Exhibit "10"

RC 1344 – Burlington, WA - Geo

Geotechnical Engineering Report

Burlington, WA

June 20, 2025 | Terracon Project No. 81255069

Prepared for:

Raising Cane's Restaurants, LLC
6800 Bishop Rd
Plano, TX 75024

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June 20, 2025

Raising Cane's Restaurants, LLC
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Attn: LuAron Foster
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E: LFoster@raisingcanes.com

Re: Geotechnical Engineering Report
RC 1344 – Burlington, WA - Geo
1075 S. Burlington Blvd.
Burlington, WA
Terracon Project No. 81255069

Dear Ms. Foster:

We have completed the scope of services for the above referenced project in general accordance with Terracon Task Order No. P81255069 dated April 9, 2025. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We have installed a monitoring well for the wet season ground water monitoring required by the local jurisdiction. We will return to the site in early May 2026 to retrieve our data loggers and prepare a letter summarizing the groundwater monitoring results. 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

Hannah E. Bortel, G.I.T.
Senior Staff Geologist

ZACHARY L. KOEHN
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REGISTERED
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Table of Contents

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.

Introduction

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed Raising Cane's to be located at 1075 S. Burlington Blvd. in Burlington, WA. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface and groundwater conditions
- Seismic considerations and liquefaction
- Ground improvement
- Site preparation and earthwork
- Foundation design and construction
- Canopy foundation design and construction
- Floor slab design and construction
- Lateral earth pressure
- Preliminary stormwater management considerations
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of borings, cone penetration testing, shear wave velocity testing, laboratory testing, geotechnical engineering analyses, and preparation of this report. A groundwater monitoring well with data loggers is actively collecting groundwater data. Data collection will continue through April 30, 2026 when we will return to the site, retrieve the data loggers, and prepare a groundwater summary letter of the monitoring results.

Drawings showing the site and exploration locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. Our final understanding of the project conditions is as follows:

Item	Description
Information Provided	▪ Email request for proposal prepared by Treysta dated March 26, 2025 ▪ Aerial Site Sketch dated July 23, 2024 ▪ SOW prepared by Raising Cane's undated
Project Description	The project will include the construction of a new approximately 2,683 square feet single-story Raising Cane's Restaurant building (Prototype 6-AV). The development will include double drive thru lanes, a canopy over a segment of the drive-thru lanes, a canopy for outdoor seating patio, enclosed dumpster area, and at-grade paved parking.
Building Construction	We anticipate that the building will be constructed using steel or timber framing and shallow foundations with slab-on-grade floors over ground improvement.
Finished Floor Elevation	Finished floor elevation for the building is anticipated to be at or near the existing grade.
Maximum Loads	Anticipated structural loads were not provided. Based on our experience with similar projects, we assumed the following maximum structural loads: ▪ Columns: 100 kips ▪ Walls: 4 kips per linear foot (klf) ▪ Slabs: 150 pounds per square foot (psf)
Grading/Slopes	The site has been previously graded and developed. The proposed finished grade elevation for the building pad is assumed to be at or near existing grade.
Free-Standing Retaining Walls	Retaining walls are not expected to be constructed as part of site development to achieve final grades.
Pavements	Pave driveway and parking will be constructed as part of the development. A preferred pavement surfacing has not been identified to us at this stage of the project. For design purposed, we assume both rigid (concrete) and flexible (asphalt) pavement sections will be considered and will assume that the following traffic demand presented as equivalent 18-kip single-axle loads (ESALs): ▪ Parking areas: 30,000 ▪ Drive aisles and entrances: 60,000 The assumed pavement design period is 20 years

Item	Description
Building Code	The following design codes are assumed: ▪ International Building code – Version 2021 (2021 IBC) ▪ Options for ASCE 7-16 and 7-22 for seismic considerations

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 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	The project is located at 1075 S. Burlington Blvd. in Burlington, WA 98233. Lotze: ~ 1.25 acres Latitude (approximate): 48.4649° N Longitude (approximate): 122.3363° W See **Site Location**
Existing Improvements	The site is currently developed with a Red Robin Restaurant. The site includes paved asphalt parking, drive aisles, and landscaping around the restaurant.
Current Ground Cover	Paved asphalt parking and drive aisles.
Existing Topography	Based on **Google Earth Pro** the site is relatively flat with elevations ranging between 30 to 34 feet.

