracon completed for this site, Terracon Report No. 81247197.
Geology	The site lies within the Skagit River valley within the City of Burlington. The surface geology of the site is mapped as Quaternary alluvium (Qal), which consists of fluvial sand, silt, and gravel with minor lacustrine deposits. Based on observations made during the subsurface investigation, subsurface conditions on the project site appeared to be consistent with the published geologic conditions. **Map reviewed**: *Geologic Map of the Port Townsend Quadrangle, Skagit County, Washington, USGS, 1989.*

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

Geotechnical Characterization

Subsurface Profile

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point is indicated on the individual logs. The individual logs can be found in the **Exploration Results** and the GeoModel (constructed using only the borings) can be found in the **Figures** attachment of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. The model layers in the table below are based on both the borings and the CPTs. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer[1]	Layer Name	USCS	General Description
-	**Surface**	--	▪ Approximately 7 inches of asphalt in the northwestern corner of the site. ▪ Approximately 2 inches of asphalt followed by two layers of concrete measuring about 2½ and 3½ inches, respectively, in the paved motel parking area. ▪ Approximately 4 to 6 inches of topsoil in vegetated areas.
1[2]	**Fill**	SM, GP-GM	Fill comprised of loose silty sand and poorly graded gravel with silt and sand. Contains trace organics. This unit extends to about 1 to 3 feet bgs.
2[3]	**Upper Alluvium A**	ML	Soft to very stiff, nonplastic, silt with variable sand content. This unit extends to approximately 3½ to 11½ feet.
3[4]	**Upper Alluvium B**	SP, SP-SM, SW, SM	Very loose to medium dense sand with variable silt and gravel content. This unit extends to the maximum depth of the soil borings and is anticipated to extend to depths of approximately 31 to 34 feet based on CPT data.
4[5]	**Upper Alluvium C**	--	Soft to very stiff clay and silt with interbedded sand. This unit underlies Upper Alluvium B and extends to depths between approximately 34 to 55 feet.
5[5, 6]	**Lower Alluvium**	--	Medium dense to dense sand with variable silt and gravel content. This unit extends to the maximum exploration depth of approximately 77 feet.

1. This summary is for convenience only. It should be used in conjunction with the entire report.
2. This soil unit was encountered at MW-01 and B-04.
3. B-04 was terminated within this unit.
4. All soil borings except B-04 were terminated within this unit.
5. Model layer and general description inferred from Soil Behavior Type (SBT) and Standard Penetration Test (SPT) N_{60} values corrected for overburden pressure using N_{60} I_c from CPT results and calculations.
6. sCPT-1 and CPT-1 were refused within this unit.

Groundwater Conditions

Monitoring wells were constructed following the advancement of soil borings MW-01 through MW-03. The soil borings were advanced in a manner consistent with the **Exploration and Testing Procedures** section. The well construction consisted of a screen interval and sand pack from the bottom of the borehole to about 15 feet below the existing ground surface (bgs). On April 9, 2024 data logging piezometers were installed in monitoring wells MW-01 through MW-03 to record and monitor groundwater levels daily. Groundwater level monitoring at these wells will continue through the wet season to observe potential fluctuations due to seasonal variations.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Mapping by the Natural Resources Conservation Service (NRCS) indicates a seasonal high groundwater level within 36 inches of the ground surface. Nearby water well reports obtained from the **Washington Geologic Information Portal** indicate that the groundwater table ranges between 8 and 12 feet below ground surface at the water wells within 5-mile of the site.

Our estimates of the seasonal groundwater conditions are based on the USDA Soil Survey, the encountered soil types, and the interpreted water levels. The water levels observed in the boreholes can be found on the boring logs in **Exploration and Laboratory Results** and are summarized below.

Exploration ID	Approximate Depth to Groundwater Inferred while Drilling (feet) [1, 2]	Approximate Depth to Groundwater Measured after Drilling (feet) [1, 3]
MW-01	9	11½
MW-02	9½	11
MW-03	9½	10½
B-01	9	-
B-02	9½	-
B-03	8½	-
B-04	8	-

1. Below ground surface.
2. Inferred from change in sample moisture while sampling or from evidence of free water on drilling equipment.
3. Measured with a water level meter at least 1 hour after drilling.

Estimated depth to groundwater was performed through pore pressure dissipation testing during CPT advancement. Groundwater levels tested at each exploration location can be found on individual CPT logs in **Exploration and Laboratory Results** section and are summarized in the table below.

Exploration ID	Approximate Depth to Groundwater (feet) [1]
sCPT-01	9
CPT-01	12

1. Below ground surface.

Seismic Considerations

Ground Motion

In 2022, the State of Washington adopted the Multi-Period Response Spectrum (MPRS) of ASCE 7-22 for determination of design ground motion values. The amendment requires use of the updated Site Class designations found in Chapter 20 of ASCE 7-22. The MPRS values were obtained from the ASCE 7-22 online tool (**https://asce7hazardtool.online/**) and are presented in the below table.

Description	Value [1]
ASCE 7-22 Site Classification[2]	F
Site Latitude	48.4697° North
Site Longitude	122.3434° West
S_S – Short Period Spectral Acceleration	1.19 g
S_1 – 1-Second Period Spectral Acceleration	0.35 g
S_{MS} – Short Period Spectral Acceleration Adjusted for Site Class	1.38 g
S_{M1} – 1-Second Spectral Acceleration Adjusted for Site Class	0.96 g
S_{DS} – Design Short Period Spectral Acceleration	0.92 g
S_{D1} – Design 1-Second Spectral Acceleration	0.64 g
PGA_M - ASCE 7, Peak Ground Acceleration Adjusted for Site Class	0.55 g

Description	Value [1]

1. The IBC requires a site profile extending to a depth of 100 feet for seismic site classification. We performed one Seismic Cone Penetration Test at the site to measure shear wave velocities to the depth of refusal at 77 feet of the subsurface materials at the site. The site properties below the exploration depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area.
2. Site Class F is because of liquefiable soils, which are soils vulnerable to potential failure or collapse under seismic loading. Values in the table are for Site Class DE based on the time-averaged shear wave velocity value of 629 ft/sec and assuming the exception in Section 20.2.1 of ASCE 7 for structures with a fundamental period of vibration equal to, or less than, 0.5 seconds applies. The fundamental period of vibration should be verified by the structural engineer.

Surface-Fault Rupture

The risk of damage from onsite fault rupture appears to be low based on review of the USGS Earthquake Hazards Program Quaternary Faults and Folds Database available online (**https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf**) accessed on April 29, 2024. The closest mapped fault is the Darrington-Devils Mountain fault zone, which lies approximately 7½ miles to the south of the proposed project site, and has a slip rate of less than 0.2 mm/yr.

Liquefaction

Liquefaction is the phenomenon where saturated soils develop high pore water pressures during seismic shaking and lose their strength characteristics. This phenomenon generally occurs in areas of high seismicity, where groundwater is shallow and loose granular soils or relatively non-plastic fine-grained soils are present. Based on the site geology and subsurface groundwater conditions, the hazard of liquefaction of the site soils is high during a design level earthquake and is most likely to trigger between the groundwater table and the bottom of Upper Alluvium B; however, liquefiable zones are anticipated within Upper Alluvium C and Lower Alluvium.

Liquefaction analysis for the CPT data obtained was performed using software Cliq developed by Geologismiki. For consideration of liquefiable soils in the upper 60 feet, we computed free-field, seismic-induced total settlements of approximately 13 and 15 inches for CPT-01 and sCPT-01, respectively. We estimate maximum differential settlements of less than 6 inches over a distance of 100 feet. Computed lateral displacements for a ground slope of 0.5 percent were 24 inches at CPT-01 and 115 inches at sCPT-01. Given that the top of the liquefiable soil is located approximately 10 feet below the ground surface, a post-liquefaction bearing capacity failure is feasible for the building structure.

Geotechnical Overview

On April 9, 2024, seven soil borings were advanced throughout the site to the maximum depth of approximately 26½ feet below ground surface (bgs). Soils observed at the time of drilling generally consisted of soft to very stiff silt with variable sand content (Model Layer 2) to approximately 3½ to 11½ feet bgs underlain by very loose to medium dense silty sand or poorly graded sand (Model Layer 3). In addition, potential fill material was observed from 1 foot to approximately 3 feet bgs in the northwestern and southeastern portion of the site (Model Layer 1). Three of the borings were converted to monitoring wells and groundwater was identified between 8 and 12 feet bgs at our exploration locations. In addition to soil borings, two CPTs were performed to a maximum depth of approximately 77 feet. The CPTs identified the Model Layer 3 extends to about 35 feet bgs. An approximately 20-foot thick layer of relatively soft to very stiff clay and silt with interbedded sand (Model layer 4) underlies Model Layer 3. Model layer 4 overlies Lower Alluvium (Model Layer 5), which consists of medium dense to dense sand with variable gravel and silt content and extends to the refusal depth of the CPTs.

Given the type of building structure and assumed risk category, we anticipate that the estimated differential settlement of 6 inches will meet the threshold criterion in Table 12.13-3 of ASCE 7-16. However, we anticipate that the lateral spread threshold in Table 12.13-2 of ASCE 7-16 will be exceeded and that a post-liquefaction bearing capacity failure is feasible. Rather than support the building on deep foundations, we anticipate that ground improvement with aggregate piers and a mat foundation or shallow spread footings connected with grade beams would mitigate the liquefaction hazards. Foundation discussion and recommendations are provided in the **Shallow Foundations** and **Ground Improvement** sections.

Augered and Cast-in-Place (ACIP) piles are anticipated to be feasible as the foundation type for the proposed canopies and pylon sign. However, the depth of liquefiable soils likely exceeds the practical depth of the ACIP piles. If the seismic performance objective of the canopies and pylon sign is no collapse, we recommend that Terracon and the structural engineer further discuss the options.

Existing uncontrolled fill soils were observed at boring locations MW-01 and B-04 to 3 and 1 feet bgs, respectively, and may be deeper at other locations. Construction over existing fill presents and inherent risk for the owner that compressible fill or unsuitable material, within or buried by the fill, will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill. We recommend removal of all existing fill and replacement with structural fill given its apparent shallow depth.

Based on our exploration results, the Upper Alluvium A (Model Layer 1) generally appears to be in a very loose to loose/soft to medium stiff state. This unit, in its native condition, would be marginal for directly supporting floor slabs and pavements. Therefore, support of floor slabs and pavement on at least two feet of compacted structural fill over native soil is recommended.

The near-surface soils encountered within the northern portion of the site contain a significant fine content (percent passing the #200 sieve); therefore, these soils will exhibit sensitivity to excessive moisture and/or disturbance and could become unstable with typical earthwork and construction traffic, especially after precipitation event. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

Specific conclusions and recommendations regarding these geotechnical considerations, as well as other geotechnical aspects of design and construction of foundation systems and other earthwork related phases of the project are outlined in the following sections. ASTM and Washington State Department of Transportation (WSDOT) specification codes cited herein respectively refer to the current manual published by the American Society for Testing & Materials and the current edition of the *Standard Specifications for Road, Bridge, and Municipal Construction, (M41-12)*.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration and Laboratory Results**), engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

Earthwork

Earthwork is anticipated to include clearing and grubbing, removal of existing fill, proofrolling the subgrade, structural fill placement, demolition of the existing building and associated utilities, and excavations for foundation elements and utility trenches. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include quality criteria necessary to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

Site Preparation

Prior to placing fill, existing vegetation, topsoil, and root mats should be removed. Complete stripping of the topsoil should be performed in the proposed building and parking/driveway areas. This includes removal of trees and their associated root systems as well as topsoil and grass root mats. All roots larger than 1 inch in diameter should be removed. Based on our explorations, the depth of stripping is approximately 4 to 6 inches, but greater stripping depths may be encountered during earthwork construction.

All prepared subgrade should be observed by Terracon prior to casting of building foundations or placement of capillary break for slab on grade floors. As mentioned in **Geotechnical Overview** section, the near-surface soil (Model Layer 1) generally contain a significant fines content and could be moisture sensitive. Maintaining the condition of subgrade after observation by Terracon will be the responsibility of the contractor. In addition, this soil unit in a native condition would be marginal for directly supporting the floor slabs and pavements. Therefore, for the floor slab and pavement subgrades, we recommend overexcavation of 2 feet and replacement with compacted structural fill.

Following the fill placement, the pavement subgrade should be proof-rolled with an adequately loaded vehicle such as a fully loaded tandem axle dump truck. The proof-rolling should be performed under the observation of the Geotechnical Engineer. Areas excessively deflecting under the proof-roll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or replaced by tested and approved structural fill. Excessively wet or dry material should either be removed or moisture conditioned and recompacted.

Although no evidence of underground facilities (such as septic tanks, cesspools, basements, and utilities) was observed during the exploration and site reconnaissance, such features could be encountered during construction. If unexpected underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Existing Fill

As noted in **Geotechnical Characterization**, borings MW-01 and B-04 encountered previously placed fill to depths ranging from about 1 to 3 feet. Support of foundations and floor slabs on existing fill is not recommended. Construction of foundations and floor slabs on existing fill carries considerate risk of unpredictable settlements. For floor slab and pavement areas, we recommend removing the existing fill and replacing with compacted structural fill.

Fill Material Types

Fill required to achieve design grade should be classified as structural fill and general fill. Structural fill is material used below, or within 10 feet of structures, pavements or constructed slopes. General fill is material used to achieve grade outside of these areas.

Fill materials should meet the following material property requirements. On-site materials are anticipated to be reusable as common fill, but moisture conditioning requirements at the time of earthwork construction may make reuse infeasible. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade.

Fill Type	Recommended Materials	Acceptable Location for Placement
Structural Fill	9-03.9(3) *Crushed Surfacing Base Course* [1] 9-03.12(1)A *Gravel Backfill for Foundations Class A* [1] 9-03.14(1) *Gravel Borrow* [1]	Beneath and adjacent to structural slabs, foundations, building appurtenances, and pavement subgrades
Common Fill	Section 9-03.14(3) *Common Borrow* [1]	Grade filling, utility trench backfill outside the building foundation and appurtenances
Free-Draining Granular Fill	Structural Fill [4] 9-03.12(2) *Gravel Backfill for Walls* [1] 9-03.12(4) *Gravel Backfill for Drains* [1]	Backfilling in wet weather, drainage layers for walls, sump drains, footing drains [5]

1. WSDOT Standard Specifications
2. Structural and common fill should consist of approved materials free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to use on this site.
3. May contain local areas of higher fines content that could make this material moisture sensitive. Particles with a nominal diameter greater than about 3 in. should be removed.
4. Material provided must be specified to be less than 5-percent passing the #200 sieve for the portion of material passing the #4 sieve.
5. Minimum particle size must be greater than drain pipe perforations.

Other earthen materials may be suitable for use in addition to the options presented in the table above. All materials should be approved by the Geotechnical Engineer prior to use. Depending on conditions at the time of construction, Terracon may recommend a geotextile for separation of native fine-grained soils and coarse-grained import soils.

Fill Placement and Compaction Requirements

Structural and general fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Lift Thickness	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used	Same as structural fill
Minimum Compaction Requirements [1,2,3]	95% of max. below foundations and within 2 feet of finished pavement subgrade 92% of max. above foundations, below floor slabs, and more than 2 feet below finished pavement subgrade	92% of max. if some settlement is tolerable
Water Content Range [1]	Granular: -2% to +2% of optimum	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the modified Proctor test (ASTM D 1557).

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

On-site materials are not considered suitable for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface, provided the soil is free of organic matter and deleterious substances. Granular soils are recommended for trench backfill in structural areas due to their relative ease of compaction in confined areas as opposed to cohesive soils.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. In our opinion, the initial lift thickness should not exceed on foot unless recommended by the manufacturer to protect utilities from damage by compacting equipment. Light, hand-operated compaction equipment in conjunction with thinner fill lift thickness may be used on backfill placed above utilities if damage resulting from heavier compaction equipment is of concern.

All trenches should be wide enough to allow for compaction around the haunches of the pipe. We recommend that utility trench excavations be completed using a smooth excavation bucket (without teeth) to reduce the potential of subgrade disturbance. If water is encountered in the excavations, it should be removed prior to fill placement.

Flexible connections for utilities that pass through building foundations are recommended to reduce potential stress associated with differential settlement that may occur between the building foundation and the improvements located outside of the building footprint.

Grading and Drainage

All grades must provide effective drainage away from the building during and after construction and should be maintained throughout the life of the structure. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks at a distance of at least 10 feet from the building.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the building for at least 10 feet beyond the perimeter of the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically observed and adjusted, as necessary, as part of the structure's maintenance program. Where paving or flatwork abuts the structure, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Earthwork Construction Considerations

Shallow excavations for the proposed structure are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of grade-supported improvements such as floor slabs and pavements. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to floor slab construction.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be observed by the Geotechnical Engineer (or others under their direction). Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, and pavements), evaluation and remediation of existing fill materials, as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 2,500 square feet of compacted fill in the building areas and 5,000 square feet in pavement areas. Where not specified by local ordinance, one density and water content test should be performed for every 100 linear feet of compacted utility trench backfill and a minimum of one test performed for every 12 vertical inches of compacted backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. If unanticipated conditions are observed, the Geotechnical Engineer should recommend improvement options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Wet Weather Earthwork

The suitability of soils used for structural fill depends primarily on their grain-size distribution and moisture content when they are placed. As the fines content (the soil fraction passing the U.S. No. 200 Sieve) increases, soils become more sensitive to small changes in moisture content. Soils containing more than about 5 percent fines (by weight) cannot be consistently compacted to a firm, unyielding condition when the moisture content is more than 2 percentage points above or below optimum. Optimum moisture content is the moisture content at which the maximum dry density for the material is achieved in the laboratory by the ASTM D1557 test procedure.

The near-surface soils have variable fines content based on our visual observations and lab testing and are considered moisture sensitive. If inclement weather or in situ soil moisture content prevents the use of on-site material as structural fill, we recommend use of materials specified in **Fill Material Types** for free-draining granular fill.

Stockpiled soils should be protected with polyethylene sheeting anchored to withstand local wind conditions and preservation of the soil's moisture content.

Ground Improvement

Liquefaction hazards for single-story buildings can often be mitigated through ground improvement rather than deep foundations. Terracon conducted a preliminary consultation with a ground improvement contractor. Ground improvement via aggregate piers that densify and stiffen the ground is likely to be the most cost-efficient method. However, the ground located outside the aggregate pier improvements will be subject to liquefaction-induced settlement. Therefore, we recommend any utilities connected to the proposed structures be designed with flexible connections to reduce damage during a seismic event.

Per the preliminary consultation, aggregate piers beneath and around the building foundations appear to be feasible to limit post liquefaction displacements and potential bearing failure. Assuming damage to the floor slab is acceptable, ground improvement beneath and around the foundation elements may suffice. However, given the small footprint of the proposed building, aggregate piers beneath the entire building footprint may be considered to help reduce costs.

It is recommended that Terracon and the structural engineer discuss the computed liquefaction-induced displacements, building tolerances, and ground improvement options. Following those discussions, a ground improvement contractor should be consulted for design of the ground improvement system. If desired, Terracon can provide a list of local ground improvement design build contractors. Recommendations for spread footings with grade beams are provided in the **Shallow Foundations** section.

Shallow Foundations

Shallow foundations are feasible with installation of ground improvement. Mat foundations perform better than spread footing foundations in lateral spreading ground, but footings with interconnected grade beams may also meet the seismic performance objective. The following design parameters are considered preliminary until Terracon discusses foundation options with the structural engineer and the values are confirmed by a ground improvement specialty contractor.

Design Parameters – Compressive Loads

Item	Description
Preliminary Maximum Net Allowable Bearing Pressure [1, 2]	3,000 to 4,000 psf - foundation bearing on Ground Improvement
Minimum Foundation Dimensions	24 inches for Spread Footing 18 inches for Wall Footing
Ultimate Passive Resistance [4] **(equivalent fluid pressures)**	400 pcf (granular backfill)
Sliding Resistance [5]	0.4 ultimate coefficient of friction - granular material
Minimum Embedment below Finished Grade [6]	18 inches
Estimated Total Settlement from Structural Loads [2]	Less than about 1 inch
Estimated Differential Settlement [2, 7]	About 2/3 of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. An appropriate factor of safety has been applied.
2. Values provided are based on the preliminary consultation with ground improvement contractors. Final allowable bearing pressures should be developed by specialty contractor that designs the ground improvement.
3. Unsuitable or soft soils should be overexcavated and replaced per the recommendations presented in **Earthwork**.
4. Passive resistance in the upper 2 feet of the soil profile should be neglected.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Should be neglected for foundations subject to net uplift conditions.

Item	Description

6. For frost protection and to reduce the effects of seasonal moisture variations in the subgrade soils. For perimeter footing and footings beneath unheated areas. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7. Differential settlements are as measured over a span of 50 feet. We should review the settlement estimates after the foundation plan has been prepared by the structural engineer.

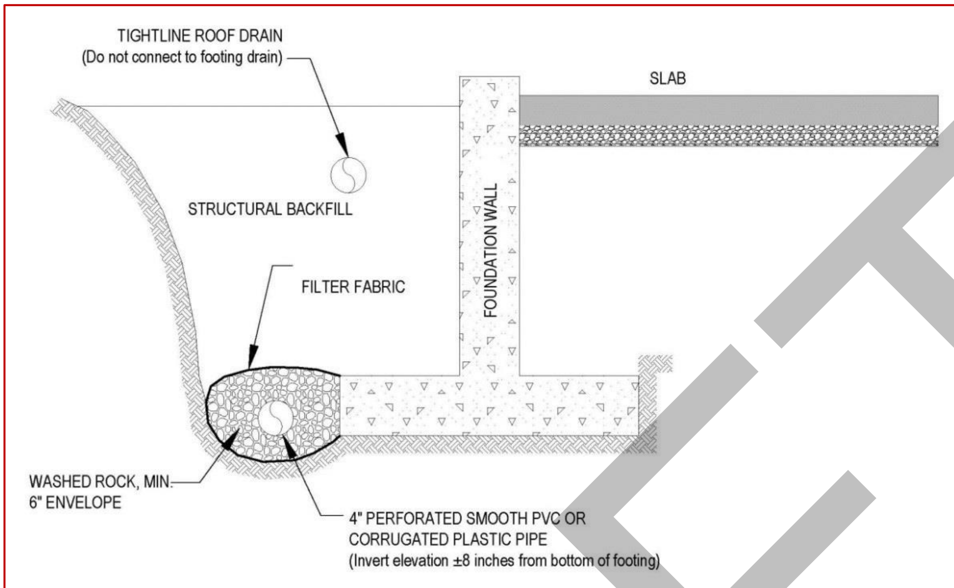
Foundation Construction Considerations

As noted in **Earthwork**, the base of footing excavations should be observed by the Geotechnical Engineer or their representative.

The use of spread footings is only feasible with ground improvement. The ground improvement design drawings typically specify the soils between the aggregate piers is undisturbed and free of organics.

Foundation Drains

We recommend the building be encircled with a perimeter foundation drain to collect exterior seepage water. This drain should consist of a 4-inch diameter perforated pipe within an envelope of washed rock, extending at least 6 inches on all sides of the pipe. The washed rock should conform to WSDOT Section 9-03.12(4), Gravel Backfill for Drains or 9-03.12(5), Gravel Backfill for Drywells. The washed rock envelope should be wrapped with filter fabric meeting the material requirements for Low Survivability Nonwoven with maximum AOS of No. 40 Geotextile for Underground Drainage found in WSDOT Section 9-33.2(1) (such as Mirafi 140N, or equal) to reduce the migration of fines from the surrounding soil. Ideally, the drain invert would be installed no more than 8 inches above or below the base of the perimeter footings. The perimeter foundation drain should not be connected to roof downspout drains and should be constructed to discharge into the site storm water system or other appropriate outlet. These recommendations are summarized in the figure below.



Canopy and Pylon Sign Foundation

Augered and Cast-in-Place (ACIP) piles are anticipated to be feasible as the foundation type for the proposed canopy and pylon sign. However, the depth of liquefiable soils likely exceeds the practical depth of the ACIP piles. If the seismic performance objective of the canopy and pylon sign is no collapse, we recommend that Terracon and the structural engineer further discuss the options.

Augered and Cast-in-Place (ACIP) Pile Design Parameters

The following table can be used to estimate capacities for individual, continuous flight auger piles, commonly referred to as ACIP piles. The values are considered adequate for estimation of allowable (safety factor applied) load carrying capacity for ACIP piles with diameter of 2 feet and ranging in depth from 10 to 30 feet. A factor of safety of 2 and 3, respectively were applied for side friction and end bearing, respectively, for static conditions. ACIP piles should be spaced at least three pile diameters apart (center-to-center) if side friction is used for compressive loads.

ACIP Design Summary [1]

Depth (feet)	Stratigraphy [2]		Allowable Skin Friction (psf) [3]	Allowable End Bearing Pressure (psf) [4]
	Model No.	Material		
0-10	1&2	Silty sand and nonplastic silt	--	--
10-30	3	Sand with variable silt content	150	250

1. Design capacities are dependent upon the method of installation and quality control parameters. The values provided are estimates and should be verified after finalization of installation protocol.
2. See Subsurface Profile in **Geotechnical Characterization** for more details on stratigraphy.
3. Applicable for compressive loading only. Reduce to 2/3 of values shown for uplift loading. The effective weight of the pile can be added to uplift load capacity to the extent permitted by IBC.
4. Minimum embedment of the piles should be 10 feet.

ACIP piles should be spaced at least three pile diameters apart (center-to-center) if side friction is used for compressive loads. For estimating purposes, the embedment of the foundations should be minimum 10 feet.

ACIP Pile Lateral Loading

The following table lists input values for use in LPILE analyses. Modern versions of LPILE provide estimated default values of k_h and E_{50} based on strength and are recommended for the project. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters.

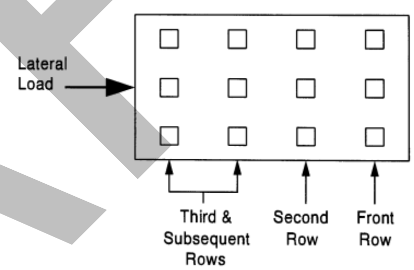
Stratigraphy [1]		L-Pile Soil Model	ϕ [2]	γ' (pcf) [2]	K (pci)	
Depth (ft; bgs)	Model No.				Static	Cyclic
0-10	1&2	Sand (Reese)	30°	100	25	--
10-30	3	Sand (Reese)	30°	36	20	--

1. See Subsurface Profile in **Geotechnical Characterization** for more details on Stratigraphy.
2. Definition of Terms:
   - ϕ: Internal friction angle
   - γ': Effective unit weight
   - K: Horizontal modulus of subgrade reaction

When piles are used in groups, the lateral capacities of the piles in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent pile. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pile foundations within a pile group are as follows:

Center to Center Pile Spacing [1,2]	P-Multiplier, P_m [3]		
	Front Row	Second Row	Third and Subsequent Rows
3B	0.8	0.4	0.3
4B	0.9	0.65	0.5
5B	1.0	0.85	0.7
6B	1.0	1.0	1.0

1. Spacing in the direction of loading. B = pile diameter
2. For the case of a single row of piles supporting a laterally loaded grade beam, group action for lateral resistance of piles would need be considered when spacing is less than three pile diameters (measured center-to-center).
3. See adjacent figure for definition of front, second and third rows.



ACIP Pile Construction Considerations

Installation of adjacent piles with a clear distance spacing of less than ten pile diameters should be delayed until grout in the initial pile has set to avoid possible grout intrusion between the piles which could jeopardize pile integrity.

Proper ACIP pile installation is highly operator-dependent and requires a greater than average dependence on quality workmanship and quality control monitoring. In addition, the successful ACIP pile completion largely depends on the equipment and installation procedures. The auger should be withdrawn in a controlled manner and a sufficient head of grout should always be maintained in the augers to prevent necking of fluid grout due to hydrostatic pressures.

If practical drilling refusal is experienced above the planned termination depth, then a boulder or other obstruction may be present, and a replacement pile should be installed. The situation should be evaluated by the Geotechnical Engineer and the Structural Engineer during the pile installation. Continued "hard" drilling to attempt to extend through an obstruction should not be performed due to the possibility of excessive soil removal.

The ACIP pile installation process should be performed under observation of the Geotechnical Engineer. The Geotechnical Engineer should document the pile installation process including soil and groundwater conditions observed, consistency with expected conditions, and details of the installed pile.

Floor Slabs

Unsuitable soils (e.g., existing fill, very loose/soft native soils) may be encountered at the floor slab subgrade level. These soils should be replaced with compacted structural fill, so the floor slab is supported on at least 2 feet of compacted structural fill.

Design parameters for floor slabs assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Floor Slab Design Parameters

Item	Description
Floor Slab Support [1]	Minimum 6 inches of free-draining of either of the following: ▪ Washed drain rock ▪ 9-03.12(1)A *Gravel Backfill for Foundations Class A* (compacted to at least 95% of ASTM D 1557) [2, 3]
Estimated Modulus of Subgrade Reaction [4]	2 feet of compacted structural fill: ▪ 180 pounds per square inch per inch (psi/in) for point loads ▪ 100 psi/in for distributed loads Preliminary design parameters with ground improvement: ▪ 320 pounds per square inch per inch (psi/in) for point loads ▪ 170 psi/in for distributed loads

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. WSDOT Standard Specification.
3. The floor slab design should include a capillary break, comprised of compacted material with less than 12% passing the No. 40 sieve and less than 5% fines (material passing the No. 200 sieve).
4. Modulus of subgrade reaction is an estimated for subgrade conditions where 2-foot replacement via structural fill or ground improvement with aggregate piers under the floor slab.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, when the project includes humidity-controlled areas, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut contraction joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations, refer to the ACI Design Manual. Joints or cracks should be sealed with a waterproof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond the floor slab, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

Pavements

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs noted in this section must be applied to the site which has been prepared as recommended in the **Earthwork** section.

Pavement Design Parameters

A California Bearing Ratio (CBR) of 5 was used for the subgrade for the asphaltic concrete (AC) pavement designs. A modulus of subgrade reaction of 150 pci was used for the Portland Cement Concrete (PCC) pavement designs. The value was empirically derived based upon our experience with the observed subgrade soils and our expectation of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**. A modulus of rupture of 550 psi was used in design for the concrete (based on correlations with a minimum 28-day compressive strength of 4,000 psi).

Based on our expectation that the parking area will be subjected to automobile traffic only and that the drive areas will be subjected to a maximum of five delivery trucks/trash collection trucks per week, it is our opinion that Chick-fil-A's minimum pavement sections noted below are acceptable for this site.

Pavement Section Thicknesses

The following table provides our opinion of minimum thickness for pavement sections:

Layer	Pavement Section Thickness (inches)				
	Sections	Asphaltic Concrete		Portland Cement Concrete	Aggregate Base Course[3]
		Surface Course [2]	Binder Course[4]		
All areas	ACC	1.5	1.5	---	8
Dumpster pad	PCC	---	---	5	4

1. May vary based on observations following proof-rolling.
2. Aggregates for asphalt surface meeting WSDOT: 9-03.8(2) ½-inch HMA requirements.
3. Aggregate base meeting WSDOT:9-03.9(3) Base Course specifications, and the requirements specified in the **Earthwork** section.
4. PG58H-22 asphalt binder.

Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections. Edge restraints (i.e. concrete curbs or aggregate shoulders) should be planned along curves and areas of maneuvering vehicles.

Proper joint spacing will also be required to prevent excessive slab curling and shrinkage cracking. Joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer. PCC pavement details for joint spacing, joint reinforcement, and joint sealing should be prepared in accordance with ACI 330 and ACI 325.

Where practical, we recommend early-entry cutting of crack-control joints in PCC pavements. Cutting of the concrete in its "green" state typically reduces the potential for micro-cracking of the pavements prior to the crack control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Openings in pavements, such as decorative landscaped areas, are sources for water infiltration into surrounding pavement systems. Water can collect in the islands and migrate into the surrounding subgrade soils thereby degrading support of the pavement. Islands with raised concrete curbs, irrigated foliage, and low permeability near-surface soils are particular areas of concern. The civil design for the pavements with these conditions should include features to restrict or collect and discharge excess water from the islands. Examples of features are edge drains connected to the stormwater collection system, longitudinal subdrains, or other suitable outlets and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Based on the possibility of shallow and/or perched groundwater, we recommend installing a pavement subdrain system to control groundwater, improve stability, and improve long-term pavement performance.

We recommend at least 8 inches of free-draining granular material be placed beneath the pavements. The use of a free draining granular base will also reduce the potential for frost action. We recommend pavement subgrades be crowned at least 2% to promote the flow of water towards the subdrains, and to reduce the potential for ponding of water on the subgrade.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic upkeep should be anticipated. Preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Pavement care consists of both localized (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Additional engineering consultation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur, and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to paved areas should slope down from the edges at a minimum 2%.
- Subgrade and pavement surfaces should have a minimum 2% slope to promote proper surface drainage.
- Install pavement drainage systems surrounding areas anticipated for frequent wetting.
- Install joint sealant and seal cracks immediately.
- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Attachments

Exploration and Testing Procedures

Field Exploration

Number of Borings	Approximate Boring Depth (feet)	Location
2	26½	Proposed canopies
1	16½	Proposed pylon sign
1	16½	Proposed infiltration area
3	11½ to 16½	Pavement areas
Number of CPT	**Approximate CPT Depth (feet)**	**Location**
2	73½ to 77	Proposed building area

Exploration Layout and Elevations: Terracon personnel provided the exploration layout using handheld GPS equipment (estimated horizontal accuracy of about ±20 feet) and referencing existing site features. Approximate ground surface elevations were estimated using Google. If elevations and a more precise boring layout are desired, we recommend explorations be surveyed.

Subsurface Exploration Procedures: We advanced the borings with a excavator-mounted rotary drill rig using continuous hollow stem flight augers. Four samples were obtained in the upper 10 feet of each boring with additional samples obtained at intervals of 5 feet thereafter. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. We observed and recorded groundwater levels during sampling. For safety purposes, all borings were backfilled with bentonite after their completion. Pavements were patched with pre-mixed concrete.

We also observed the boreholes at the completion of drilling for the presence of groundwater. The groundwater levels are shown on the attached boring logs.

The sampling depths, penetration distances, and other sampling information were recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer or geologist. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D1140 Standard Test Method for determining the Amount of Material Finer than 75-µm (No.200) Sieve in Soils by Washing

ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The laboratory testing program included observation of soil samples by an engineer or geologist. Based on the results of our field and laboratory programs, we described and classified the soil samples in general accordance with the Unified Soil Classification System.

Photography Log

Photo 1: Asphalt and concrete pavement exposed at the edge of the parking area near MW-03.

Photo 2: Interbedded sand and silt observed in sample between 7½ and 9 feet bgs at boring location B-04.

Photo 3: Thickness of the remnant asphalt in the in the historic gasoline station pavement area.

Photo 4: Monitoring well installed at MW-03.

Site Location and Exploration Plans

Contents:

Note: All attachments are one page unless noted above.

Site Location

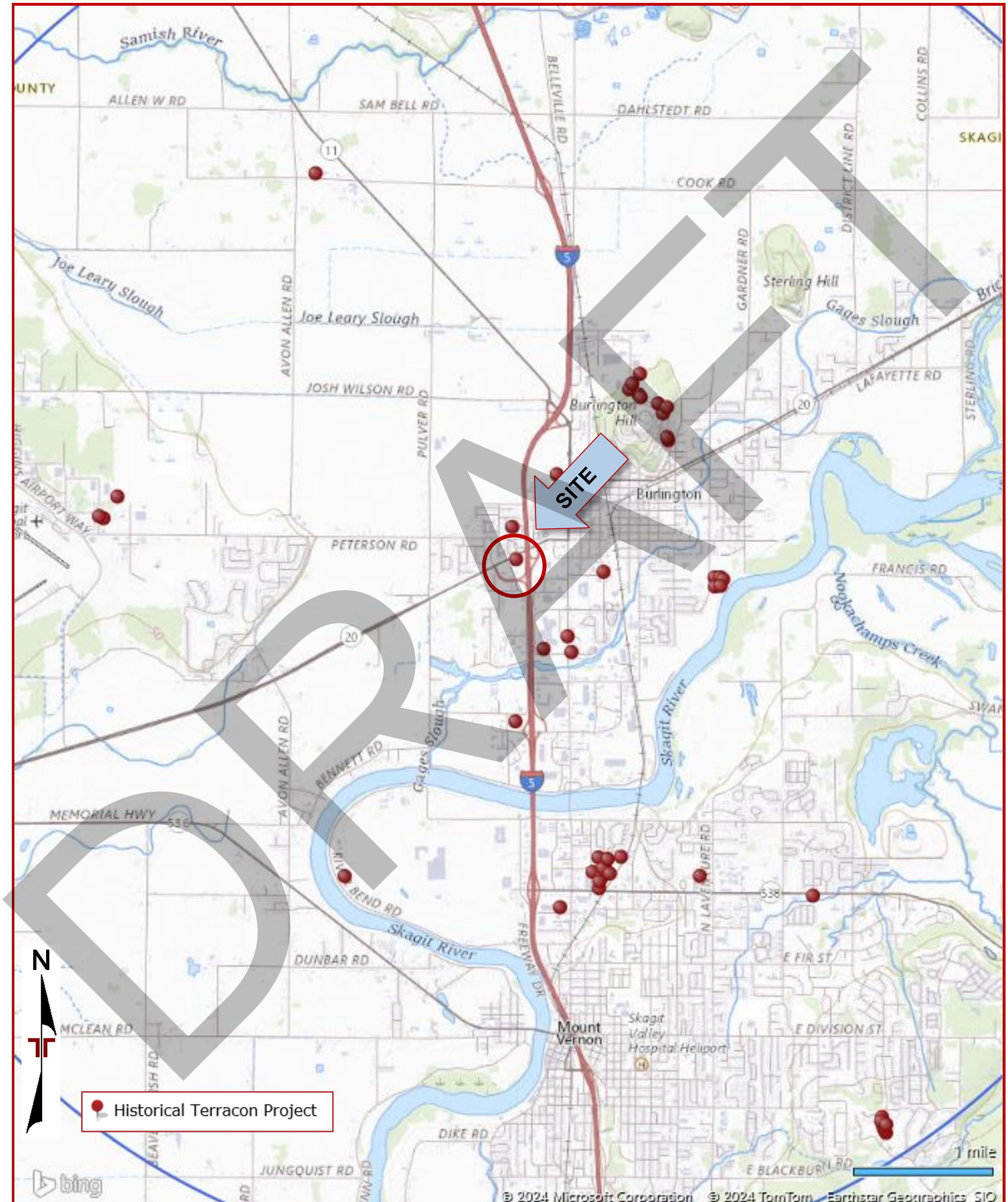


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES
MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan with Project Overlay

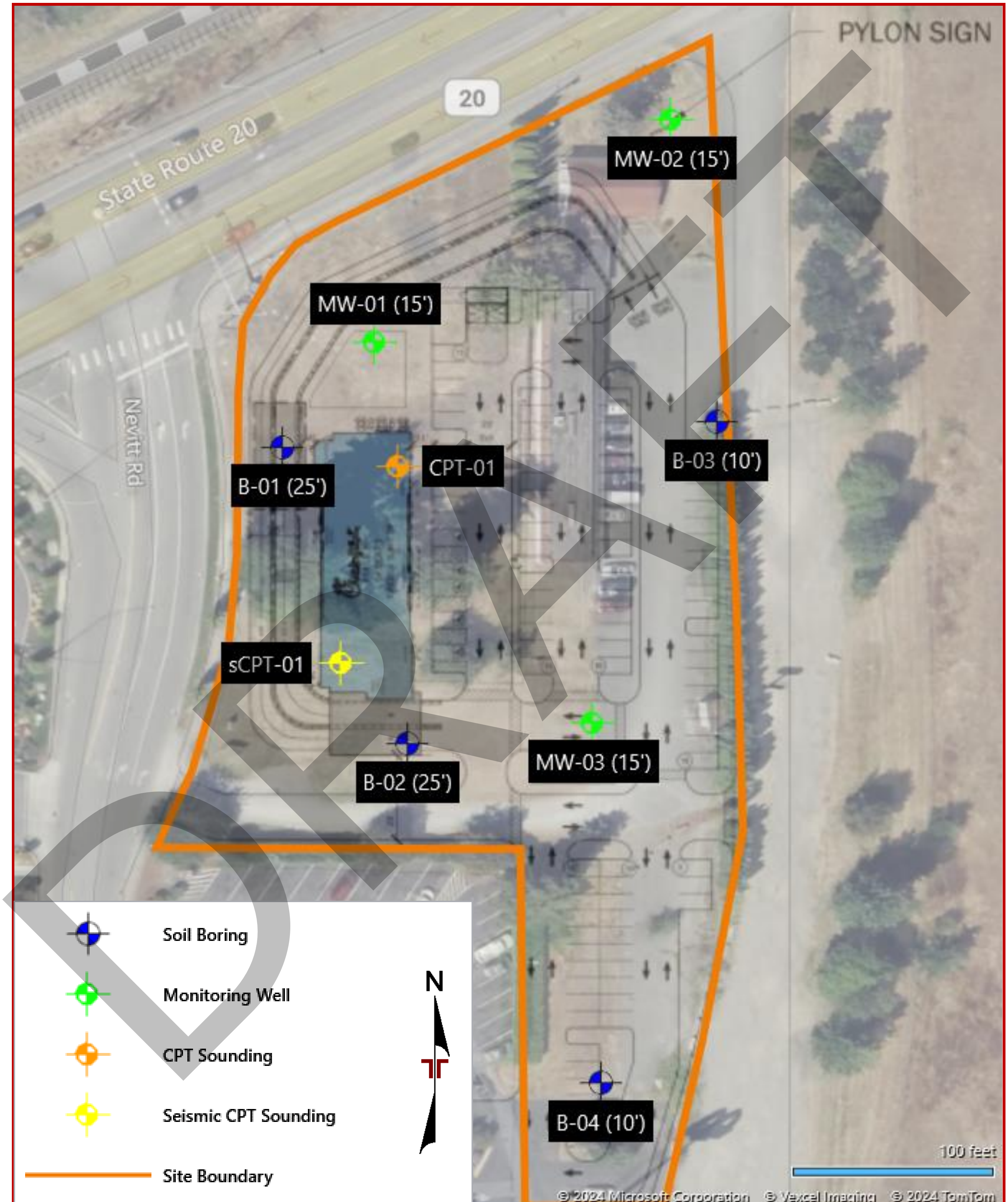


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

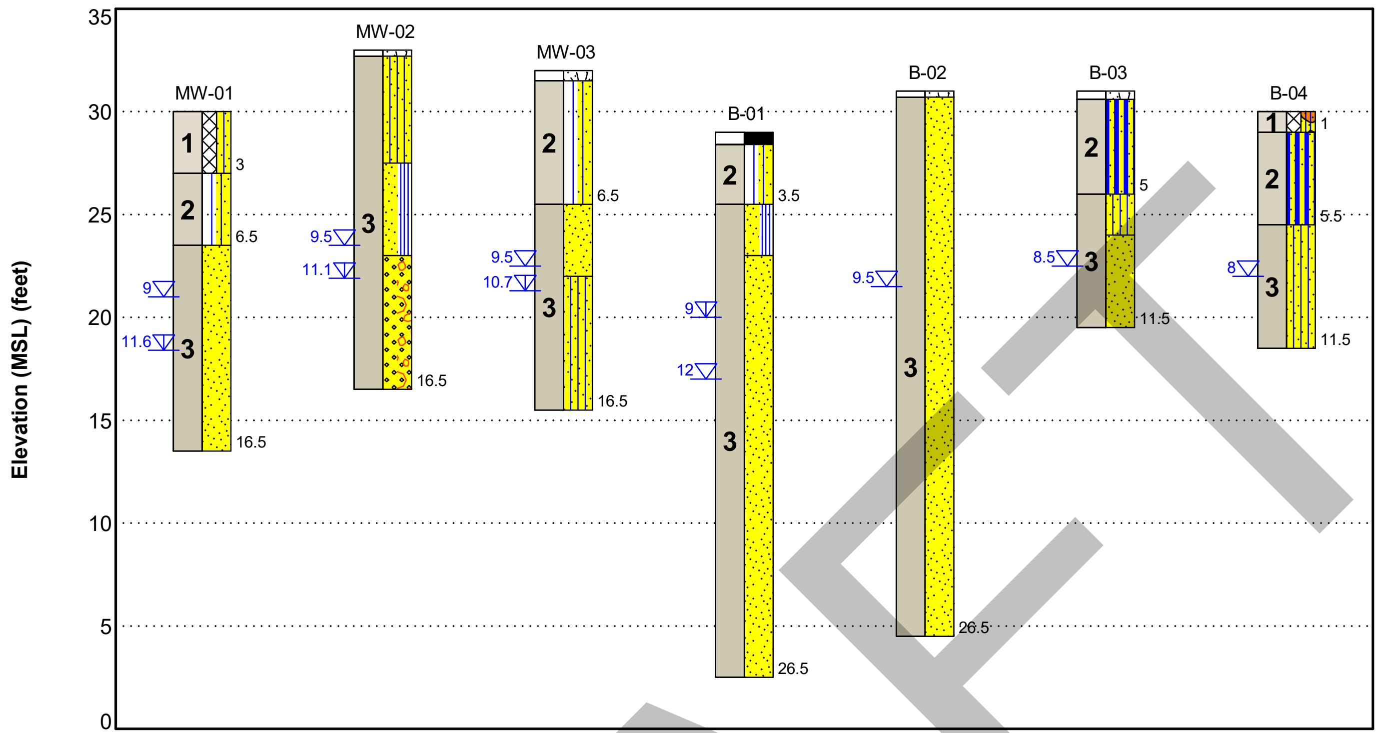
MAP PROVIDED BY MICROSOFT BING MAPS

Exploration and Laboratory Results

Contents:

21905 64th Ave W, Ste 100
Mountlake Terrace, WA

GeoModel



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	FILL	Fill comprised of silty sand and poorly graded gravel with silt and sand, trace organics, loose, damp to moist
2	UPPER ALLUVIUM A	Silt with variable sand content, occasional trace organics, nonplastic, brown to brownish gray, damp to moist, soft to very stiff
3	UPPER ALLUVIUM B	Sand with variable silt and gravel content, damp to wet, grayish brown to gray, generally fine to medium grained, very loose to medium dense, occasional wood debris present

LEGEND

- Silty Sand
- Silt with Sand
- Poorly-graded Sand
- Topsoil
- Poorly-graded Sand with Silt
- Well-graded Sand with Gravel
- Asphalt
- Sandy Silt
- Poorly-graded Gravel with Silt and Sand

First Water Observation
Second Water Observation

The groundwater levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project. Numbers adjacent to soil column indicate depth below ground surface.

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. MW-01

Location: See Exploration Plan

Latitude: 48.4698° Longitude: -122.3437°

Depth (Ft.) Elevation: 30 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Installation Details
1	3.0	**FILL - SILTY SAND (SM)**, trace organics, fine to medium grained, grayish brown, damp, loose	27	Concrete; Bentonite
2	6.5	**SILT WITH SAND (ML)**, trace organics, nonplastic, brown, damp to moist, medium stiff to stiff at ~5 feet: increase in sand and moisture content, becomes sandy silt (ML), grayish brown	23.5	Bentonite
3	16.5	**POORLY GRADED SAND (SP)**, fine to medium grained, gray, moist, loose at ~9 feet: becomes wet at ~10 feet: grain size increasing with depth at ~11.5 feet: trace gravel at ~15 feet: becomes fine to coarse grained	13.5	Sand; Screen (Slot Size 0.01 inches)

Boring Terminated at 16.5 Feet

Sample ID	Recovery (In.)	Field Test Results	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
S-1	13	4-4-4 N=8	27.2	NP	74
S-2	12	1-2-3 N=5			
S-3	15	3-4-3 N=7			
S-4	12	2-3-3 N=6			
S-5	18	1-5-4 N=9	19.5		0

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

Surface conditions: bare ground and sparce vegetation with abundant anthropogenic debris

Water Level Observations

Inferred from change in sample moisture

Measured 1 hour after drilling

Advancement Method
Hollow Stem Auger

Abandonment Method
Monitoring well installed with 2-inch PVC. Well ID: BPG 848

Drill Rig
Excavator-Mounted Rig EC5

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. MW-02

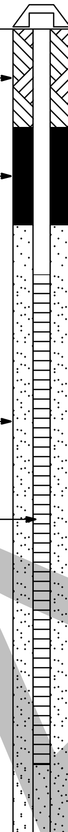
Location: See Exploration Plan

Latitude: 48.4701° Longitude: -122.3431°

Depth (Ft.) Elevation: 33 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Installation Details
3	0.3	**SILT (ML)**, with organics, dark brown, moist, ~4 inches of topsoil with abundant fine roots	32.7	Concrete
	5.5	**SILTY SAND (SM)**, trace organics (fine roots), fine grained, grayish brown, damp, loose increasing silt content with depth	27.5	Bentonite
	10.0	**POORLY GRADED SAND WITH SILT (SP-SM)**, fine grained, brownish gray, damp to moist, loose at ~8 feet: iron oxidation present at ~9.5 feet: becomes wet, increasing grain size with depth	23	Sand Screen (Slot Size 0.01 inches)
	16.5	**WELL GRADED SAND WITH GRAVEL (SW)**, fine to coarse grained, brownish gray, wet, loose at ~15 feet: trace gravel	16.5	

Boring Terminated at 16.5 Feet



Sample ID	Recovery (In.)	Field Test Results	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
S-1	14	1-2-4 N=6	14.8		21
S-2	18	2-2-2 N=4			
S-3	13	2-3-3 N=6			
S-4	10	4-4-5 N=9	9.5		2
S-5	18	2-2-2 N=4			

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
- Inferred from change in sample moisture
- Measured 1 hour after drilling
- borehole caves in due to loose soils

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

fine roots: root diameter < 1/8-inch

Surface conditions: moderately vegetated with grasses

Advancement Method
Hollow Stem Auger

Abandonment Method
Monitoring well installed with 2-inch PVC. Well ID: BPG 817

Drill Rig
Excavator-Mounted Rig EC5

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. MW-03

Location: See Exploration Plan

Latitude: 48.4693° Longitude: -122.3433°

Depth (Ft.) Elevation: 32 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Installation Details
	0.5	**SILT (ML)**, with organics, dark brown, moist, ~6 inches of topsoil with abundant fine roots	31.5	Concrete
2	6.5	**SILT WITH SAND (ML)**, nonplastic, brown, damp to moist, soft	25.5	Bentonite
3	10.0	**POORLY GRADED SAND (SP)**, fine to medium grained, brownish gray, moist, very loose at ~7 feet: increase in silt with depth, becomes poorly graded sand with silt (SP-SM) at ~9 feet: becomes wet	22	Sand
3	16.5	**SILTY SAND (SM)**, fine to medium grained, brownish gray, wet, loose	15.5	Screen (Slot Size 0.01 inches)

Boring Terminated at 16.5 Feet

Depth (Ft.)	Water Level Observations	Sample Type	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
2.5–4		Split spoon	17	1-2-1 N=3	S-1	10.0	NP	
5–6.5		Split spoon	11	1-1-1 N=2	S-2			
7.5–9		Split spoon	15	1-2-2 N=4	S-3	10.4		4
~9	Inferred from change in sample moisture							
~10.5	Measured 1 hour after drilling							
10–11.5		Split spoon	13	1-2-2 N=4	S-4			
15–16.5		Split spoon	10	2-2-3 N=5	S-5			

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations

Inferred from change in sample moisture

Measured 1 hour after drilling

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

Surface conditions: moderately vegetated with grasses

Advancement Method
Hollow Stem Auger

Abandonment Method
Monitoring well installed with 2-inch PVC. Well ID: BPG 818

Drill Rig
Excavator-Mounted Rig EC55L

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Proposed Chick-fil-A - 05839 Burlington, WA
680 Hwy 20 | Burlington, WA
Terracon Project No. 81245040

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. B-01

Location: See Exploration Plan

Latitude: 48.4697° Longitude: -122.3439°

Elevation: 29 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)
	0.6	**ASPHALT**, ~7 inches of asphalt	28.4
2	3.5	**SILT WITH SAND (ML)**, trace gravel, trace organics (wood debris), nonplastic, brown, damp, stiff	25.5
3	6.0	**POORLY GRADED SAND WITH SILT (SP-SM)**, fine grained, brown to gray brown, damp, loose	23
3	26.5	**POORLY GRADED SAND (SP)**, fine to medium grained, gray brown, damp, very loose to medium dense at ~9 feet: becomes wet at ~15 feet: trace gravel at ~20 feet: increasing grain size with depth, becomes fine to coarse grained	2.5

Boring Terminated at 26.5 Feet

Sample ID	Recovery (In.)	Field Test Results	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
S-1	6	7-6-5 N=11			
S-2	11	3-4-4 N=8			
S-3	9	3-4-4 N=8			
S-4	11	1-1-2 N=3	28.2		4
S-5	18	3-5-6 N=11			
S-6	18	6-7-8 N=15	20.5		2
S-7	18	8-9-9 N=18			

Depth (Ft.): 5, 10, 15, 20, 25

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
- Inferred from CPT
- Inferred from change in sample moisture

Drill Rig
Excavator-Mounted Rig EC55L

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

Advancement Method
Hollow Stem Auger

Abandonment Method
Boring backfilled with bentonite chips upon completion.

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. B-02

Location: See Exploration Plan

Latitude: 48.4693° Longitude: -122.3437°

Depth (Ft.) Elevation: 31 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation	Water Level Observations	Sample Depth (Ft.)	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
3	0.3	**SILT (ML)**, with organics, dark brown, moist, ~4 inches of topsoil with abundant fine roots	30.7								
		POORLY GRADED SAND (SP), fine to medium grained, grayish brown, damp to moist, loose									
					2.5–4	15	2-3-3 N=6	S-1			
		grain size and moisture content increasing with depth			5–6.5	6	2-4-4 N=8	S-2			
		at ~7.5 feet: iron oxidation present			7.5–9	18	3-4-3 N=7	S-3	17.6		3
		at ~9.5 feet: becomes wet, grades to brownish gray		▽ (~9.3)	10–11.5	18	3-3-5 N=8	S-4			
		at ~15 feet: grades to gray			15–16.5	18	3-4-4 N=8	S-5	27.8		3
					20–21.5	6	2-4-4 N=8	S-6			
		between ~21.5 and 26.5 feet: abundant wood debris present, trace gravel									
	26.5		4.5		25–26.5	18	4-3-4 N=7	S-7			

Boring Terminated at 26.5 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations

▽ Inferred from change in sample moisture

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

fine roots: root diameter < 1/8-inch

Surface conditons: moderately vegetated with grasses

Advancement Method
Hollow Stem Auger

Abandonment Method
Boring backfilled with bentonite chips upon completion.

Drill Rig
Excavator-Mounted Rig EC55L

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Proposed Chick-fil-A - 05839 Burlington, WA
680 Hwy 20 | Burlington, WA
Terracon Project No. 81245040

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. B-03

Location: See Exploration Plan

Latitude: 48.4697° Longitude: -122.3431°

Elevation: 31 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation
	0.4	**SILT (ML)**, with organics, dark brown, damp, ~4 inches of topsoil with abundant fine to medium size roots	30.6
2	5.0	**SANDY SILT (ML)**, trace organics (fine to medium size roots), iron oxidation present, nonplastic, grayish brown, damp, stiff to very stiff, iron oxidation present sand content increasing with depth	26
3	7.0	**SILTY SAND (SM)**, fine grained, grayish brown, damp to moist, (medium dense) at ~5 feet: driller notes obstruction, blow counts may be overstated at sample S-2	24
3	11.5	**POORLY GRADED SAND (SP)**, fine to medium grained, grayish brown, moist to wet, medium dense to loose at ~9 feet: grades to gray, trace gravel present	19.5

Boring Terminated at 11.5 Feet

Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
10	8-7-8 N=15	S-1			
13	33-28-35 N=63	S-2	11.1		16
10	8-4-6 N=10	S-3			
18	3-4-5 N=9	S-4			

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
Inferred from change in sample moisture

Notes
Elevation Reference: Elevations approximated from Google Earth
Samples obtained using a 2" O.D. split spoon sampler
Fine roots: root diameter: < 1/8-inch
Medium roots: root diamter < 1/8-inch & > 3/8-inch
Surface conditions: moderately vegetated with grasses and shrubs

Advancement Method
Hollow Stem Auger

Abandonment Method
Boring backfilled with bentonite chips upon completion.

Drill Rig
Excavator-Mounted Rig EC55L

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Boring Log No. B-04

Location: See Exploration Plan

Latitude: 48.4688° Longitude: -122.3433°

Elevation: 30 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
1	1.0	**FILL - POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM)**, trace organics, fine grained, rounded, brown, moist	29			Grab			G-1			
2	5.5	**SANDY SILT (ML)**, nonplastic, grayish brown, damp, stiff, iron oxidation present at ~2.5 feet: PP = 0.75 to 1.00 tsf sand content increasing with depth	24.5			Split spoon	11	5-6-7 N=13	S-1	16.7		56
3		**SAND AND SILT**, brownish gray, damp to moist, interbedded stiff to soft, nonplastic silt (ML) / loose to very loose, fine grained, poorly graded sand (SP) and silty sand (SM)		5		Split spoon	18	3-4-5 N=9	S-2			
		at ~8 feet: becomes wet			▽	Split spoon	15	2-4-5 N=9	S-3			
	11.5	at ~9.5 feet: iron oxidation present at ~10 feet: PP = 0.75 to 1.00 tsf	18.5	10		Split spoon	15	2-2-1 N=3	S-4			

Boring Terminated at 11.5 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
▽ Inferred from change in sample moisture

Drill Rig
Excavator-Mounted Rig EC55L

Hammer Type
Rope and Cathead

Driller
Boretec, Inc.

Logged by
BAS, MTS

Boring Started
04-09-2024

Boring Completed
04-09-2024

Notes

Elevation Reference: Elevations approximated from Google Earth

Samples obtained using a 2" O.D. split spoon sampler

Pocket Penetrometer (PP) used to measure the compressive soil strength in the field in tons per square feet (tsf).

Surface conditions: gravel surfacing and light vegetation

fine roots: root diameter < 1/8-inch

medium roots: root diameter > 1/8-inch & < 3/8-inch

Advancement Method
Hollow Stem Auger

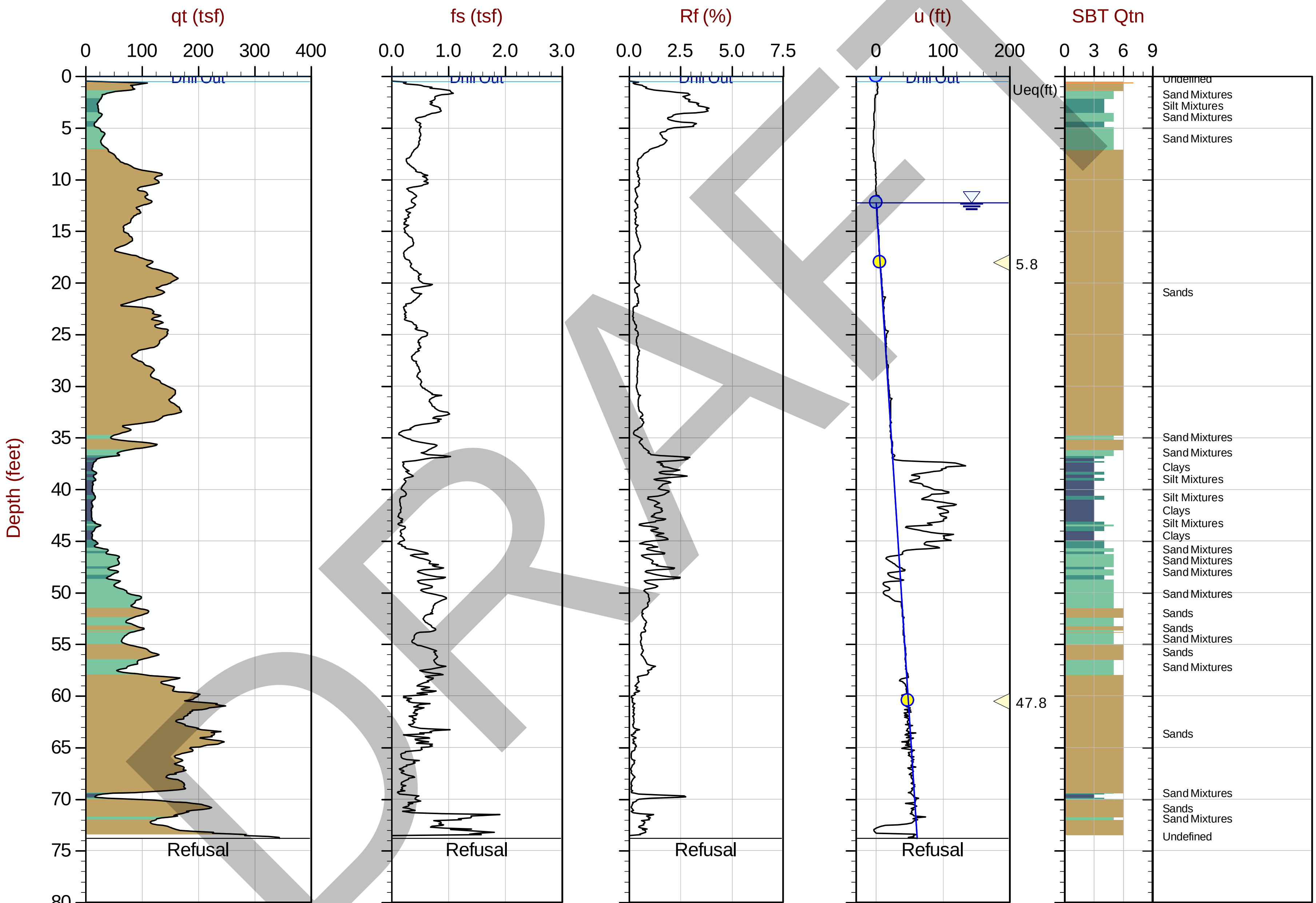
Abandonment Method
Boring backfilled with bentonite chips upon completion.

CONETEC

Terracon

Job No: 24-59-27466
Date: 2024-04-09 12:53
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: CPT-01
Cone: 855:T1500F15U35



Max Depth: 22.500 m / 73.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46966 Long: -122.34368

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

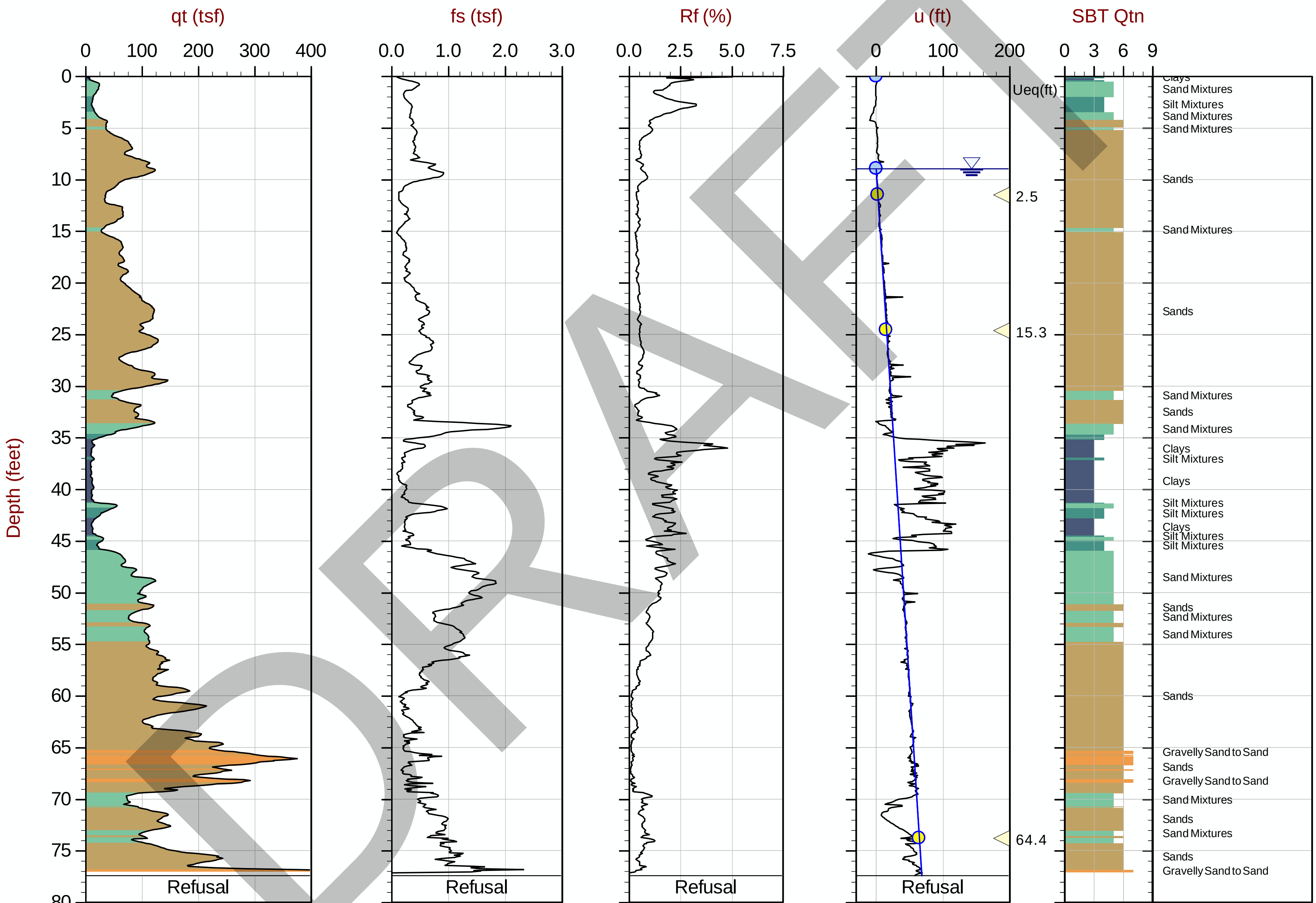
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35



Max Depth: 23.600 m / 77.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46938 Long: -122.34379

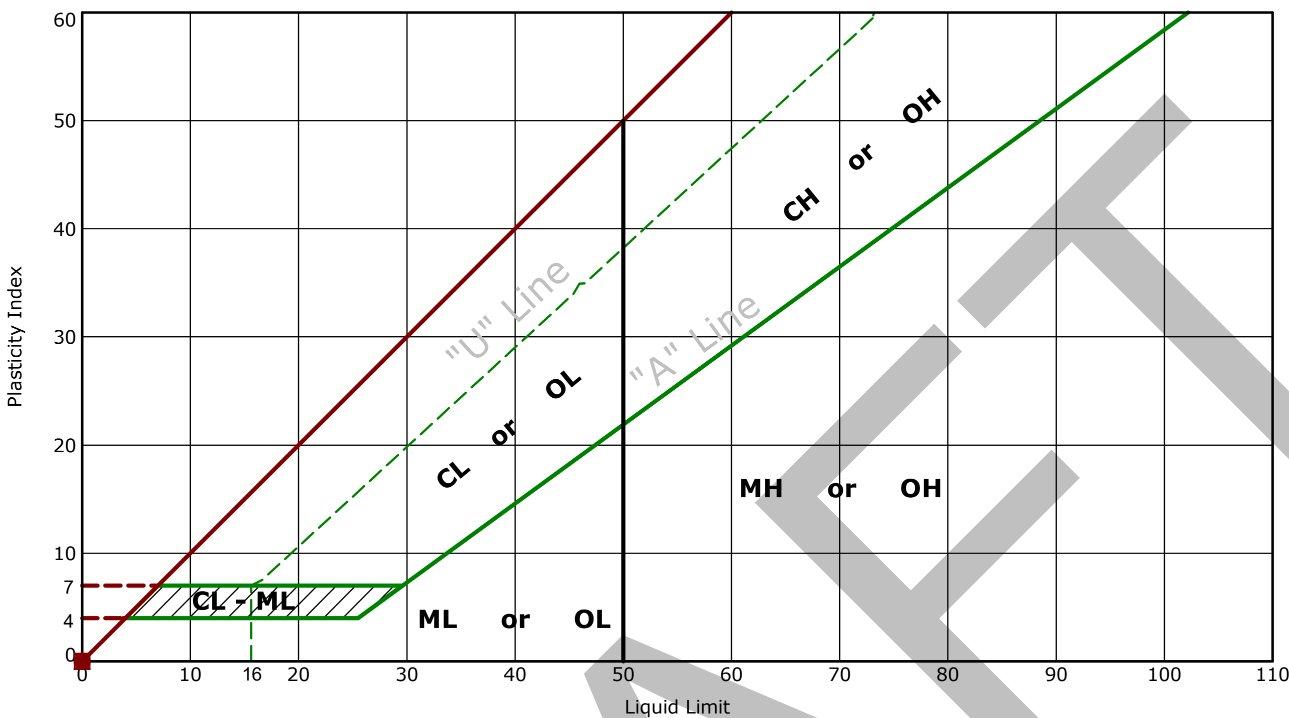
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Atterberg Limit Results

ASTM D4318

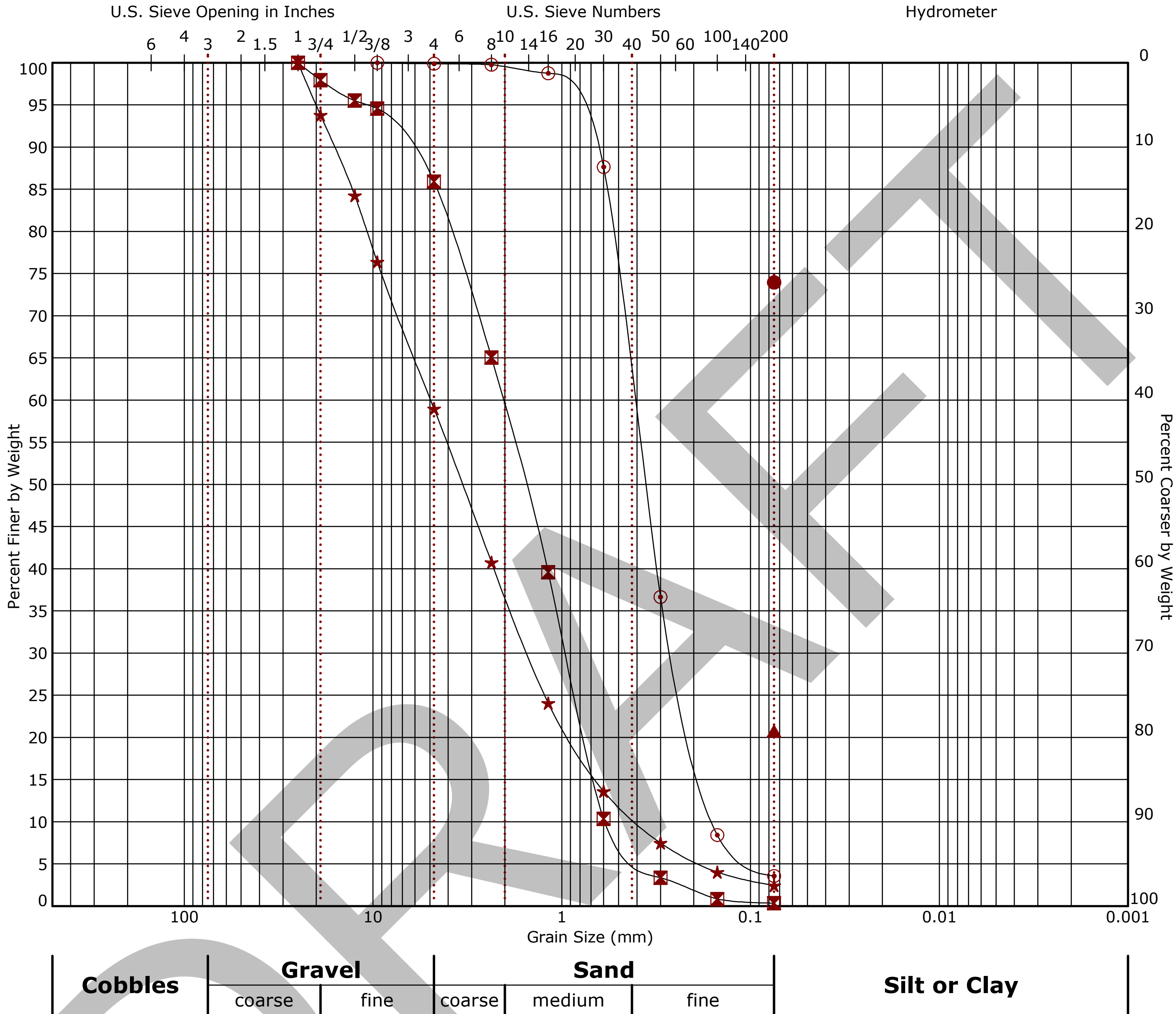


	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	MW-01	2.5 - 4	NP	NP	NP	74.0	ML	SILT with SAND
☒	MW-03	2.5 - 4	NP	NP	NP		ML	SILT with SAND

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	MW-01	2.5 - 4	SILT with SAND	ML	NP	NP	NP		
⊠	MW-01	15 - 16.5	POORLY GRADED SAND	SP				0.75	3.55
▲	MW-02	2.5 - 4	SILTY SAND	SM					
★	MW-02	10 - 11.5	WELL-GRADED SAND with GRAVEL	SW				1.16	12.41
⊙	MW-03	7.5 - 9	POORLY GRADED SAND	SP				1.01	2.64

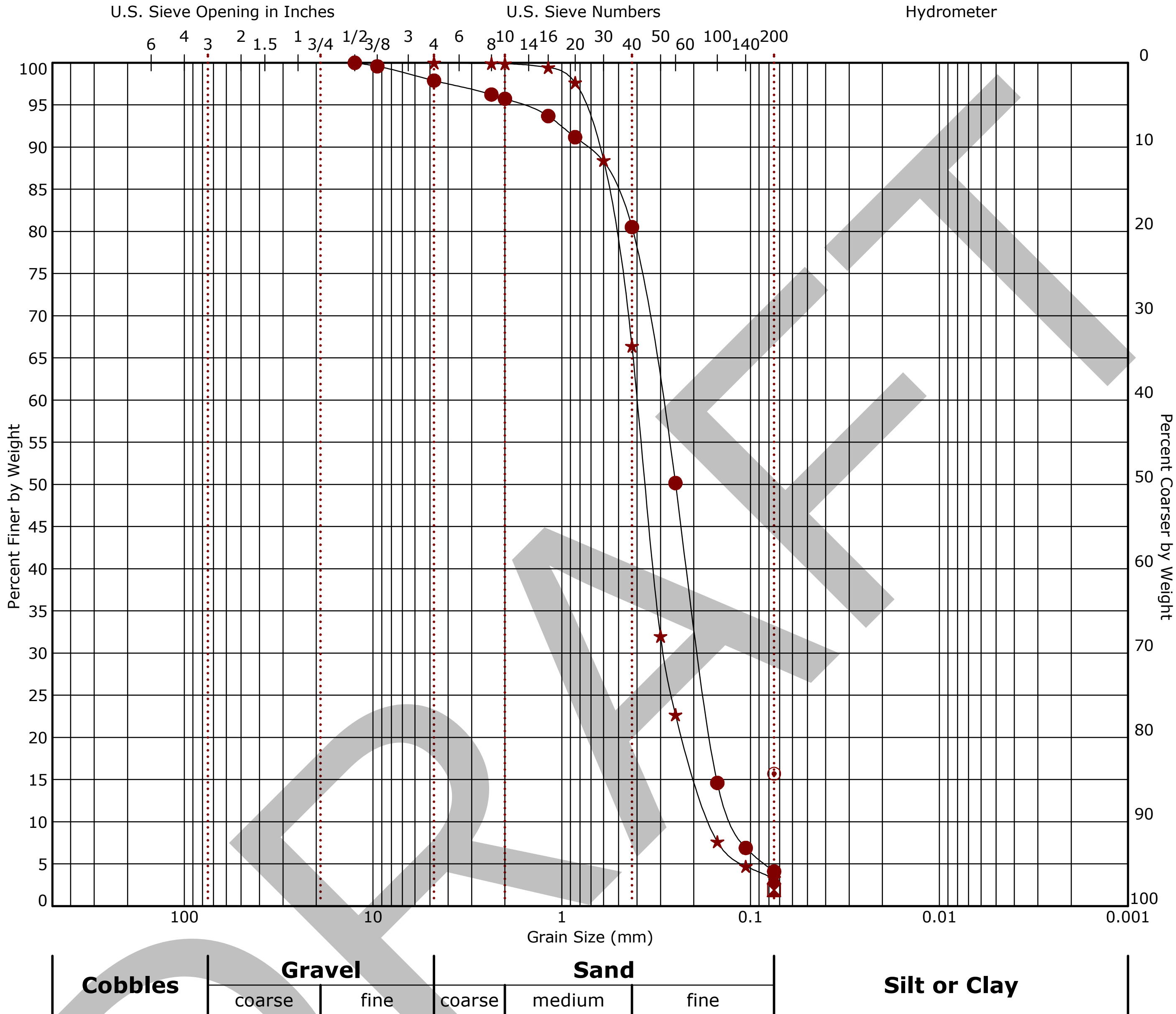
	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	MW-01	2.5 - 4	0.075							74.0		
⊠	MW-01	15 - 16.5	25	2.057	0.945	0.58	0.0	14.1	85.5	0.4		
▲	MW-02	2.5 - 4	0.075							20.7		
★	MW-02	10 - 11.5	25	4.948	1.51	0.399	0.0	41.0	56.5	2.4		
⊙	MW-03	7.5 - 9	9.5	0.412	0.255	0.156	0.0	0.1	96.3	3.6		

Proposed Chick-fil-A - 05839 Burlington, WA
680 Hwy 20 | Burlington, WA
Terracon Project No. 81245040

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Grain Size Distribution

ASTM D422 / ASTM C136



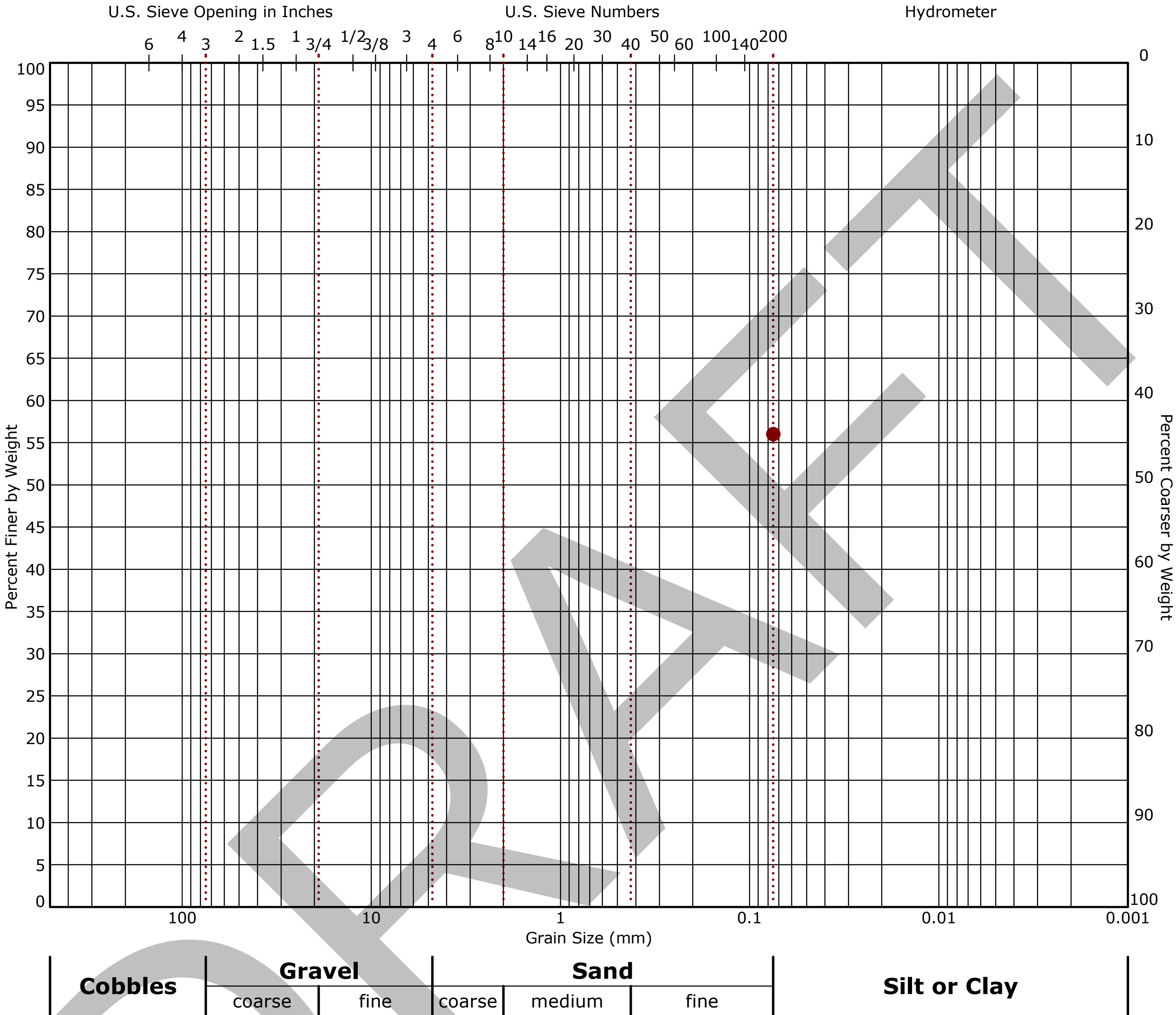
	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	B-01	10 - 11.5	POORLY GRADED SAND	SP				0.97	2.44
⊠	B-01	20 - 21.5	POORLY GRADED SAND	SP					
▲	B-02	7.5 - 9	POORLY GRADED SAND	SP					
★	B-02	15 - 16.5	POORLY GRADED SAND	SP				1.29	2.45
⊙	B-03	5 - 6.5	SILTY SAND	SM					

	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-01	10 - 11.5	12.5	0.297	0.187	0.122	0.0	2.1	93.8	4.1		
⊠	B-01	20 - 21.5	0.075							1.9		
▲	B-02	7.5 - 9	0.075							3.3		
★	B-02	15 - 16.5	4.75	0.398	0.288	0.163	0.0	0.0	96.7	3.3		
⊙	B-03	5 - 6.5	0.075							15.7		

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	B-04	2.5 - 4	SANDY SILT	ML					

	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-04	2.5 - 4	0.075							56.0		

Supporting Information

Contents:

Terracon
21905 64th Ave W, Ste 100
Mountlake Terrace, WA

General Notes

Sampling	Water Level	Field Tests
Grab Sample; Standard Penetration Test	Water Initially Encountered Water Level After a Specified Period of Time Water Level After a Specified Period of Time Cave In Encountered Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	N — Standard Penetration Test Resistance (Blows/Ft.) (HP) — Hand Penetrometer (T) — Torvane (DCP) — Dynamic Cone Penetrometer UC — Unconfined Compressive Strength (PID) — Photo-Ionization Detector (OVA) — Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		Consistency of Fine-Grained Soils (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Relative Density	**Standard Penetration or N-Value (Blows/Ft.)**	**Consistency**	**Unconfined Compressive Strength Qu (tsf)**	**Standard Penetration or N-Value (Blows/Ft.)**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

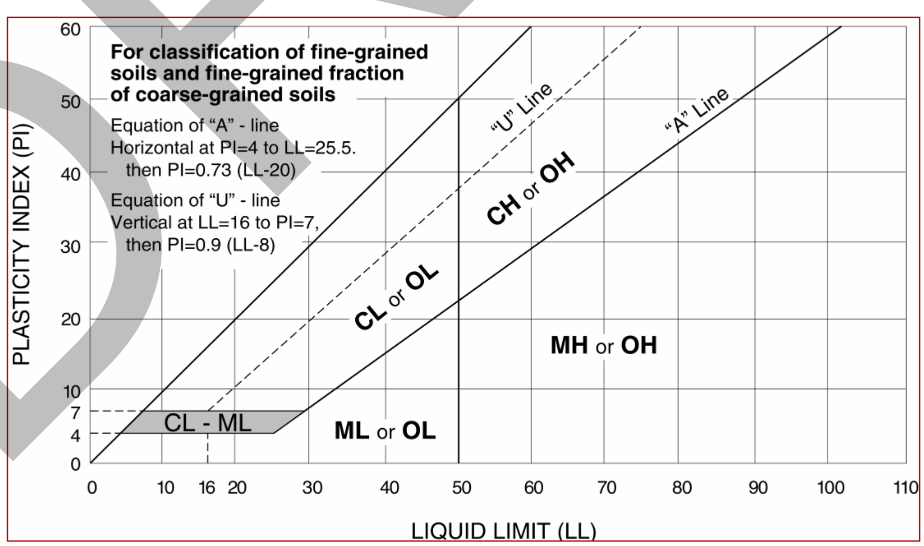
Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



PRESENTATION OF SITE INVESTIGATION RESULTS

Proposed Chick-fil-A, Burlington, WA

Prepared for:

Terracon

ConeTec Job No: **24-59-27466**

Project Start Date: 2024-04-09
Project End Date: 2024-04-09
Release Date: 2024-04-17

Report Prepared by:

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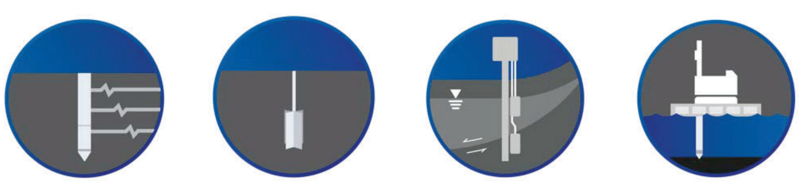

CONETEC

ABOUT THIS REPORT

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

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

Project	
Client	Terracon
Project	Proposed Chick-fil-A, Burlington, WA
ConeTec Project Number	24-59-27466
Test Types	CPTu/SCPTu
Additional Comments	None

Contents

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

- **Site Map**
- **Limitations and Closure**
- **Project Information**
- **Methodology Statements**
- **Report Appendices**

SITE MAP

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

ConeTec Job Number: 24-59-27466

Client: Terracon

Project: Proposed Chick-fil-A, Burlington, WA

Date: 2024-04-17

LIMITATIONS

3rd Party Disclaimer

The “Report” refers to this report titled: Proposed Chick-fil-A, Burlington, WA

The Report was prepared by ConeTec for: Terracon

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: Terracon

The “Report” refers to this report titled: Proposed Chick-fil-A, Burlington, WA

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

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

Closure

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

Report prepared by: MH, JM

PROJECT INFORMATION

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

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

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC921:T1500F15U35	921	15	225	1500	15	35
EC855:T1500F15U35	855	15	225	1500	15	35

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

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

ConeTec

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.

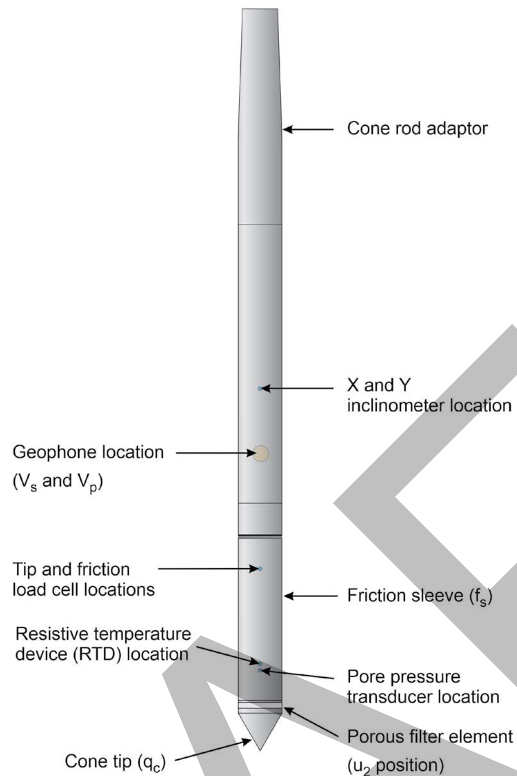

Figure CPTu. Piezocone Penetrometer (15 cm²)

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

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

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

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

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

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

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

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

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

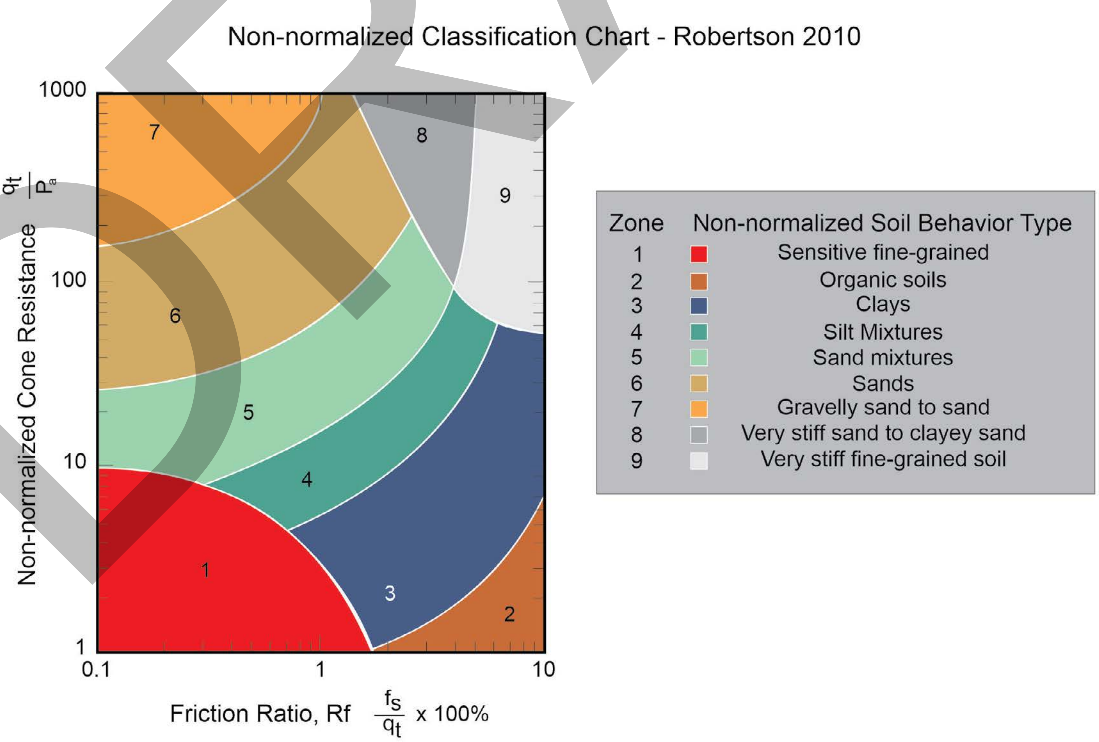

Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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

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

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

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

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

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

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

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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).

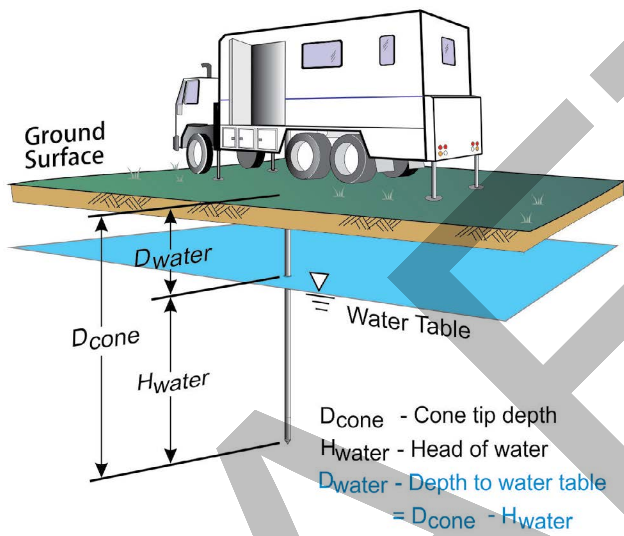

Figure PPD-1. Pore pressure dissipation test setup

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.

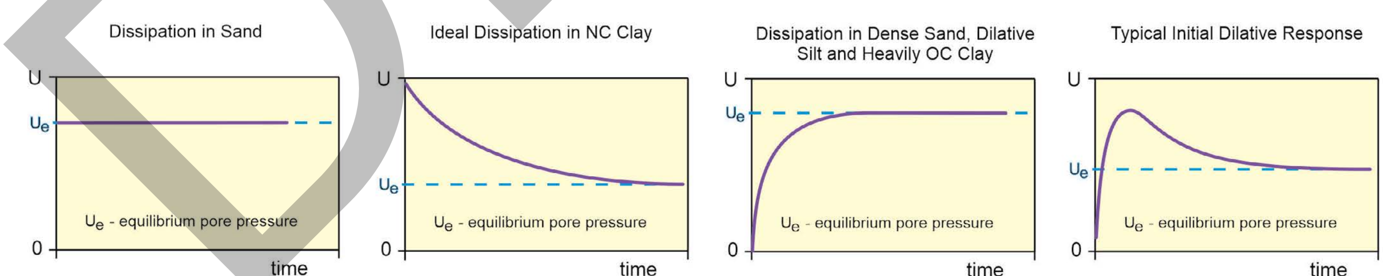

Figure PPD-2. Pore pressure dissipation curve examples

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.

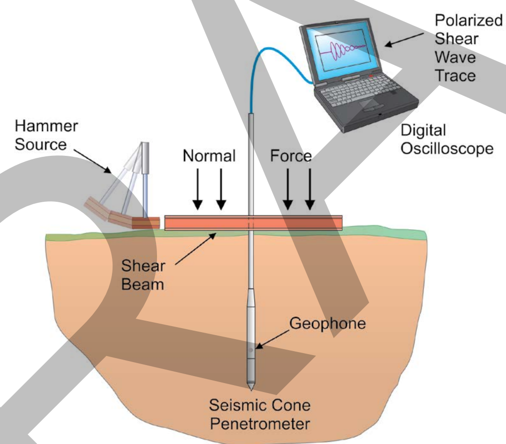

Figure SCPTu-1. Illustration of the SCPTu system

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).

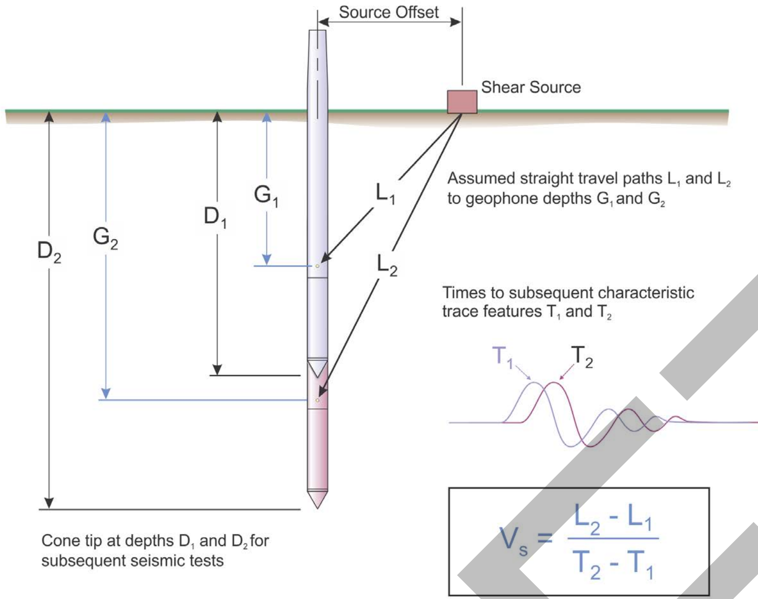

Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

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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**
- **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**
- **Supplementary Documents and Materials**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No:	24-59-27466
Client:	Terracon
Project:	Proposed Chick-fil-A, Burlington, WA
Start Date:	2024-04-09
End Date:	2024-04-09

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Seismic Intervals	Latitude[2]	Longitude[2]	Refer to Notation Number
CPT-01	24-59-27466_CP01	2024-04-09	855:T1500F15U35	15	12.25	73.82		48.46966	-122.34368	
SCPT-01	24-59-27466_SP01	2024-04-09	921:T1500F15U35 855:T1500F15U35	15	8.96	77.43	25	48.46938	-122.34379	3
Totals	2 Soundings					151.24 ft	25			

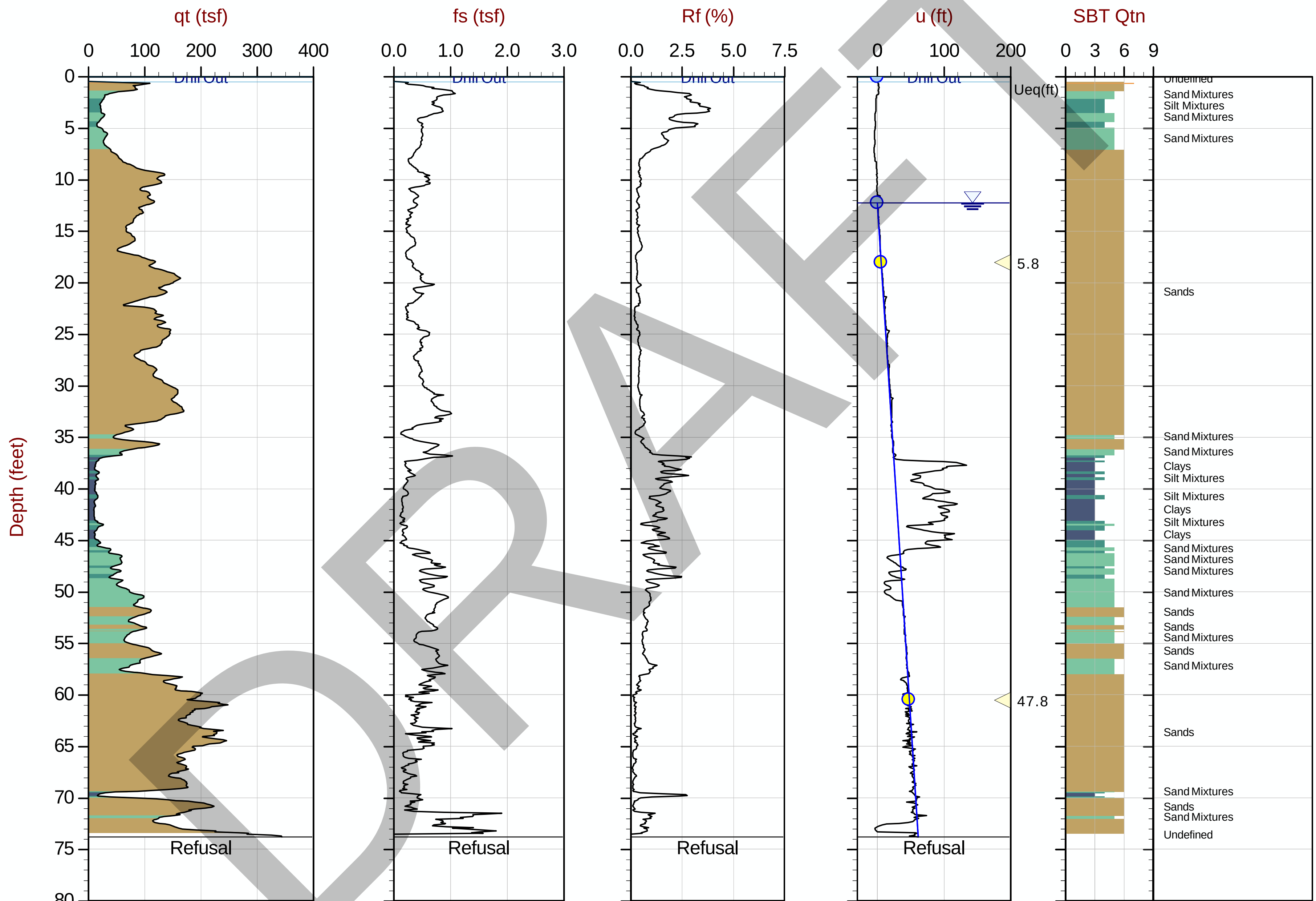
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).
3. Primary cone (EC921) experienced power issues at 70.8ft below surface. Secondary cone (EC855) was deployed to continue collecting data beyond 70.8ft.

ConeTec

Terracon

Job No: 24-59-27466
Date: 2024-04-09 12:53
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: CPT-01
Cone: 855:T1500F15U35



Max Depth: 22.500 m / 73.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46966 Long: -122.34368

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

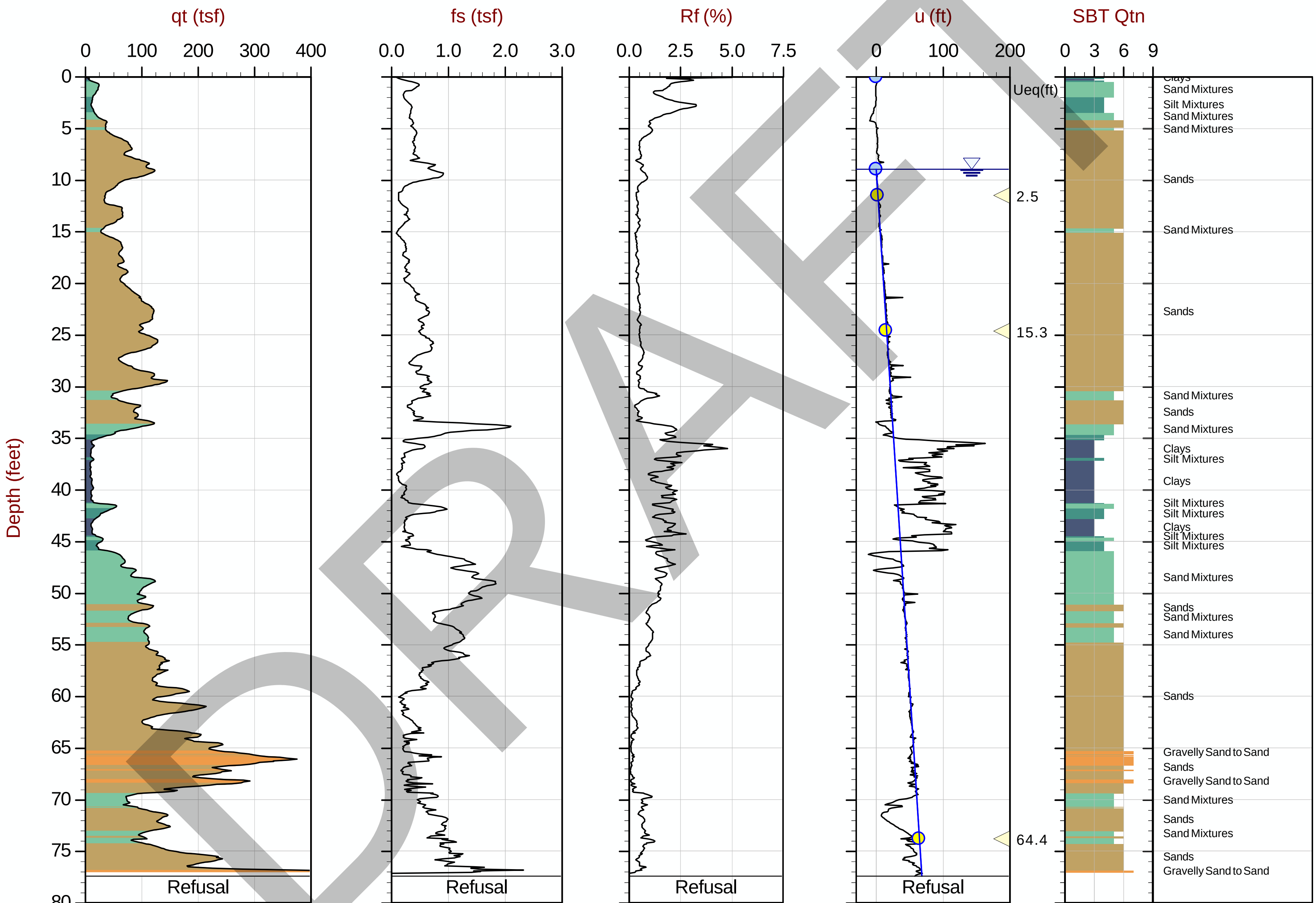
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35



Max Depth: 23.600 m / 77.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46938 Long: -122.34379

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.

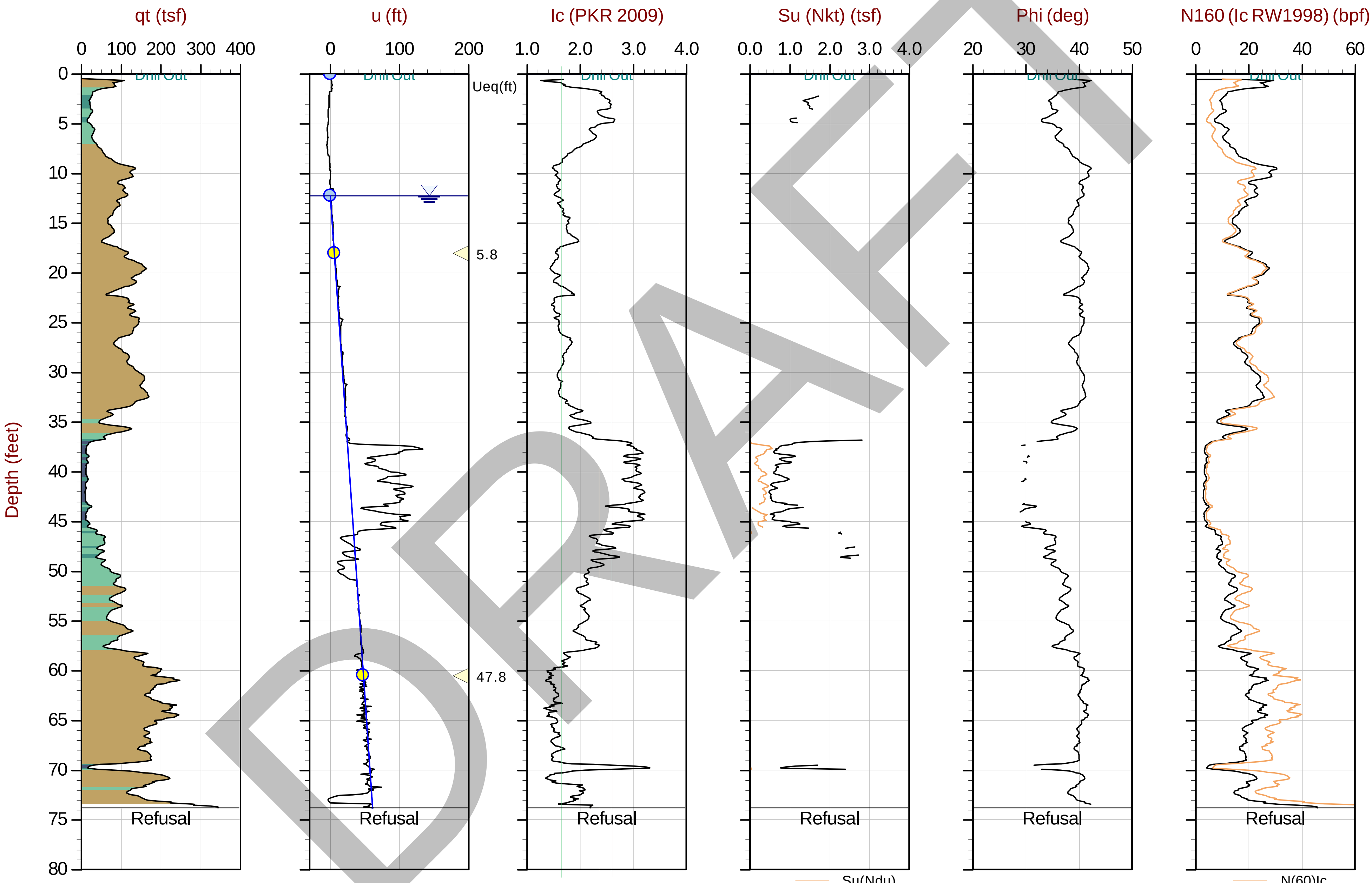
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

ConeTec

ConeTec

Terracon

Job No: 24-59-27466
Date: 2024-04-09 12:53
Site: Proposed Chick-fil-A, Burlington, WA
Sounding: CPT-01
Cone: 855:T1500F15U35



Max Depth: 22.500 m / 73.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46966 Long: -122.34368

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

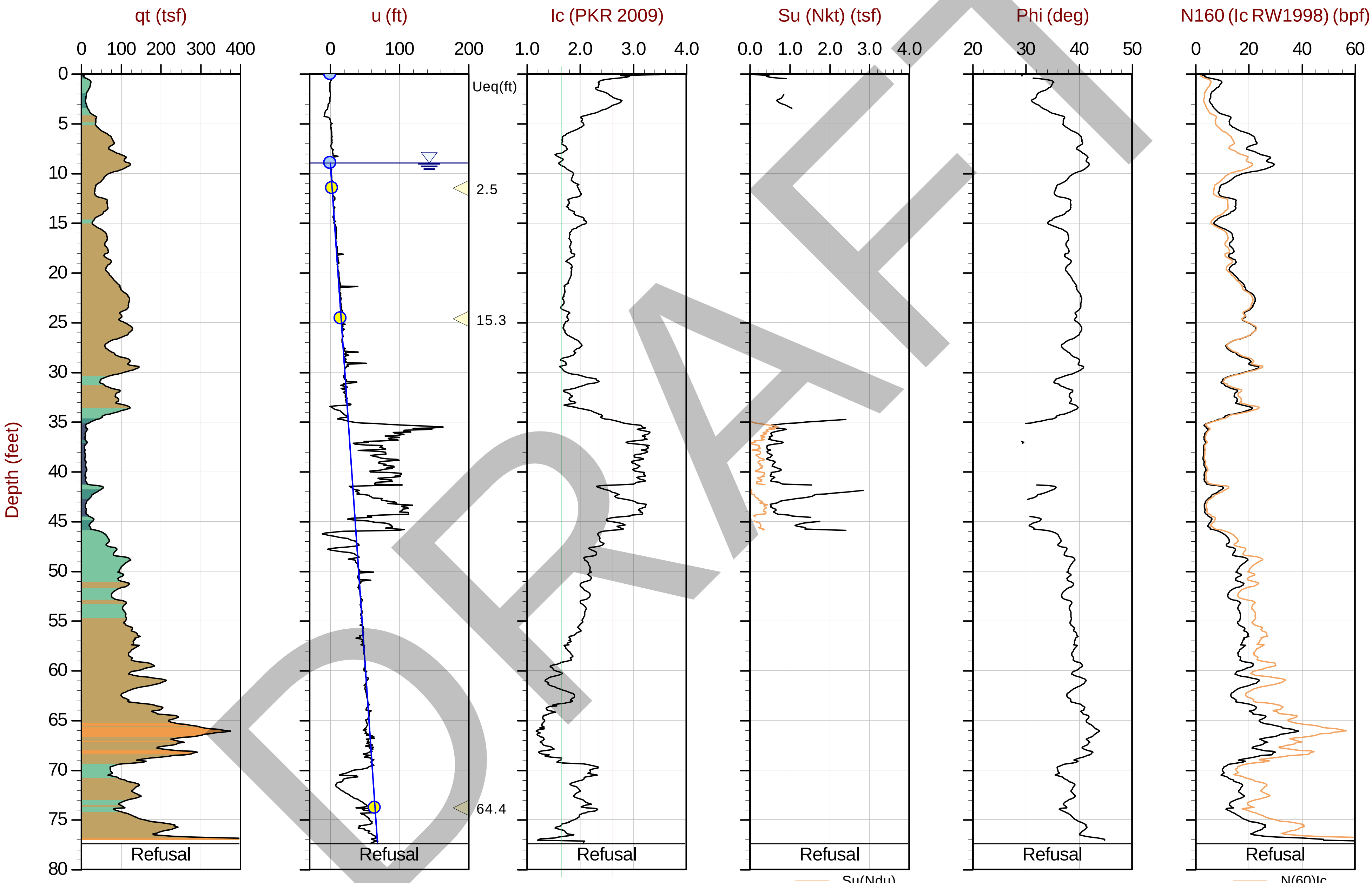
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35



Max Depth: 23.600 m / 77.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46938 Long: -122.34379

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

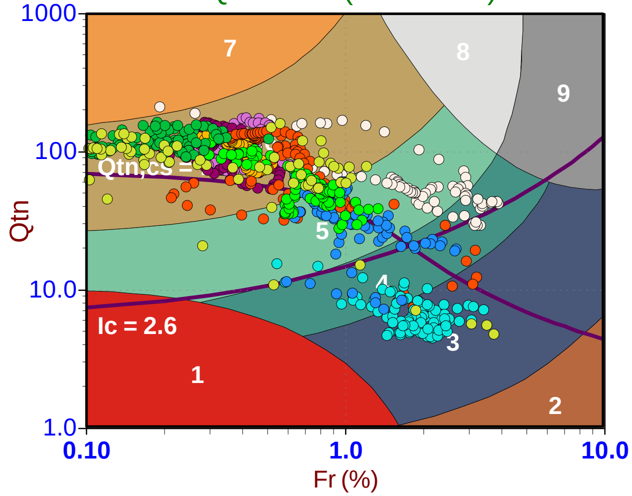
CONETEC

Terracon

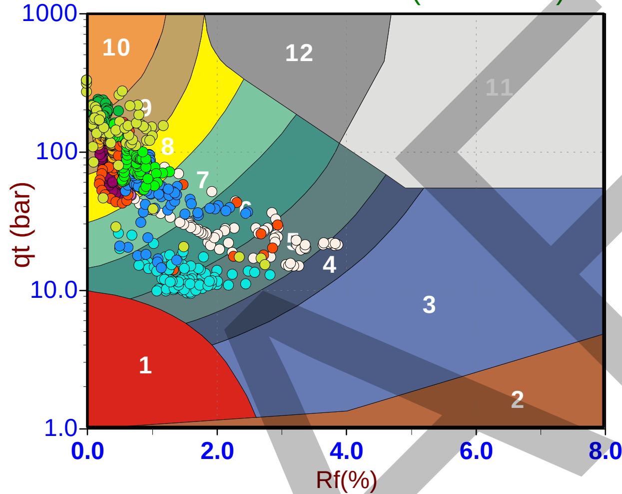
Job No: 24-59-27466
Date: 2024-04-09 12:53
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: CPT-01
Cone: 855:T1500F15U35

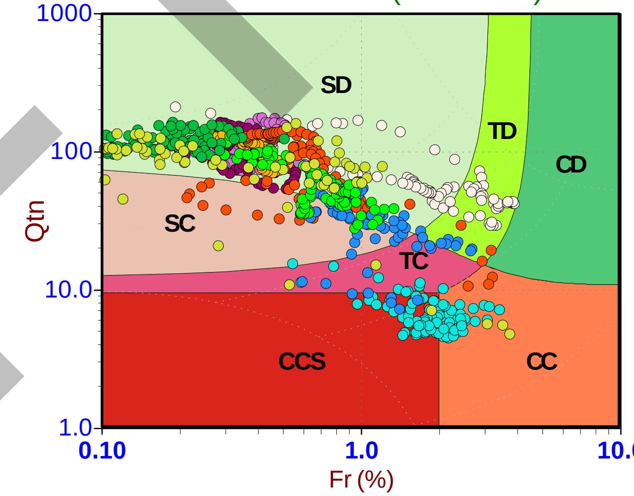
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

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

Legend

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

Legend

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

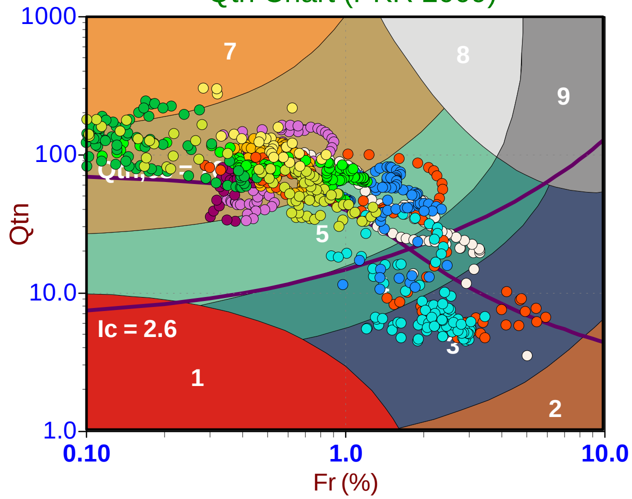
CONETEC

Terracon

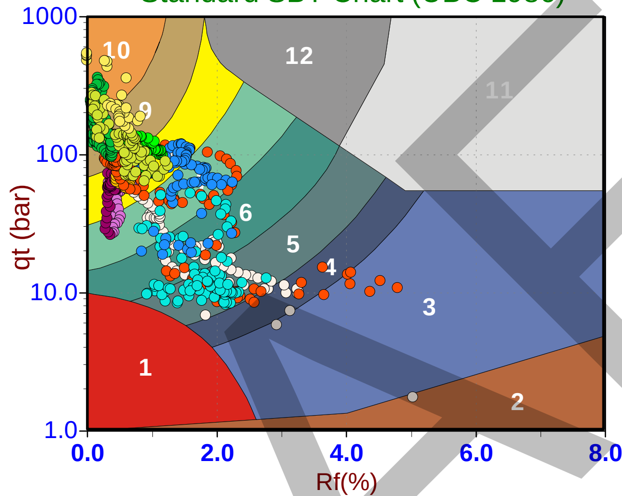
Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35

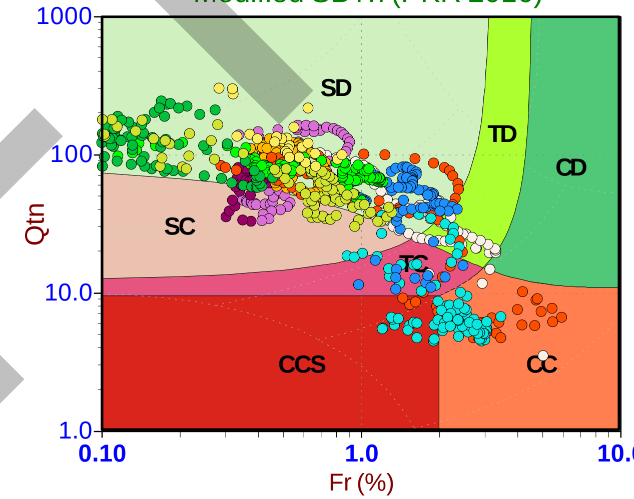
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



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

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

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

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

Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots

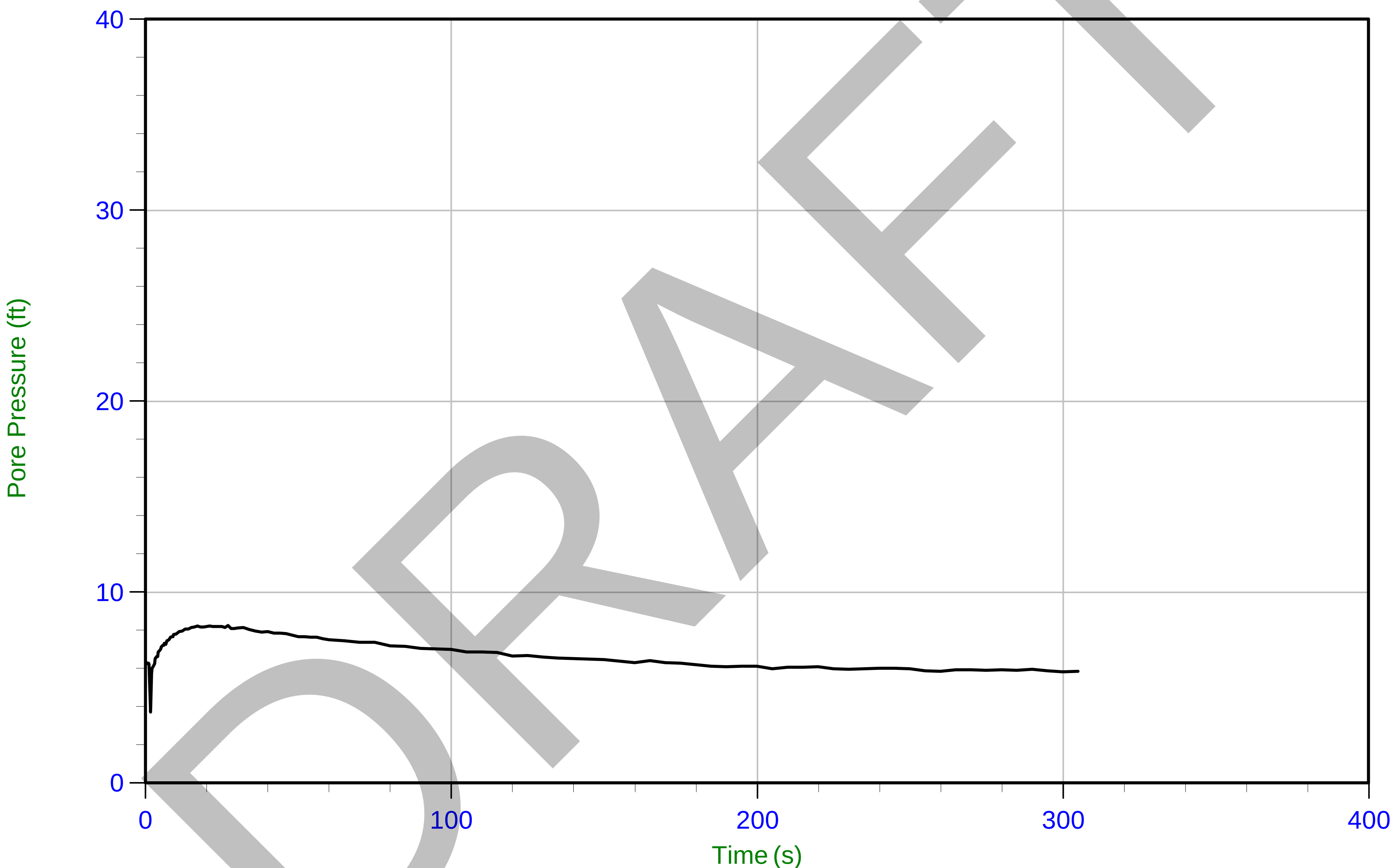
ConeTec

Job No: 24-59-27466
Client: Terracon
Project: Proposed Chick-fil-A, Burlington, WA
Start Date: 2024-04-09
End Date: 2024-04-09

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Refer to Notation Number
CPT-01	24-59-27466_CP01	15	305	18.04	5.8	12.3	
CPT-01	24-59-27466_CP01	15	460	60.53	47.8	12.7	
SCPT-01	24-59-27466_SP01	15	410	11.48	2.5	9.0	
SCPT-01	24-59-27466_SP01	15	275	24.61	15.3	9.3	
SCPT-01	24-59-27466_SP01	15	200	73.82	64.4	9.4	
Totals			27 min				

Terracon	Job No: 24-59-27466	Sounding: CPT-01
	Date: 2024-04-09 12:53	Cone: 855:T1500F15U35 Area=15 cm²
	Site: Proposed Chick-fil-A, Burlington, WA	



Trace Summary:

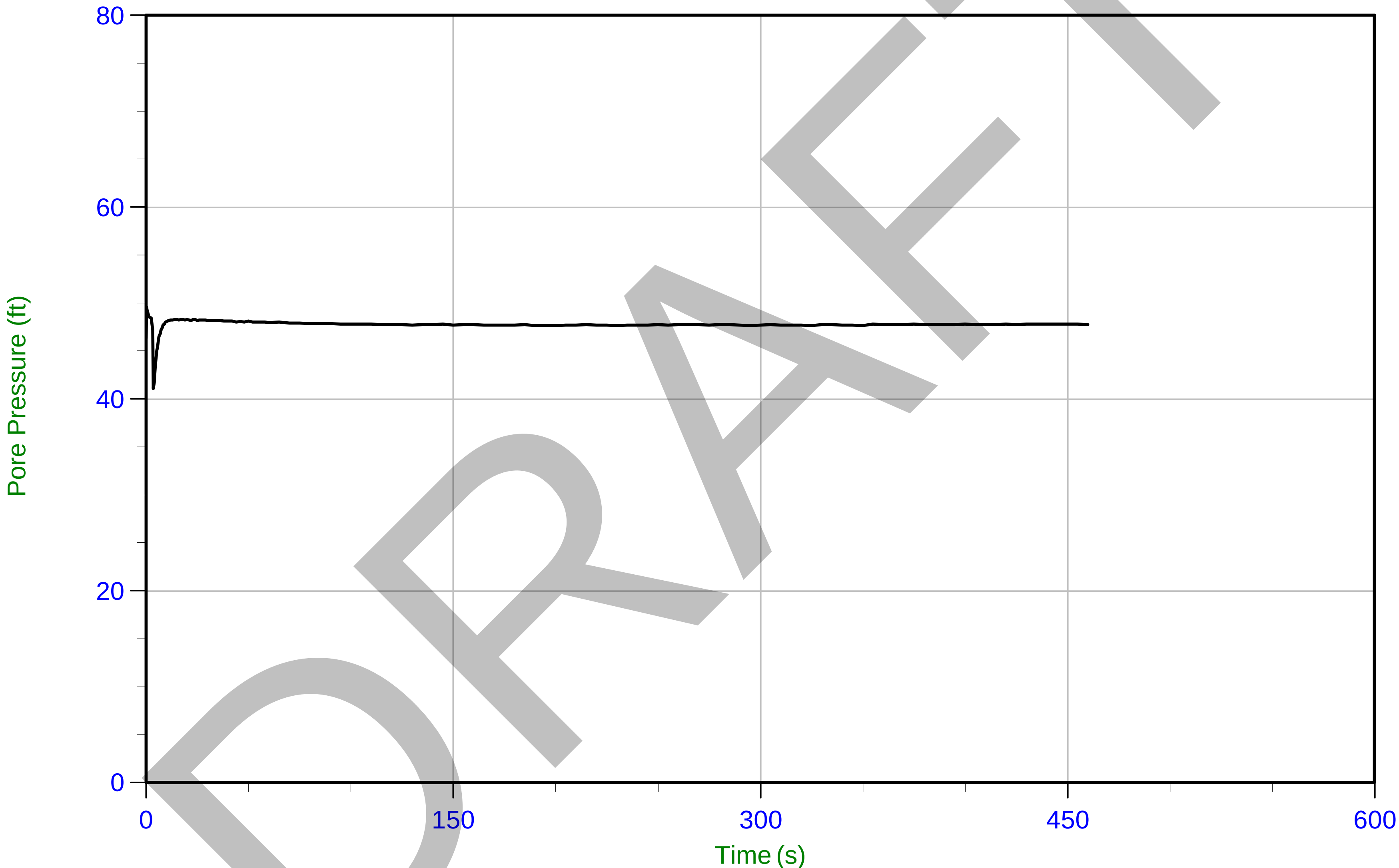
Filename: 24-59-27466_CP01.PPR4	u Min: 3.7 ft	WT: 3.735 m / 12.254 ft
Depth: 5.500 m / 18.044 ft	u Max: 8.2 ft	Ueq: 5.8 ft
Duration: 305.0 s	u Final: 5.9 ft	

ConeTec

Terracon

Job No: 24-59-27466
Date: 2024-04-09 12:53
Site: Proposed Chick-fil-A, Burlington, WA

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



Trace Summary:

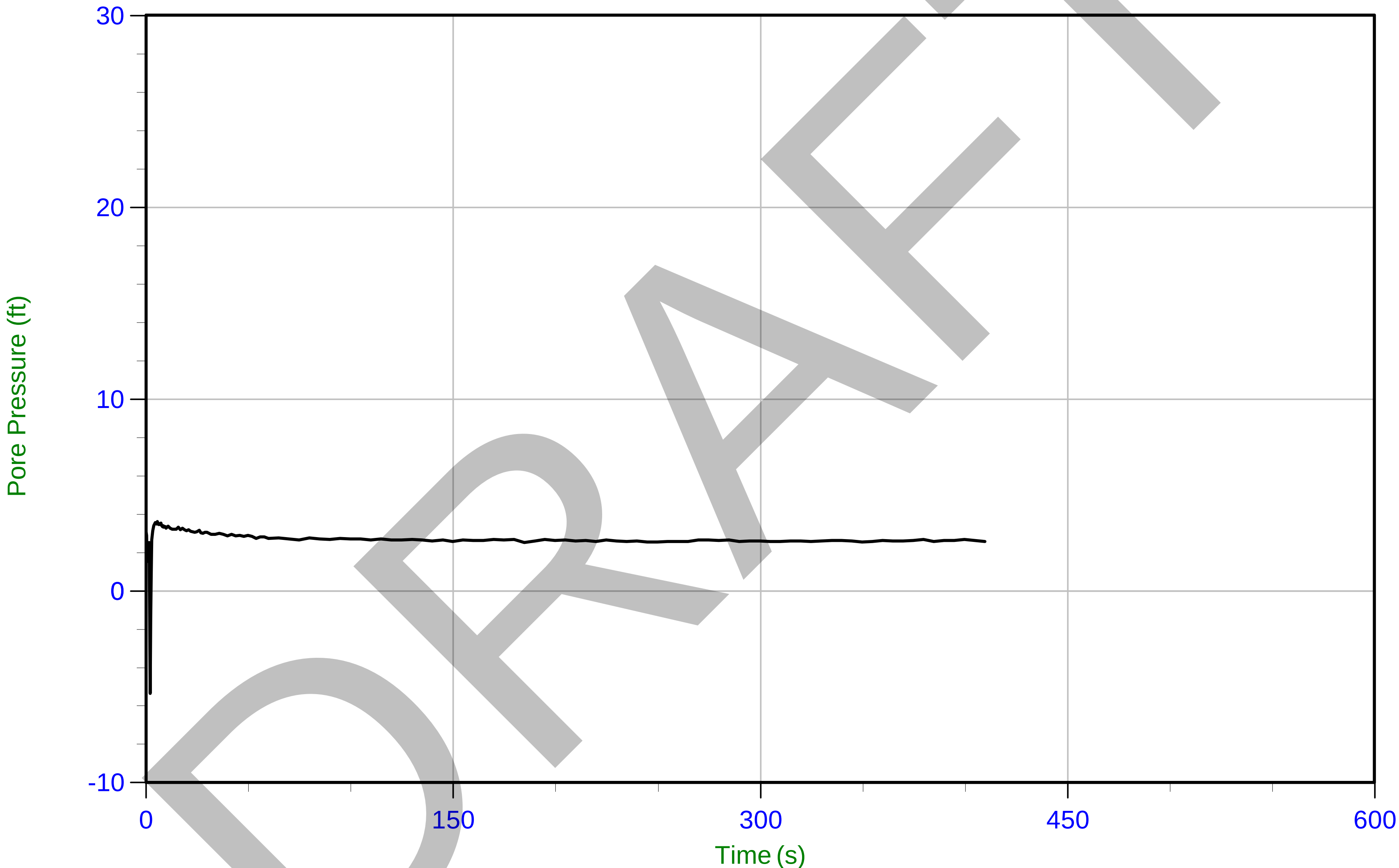
Filename: 24-59-27466_CP01.PPR4
Depth: 18.450 m / 60.531 ft
Duration: 460.0 s

u Min: 41.1 ft
u Max: 49.6 ft
u Final: 47.8 ft

WT: 3.884 m / 12.743 ft
Ueq: 47.8 ft

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA
Sounding: SCPT-01
Cone: 921:T1500F15U35 Area=15 cm²

Terracon



Trace Summary:

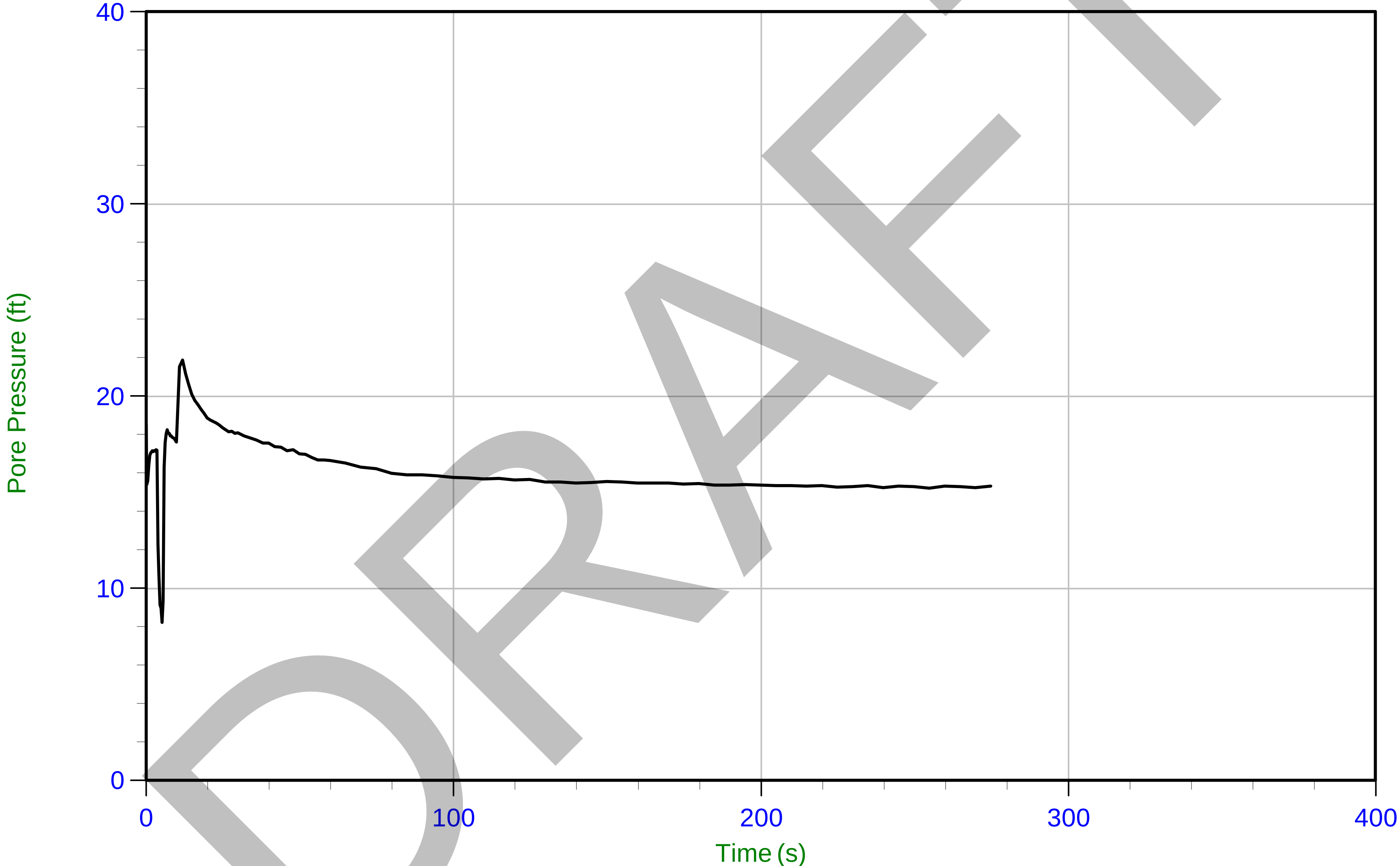
Filename: 24-59-27466_SP01.PPR4
Depth: 3.500 m / 11.483 ft
Duration: 410.0 s

u Min: -5.3 ft
u Max: 3.6 ft
u Final: 2.6 ft

WT: 2.730 m / 8.957 ft
Ueq: 2.5 ft

ConeTec

Terracon	Job No: 24-59-27466	Sounding: SCPT-01
	Date: 2024-04-09 11:49	Cone: 921:T1500F15U35 Area=15 cm²
	Site: Proposed Chick-fil-A, Burlington, WA	



Trace Summary:

Filename: 24-59-27466_SP01.PPR4
Depth: 7.500 m / 24.606 ft
Duration: 275.0 s

u Min: 8.2 ft
u Max: 21.9 ft
u Final: 15.3 ft

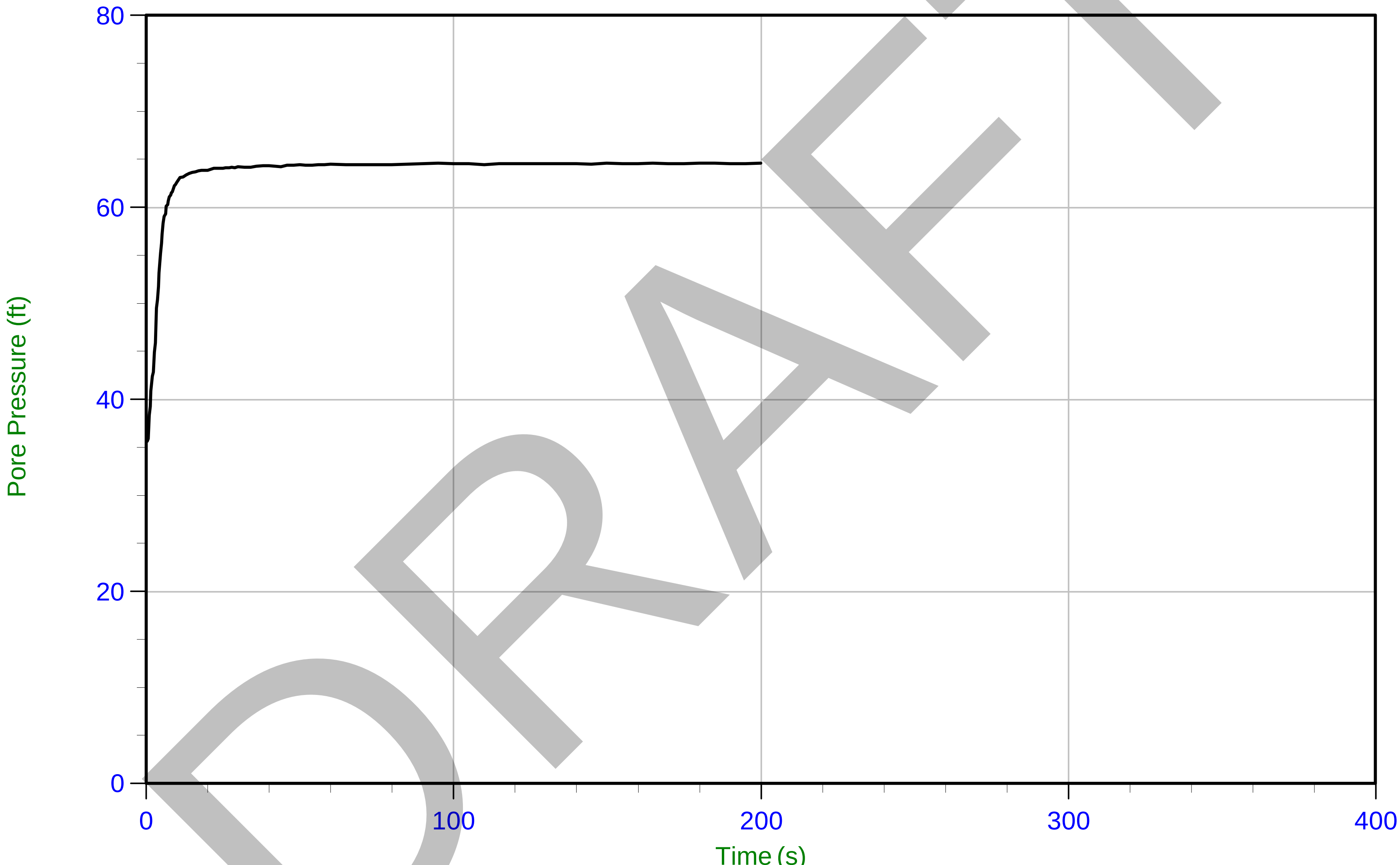
WT: 2.848 m / 9.344 ft
Ueq: 15.3 ft

ConeTec

Terracon

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-59-27466_SP01.PPR4
Depth: 22.500 m / 73.818 ft
Duration: 200.0 s

u Min: 35.7 ft
u Max: 64.6 ft
u Final: 64.6 ft

WT: 2.864 m / 9.396 ft
Ueq: 64.4 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

Job No: 24-59-27466
Client: Terracon
Project: Proposed Chick-fil-A, Burlington, WA
Sounding ID: SCPT-01
Date: 2024-04-09

Seismic Source: Beam
Seismic Offset (ft): 1.74
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.92	4.27	4.61			
8.20	7.55	7.74	3.14	5.86	536
11.48	10.83	10.97	3.22	6.81	474
14.76	14.11	14.22	3.25	6.75	481
18.04	17.39	17.48	3.26	6.87	475
21.33	20.67	20.74	3.27	6.69	489
24.61	23.95	24.01	3.27	5.48	597
27.89	27.23	27.29	3.27	5.83	562
31.17	30.51	30.56	3.28	6.04	543
34.45	33.79	33.84	3.28	6.35	516
37.73	37.07	37.11	3.28	6.34	517
41.01	40.35	40.39	3.28	6.17	531
44.29	43.64	43.67	3.28	5.46	600
47.57	46.92	46.95	3.28	5.64	581
50.85	50.20	50.23	3.28	5.27	623
54.13	53.48	53.51	3.28	5.21	630
57.41	56.76	56.79	3.28	4.62	710
60.70	60.04	60.06	3.28	4.92	667
63.98	63.32	63.34	3.28	4.14	792
67.26	66.60	66.62	3.28	4.40	745
70.47	69.82	69.84	3.21	3.88	828
73.82	73.16	73.18	3.35	4.96	674
77.43	76.77	76.79	3.61	4.54	795

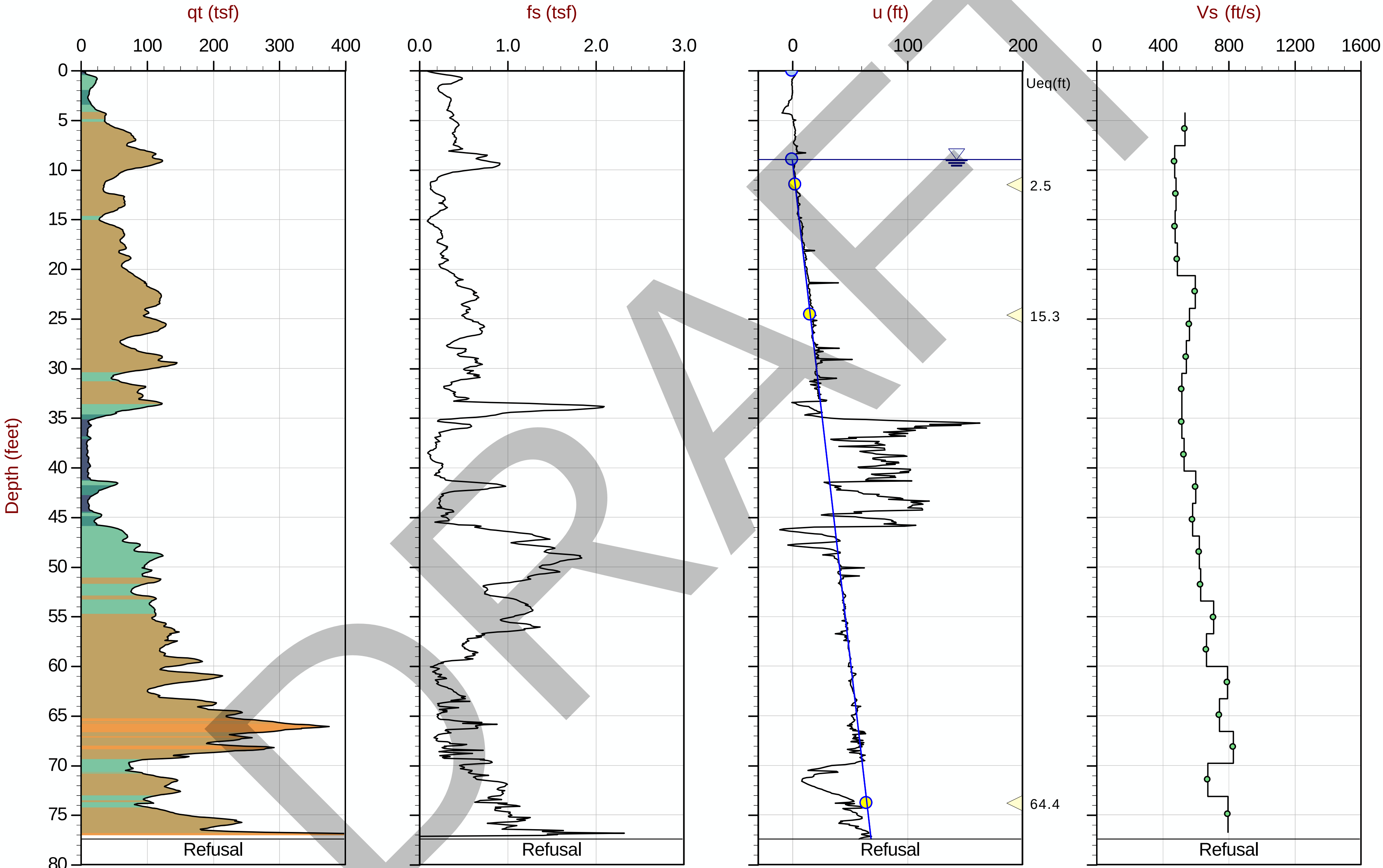
SCPTu Test Plots

CONETEC

Terracon

Job No: 24-59-27466
Date: 2024-04-09 11:49
Site: Proposed Chick-fil-A, Burlington, WA

Sounding: SCPT-01
Cone: 921:T1500F15U35



Max Depth: 23.600 m / 77.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27466_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46938 Long: -122.34379

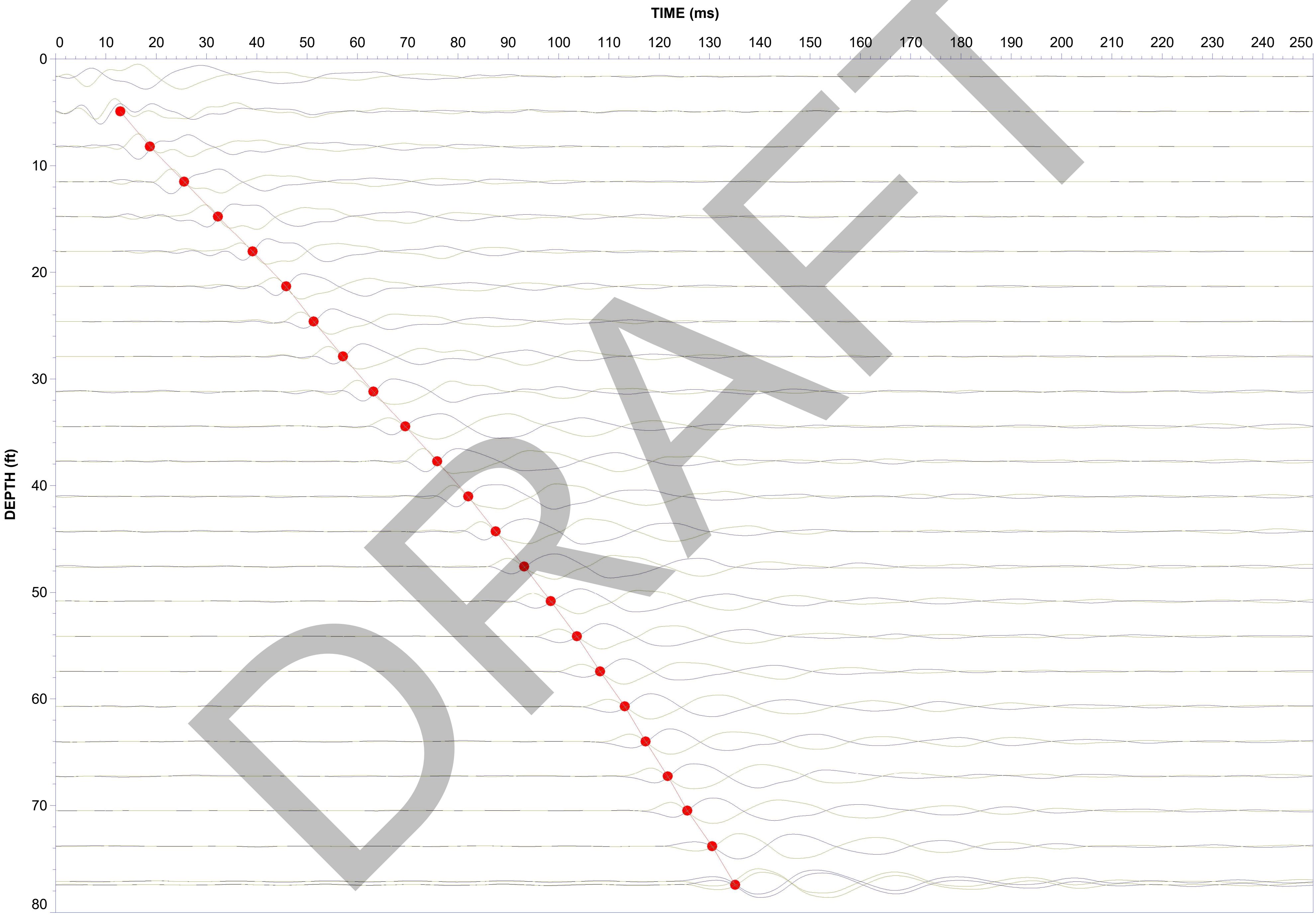
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

ConeTec

Job No: 24-59-27466
Filter: BP 0 - 175 Hz
Client: Terracon
Date: 2024-04-08
Project: Proposed Chick-fil-A, Burlington, WA
Cone: 921:T1500F15U35
Analysis: Shear Wave
Sounding: SCPT-01

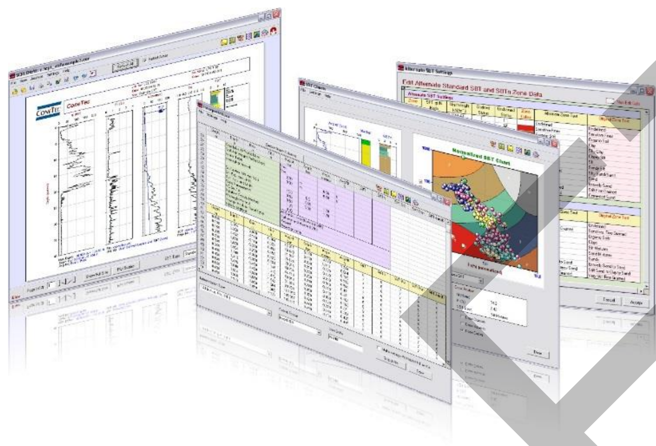


Supplementary Documents and Materials

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.

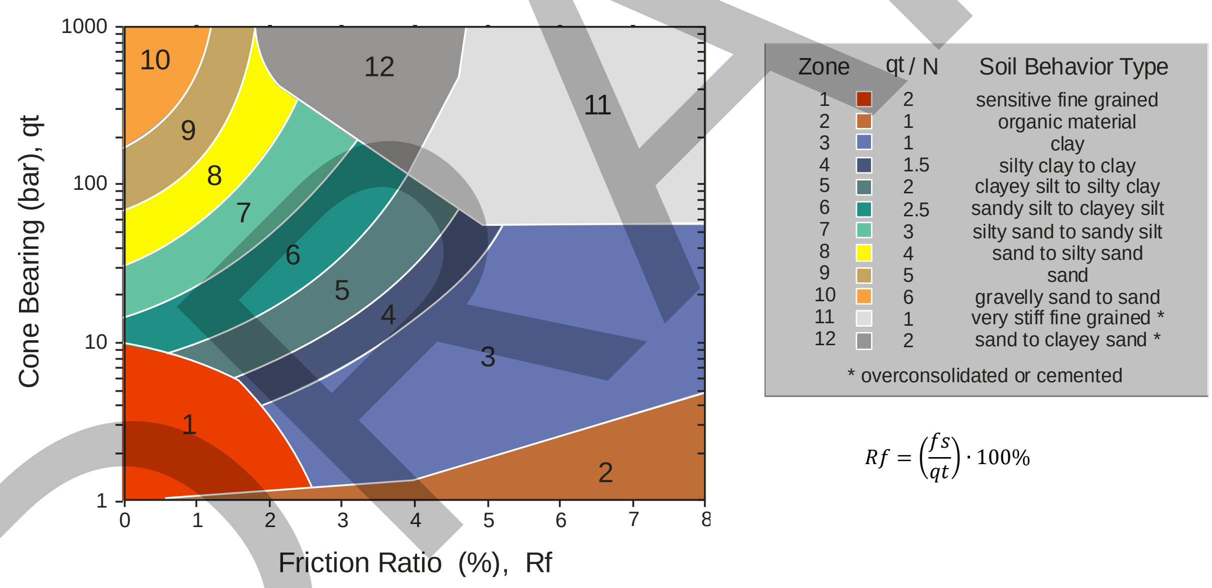


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

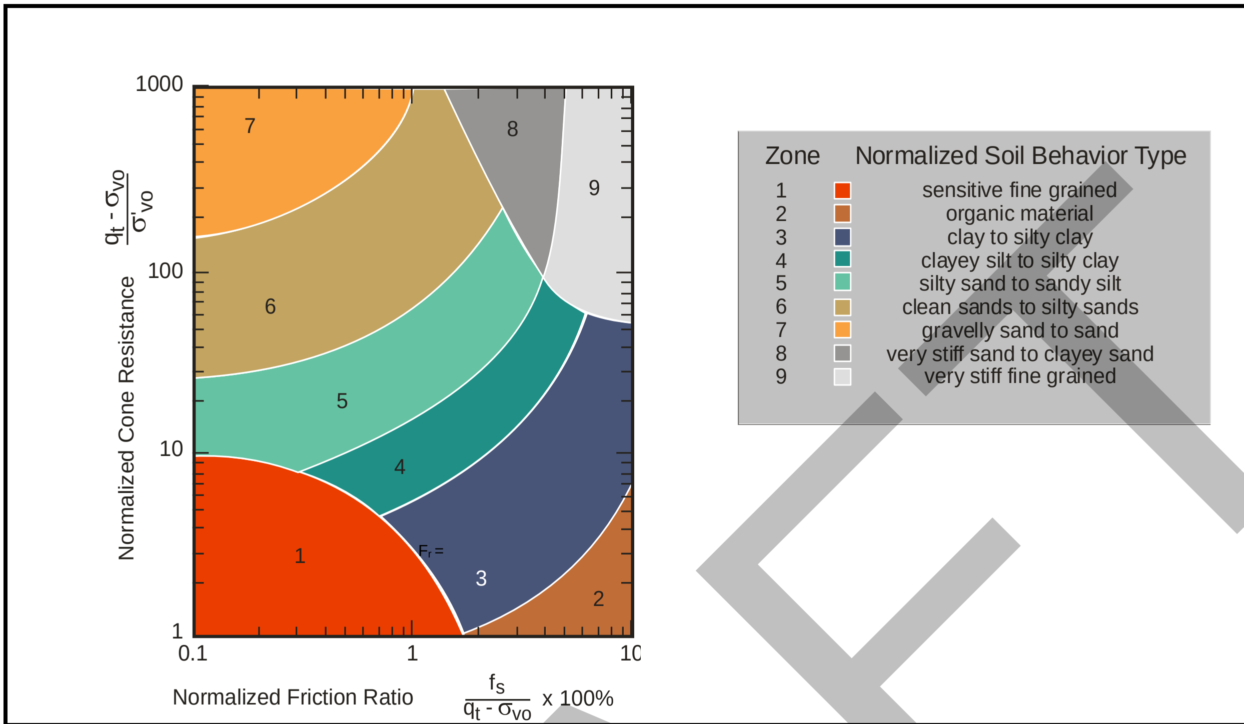


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

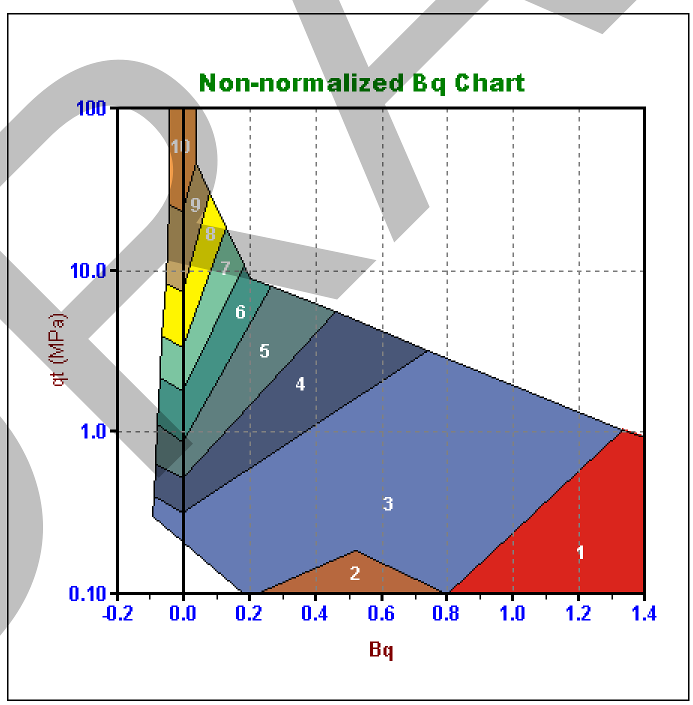


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

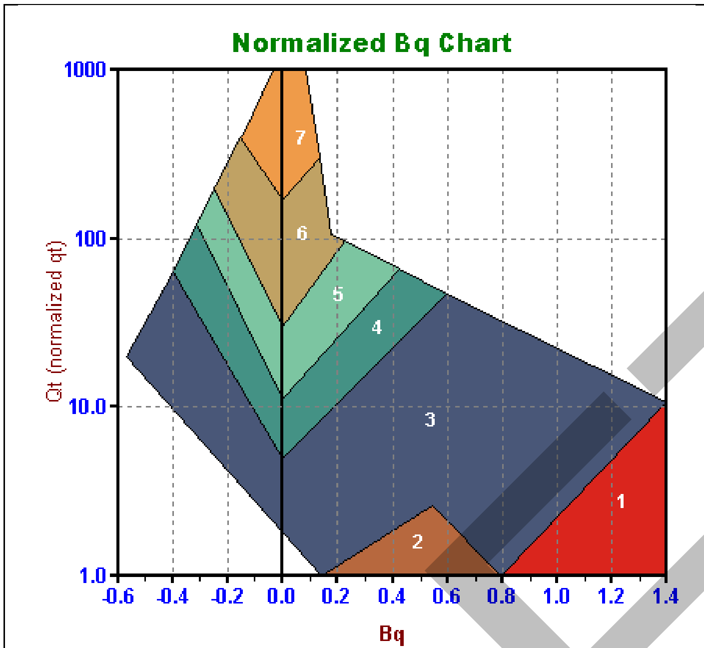


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

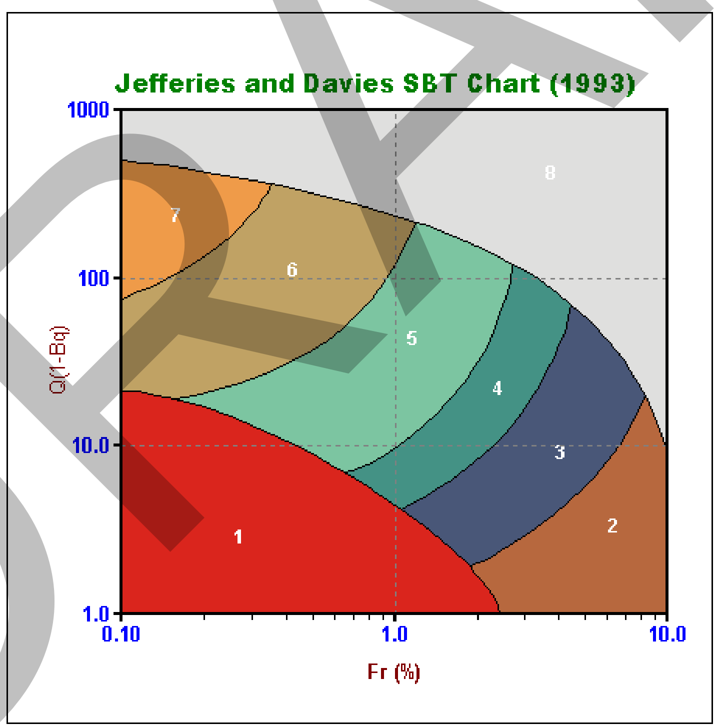


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

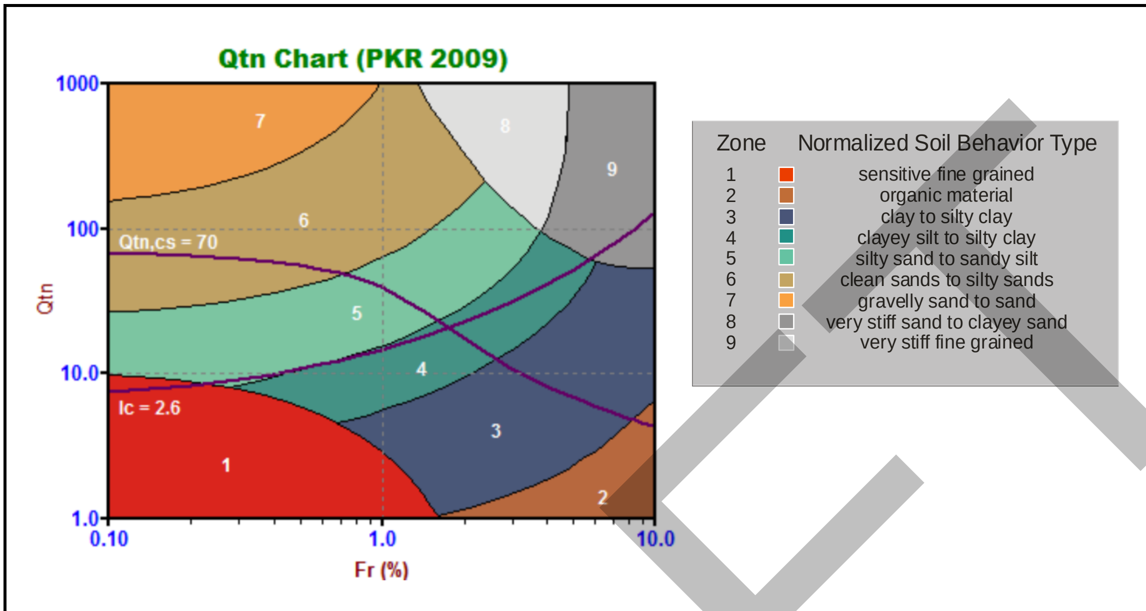


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

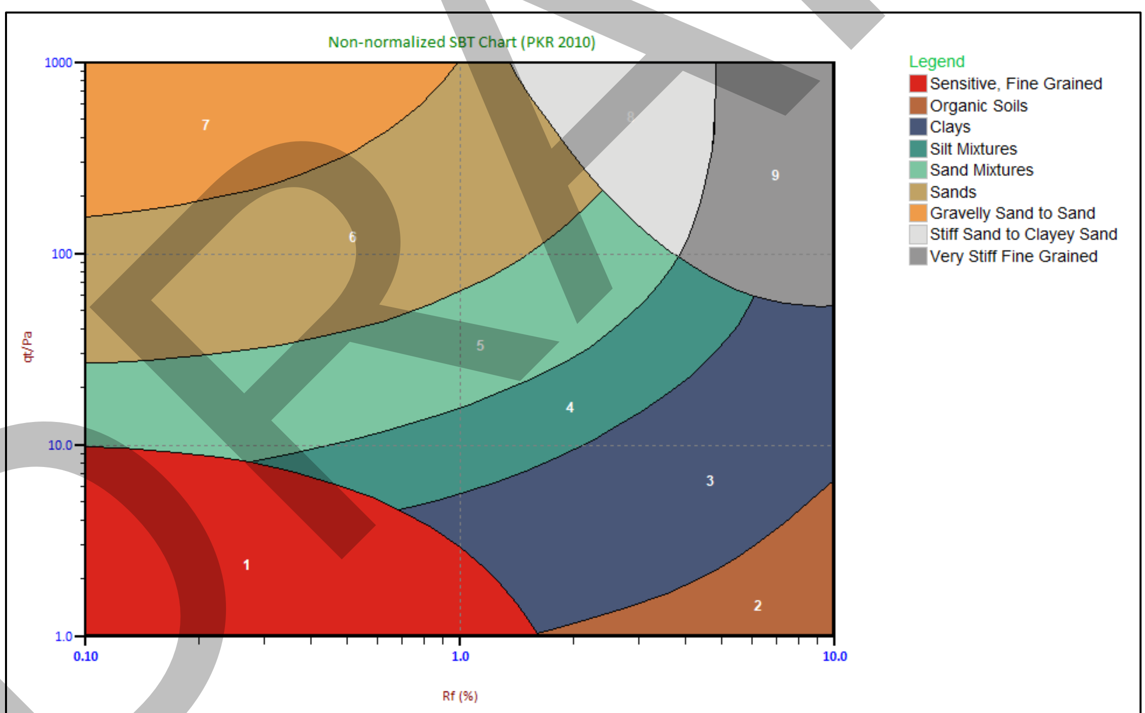


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

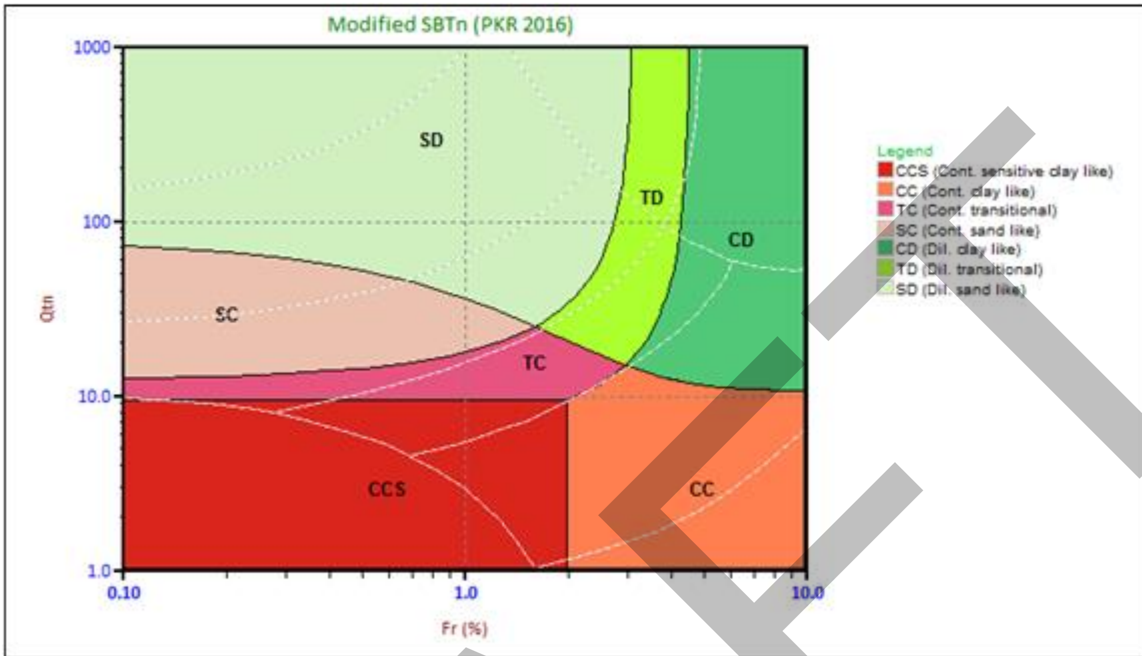


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

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

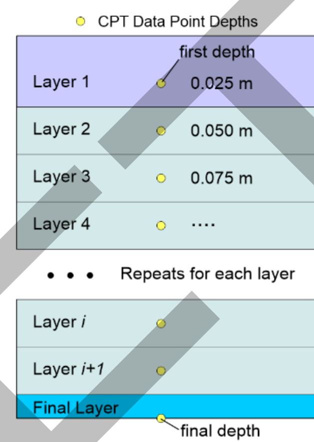
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

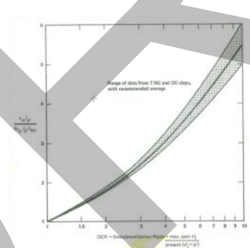
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

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

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

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining* I_c *And* *Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c *(PKR 2009) =* $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	*n (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/σ_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	$= S_u (N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}I_c$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/\ (N_1)_{60cs}I_c = 8.5\ (1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0,\ \beta = 1.0$ FC ≥ 35% $\alpha = 5.0,\ \beta = 1.2$ 5% < FC < 35% $\alpha = exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\dfrac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ 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

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16	Robertson, P.K., 2010a, "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
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No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
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28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
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37	Baldi, G., Bellotti, R., Ghionna,V., Jamiolkowski, M. and Lo Presti, D. 1989. Modulus of sands from CPTs and DMTs. *Proc. Intl. Conf. on Soil Mechanics & Foundation Engineering*, Vol. 1 (ICSMFE, Rio de Janeiro), Balkema, Rotterdam: 165–170. www.issmge.org
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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	EC855	Model	A15 T1500 F15 U35
Date	2024-02-08		
Technician Performing Calibration	Richard Chen	Signature	
Calibration Approved By	Vishrut Khunt	Signature	

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

Cone Information				
Tip Stress Limit	1500	bar	Tip End Area	15 cm^2
Friction Stress Limit	15	bar	Friction Surface Area	225 cm^2
Pressure Limit	35	bar	RTD Location	Pressure Carrier
X-Inclinometer Limit	30	degrees	Geophone	X and Z
Y-Inclinometer Limit	30	degrees	Temperature Range	-20°C to 60°C

Baseline Summary: (For Reference Only)

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

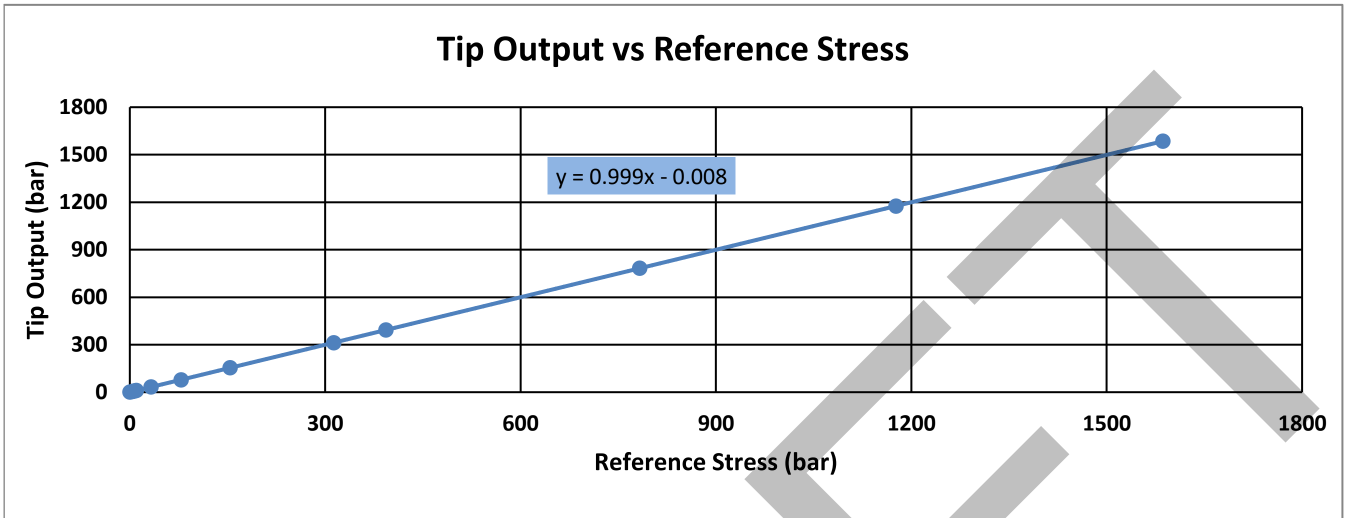
Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

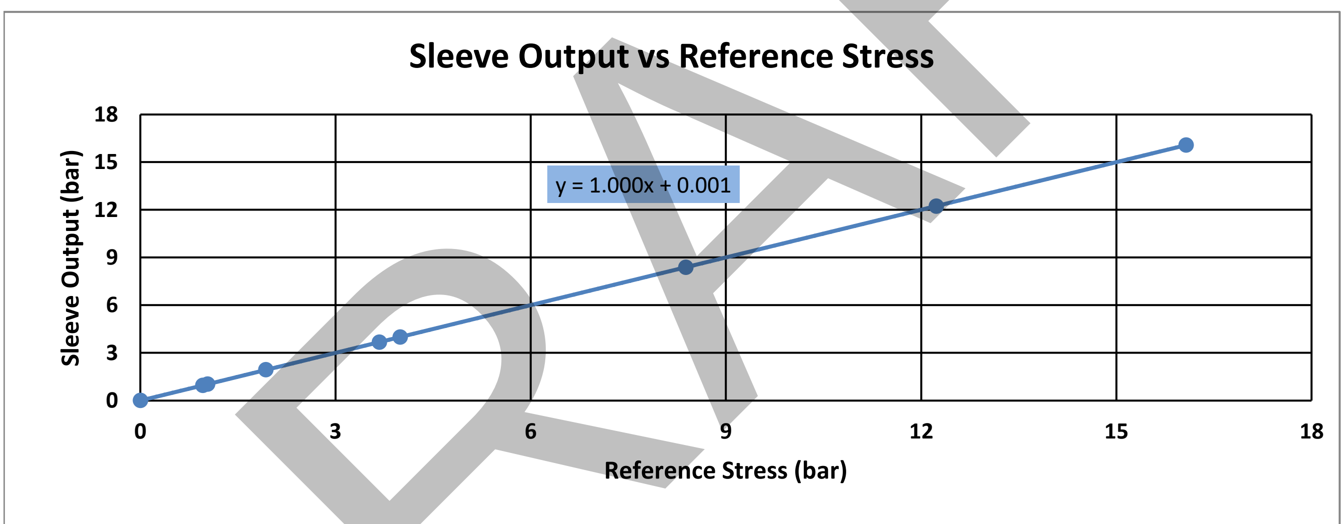
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

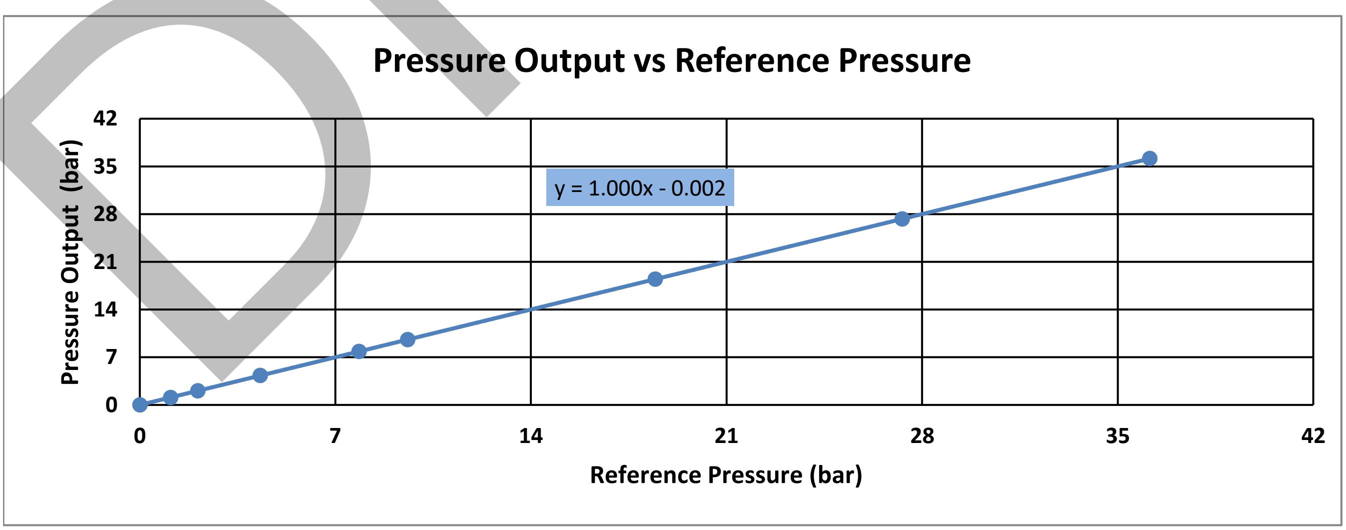
Calibrated with Adara calibration procedure EC_CPTCAL-2.2

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

Cone Output vs Reference Stress/Pressure Plots







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Calibration Results

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

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

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

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

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

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

Temperature Calibration			
Full Scale Error	0.18%	PASS	

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

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

Adara Systems Ltd.
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Testing Equipment Details

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

Adara Error Definitions

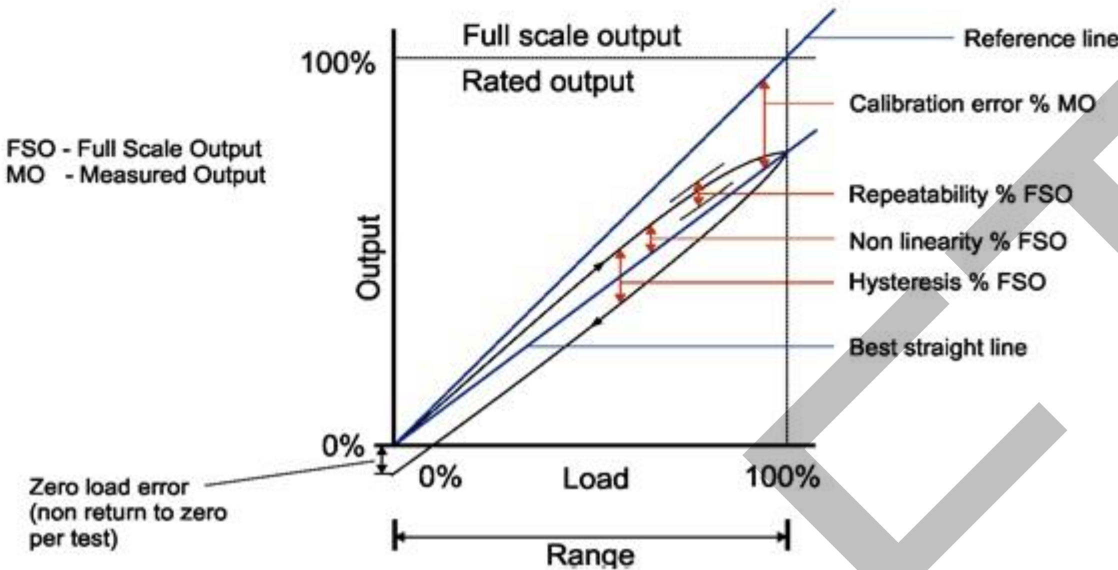

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

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

Temperature Check Passing Criteria

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

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ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

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

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

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

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

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

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

References

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

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

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

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CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC921	Model	A15 T1500 F15 U35
Date	2023-11-02		
Technician Performing Calibration	Chris Kim	Signature	
Calibration Approved By	Vishrut Khunt	Signature	

Lab Condition	As Found	As Left		
Lab Temperature	N/A	22˚C		
Lab Humidity	N/A	42%	Reason for Calibration	Repair

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

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	0.002	0.039
Sleeve	bar	0.000	-0.006
Pressure	bar	0.001	1.007
X-Inclinometer	degrees	0.075	-0.025
Y-Inclinometer	degrees	0.450	0.000
Temperature	˚C	23.297	21.897

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

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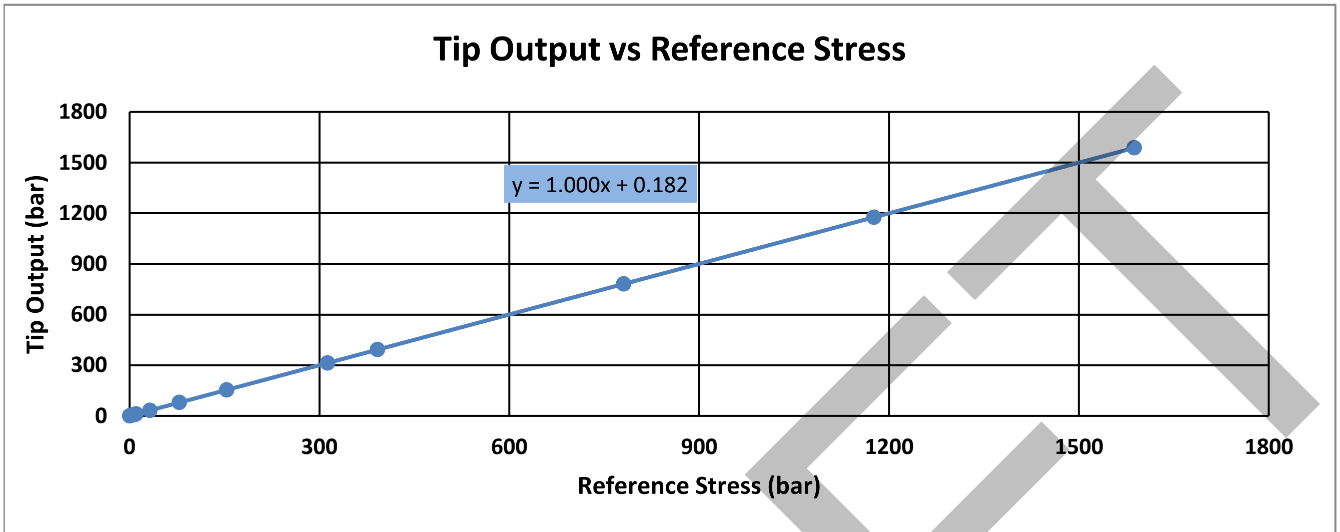
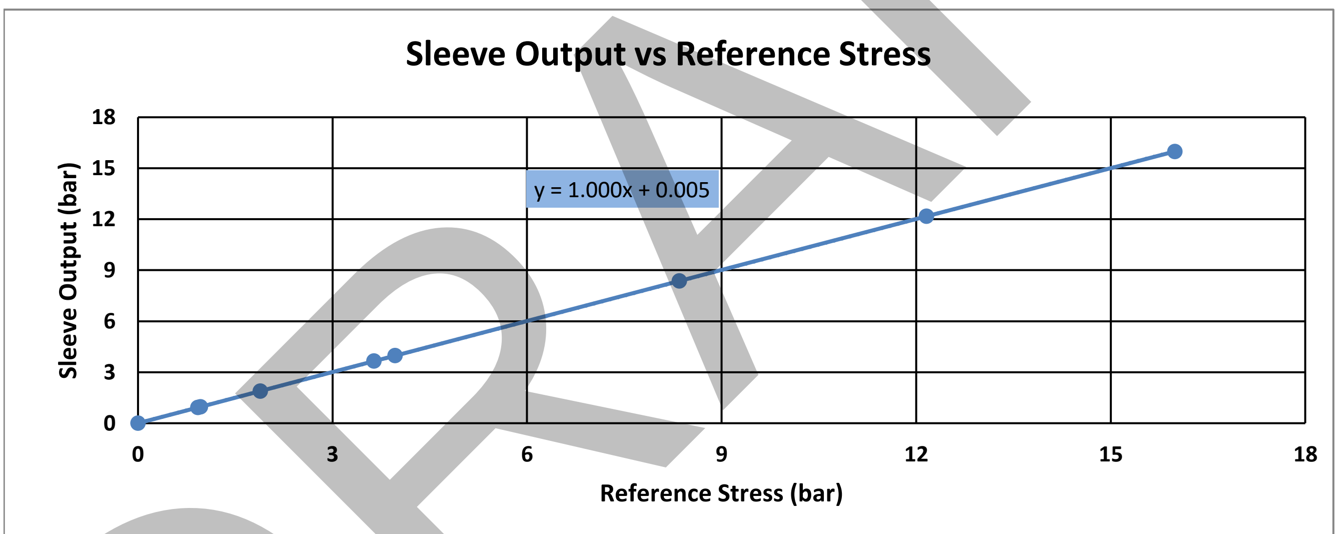
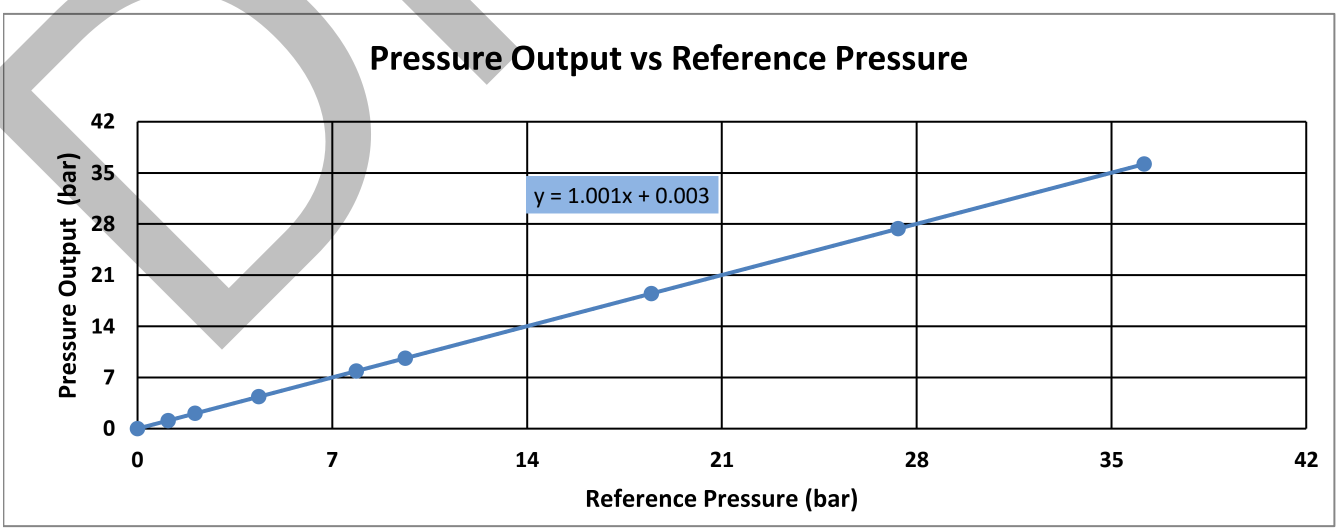
Cone Output vs Reference Stress/Pressure Plots







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Calibration Results

Tip Calibration					
	As Found			As Left	
Max. Non Linearity	105.83%	FAIL	Max. Non Linearity	0.04%	PASS
Calibration Error	100.00%	FAIL	Calibration Error	0.14%	PASS

Sleeve Calibration					
	As Found			As Left	
Max. Non Linearity	106.53%	FAIL	Max. Non Linearity	0.12%	PASS
Calibration Error	99.95%	FAIL	Calibration Error	0.36%	PASS

Pressure Calibration					
	As Found			As Left	
Max. Non Linearity	102.77%	FAIL	Max. Non Linearity	0.07%	PASS
Calibration Error	99.92%	FAIL	Calibration Error	0.13%	PASS

X-Inclinometer Calibration					
	As Found			As Left	
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.17%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.33%	PASS

Y-Inclinometer Calibration					
	As Found			As Left	
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.54%	PASS
Calibration Error	N/A	N/A	Calibration Error	1.08%	PASS

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

Temperature Calibration			
Full Scale Error	0.13%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	6.9	21.2	42.6	˚C
Tip	1.277	0.469	-1.619	bar
Sleeve	0.005	-0.002	-0.007	bar
Pressure	1.079	1.055	0.992	bar
Temperature	6.966	21.064	42.609	˚C

Tip Temperature Coefficient	-0.082 bar/˚C	PASS
Sleeve Temperature Coefficient	0.000 bar/˚C	PASS
Pressure Temperature Coefficient	-0.002 bar/˚C	PASS

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Testing Equipment Details

Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3061	100214	2024-01-05
Tektronix Function Generator	AFG1022	1820013	100167	2023-11-06
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

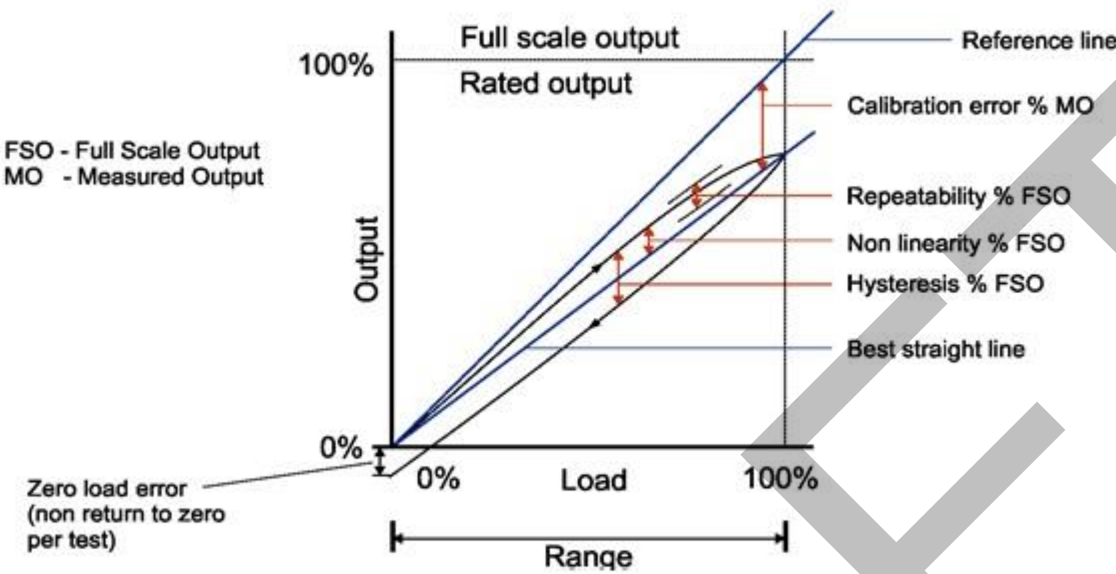

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

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.