Geotechnical Characterization

Mapped Surface Geology

The site is mapped as quaternary aged fragmental volcanic rocks and deposits (including lahars) – Qvl based on the Washington Department of Natural Resources Geologic Information Portal. In general, the subsurface explorations were consistent with the mapped geology and other nearby subsurface explorations. Suspected lahar deposits seen at the site can exhibit similar characteristics as alluvial deposits such as being liquefiable.

Subsurface Conditions

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 are indicated on the individual logs. The individual logs and the GeoModel can be found in the **Exploration Results** section of this report.

Soil Layer	Layer Name	General Description
---	**Surface Material**	Approximately 3-inches asphalt
1	**Existing Fill**	Grayish brown, moist, medium dense, well graded GRAVEL with sand (GW) and silty SAND (SM), fine to coarse
2	**Native Silt and Sand**	Grayish brown to gray, moist to wet, loose to medium dense silty SAND (SM), soft to stiff SILT with variable sand content (ML)

Groundwater Conditions

The explorations were observed during advancement for the presence and level of groundwater. The water levels observed at each exploration are summarized below.

Exploration ID	Shallowest Observed Groundwater (feet) [1, 2]
B-01	7½
B-02	7½
B-03	7½
B-04	7½
B-05	6
B-06	7½

1. Below ground surface.
2. Inferred from change in sample moisture

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

Exploration ID	Shallowest Observed Groundwater (feet) [1]
sCPT-01	10.6
CPT-02	10.2

1. Below ground surface.

A monitoring well was constructed following the advancement of soil boring B-03. 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 May 21, 2025 a piezometer data logger was installed in the monitoring well to observe potential fluctuations through the wet season (typically early December through late April).

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.

Seismic Site Class and Hazards

Ground Motion

In 2024, the State of Washington adopted the 2021 IBC allowing 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	7-22 Value [1]	7-16 Values [1]
Site Classification[2]	F	
Site Classification – Based on $V_{S,100}$	D	
Site Latitude	48.4649° North	
Site Longitude	122.3363° West	
S_S – Short Period Spectral Acceleration	1.19 g	1.06 g
S_1 – 1-Second Period Spectral Acceleration	0.35 g	0.38 g
S_{MS} – Short Period Spectral Acceleration Adjusted for Site Class	1.38 g	1.14 g

Description	7-22 Value [1]	7-16 Values [1]
S_{M1} – 1-Second Spectral Acceleration Adjusted for Site Class	0.79 g	--
S_{DS} – Design Short Period Spectral Acceleration	0.92 g	0.76 g
S_{D1} – Design 1-Second Spectral Acceleration	0.53 g	--
PGA_M - ASCE 7, Peak Ground Acceleration Adjusted for Site Class	0.58 g	0.52 g

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 69 feet of the subsurface materials at the site. The properties below the exploration depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. The approximate weighted average shear wave velocity is 703 ft/sec.
2. Site Class F is due to liquefiable soils. Values for Site Class D per 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. The fundamental period of vibration should be verified by the structural engineer. Site Class D is based on shear wave velocity testing at the site.

Surface-Fault Rupture

The hazard 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 June 3, 2025. The closest mapped fault is the Darrington-Devils Mountain fault zone, which lies approximately 7 miles to the south, 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 10 and 60 feet below the ground surface. The likelihood of lateral spreading is low while the likelihood of flow sliding is negligible largely due to the absence of a free-face slope.

Liquefaction was evaluated using CLiq (v.3.5.2.10) developed by GeoLogismiki Software. We estimate a free-field, seismic-induced settlement of up to 10 inches, with differential settlements on the order of 5 inches.

Per ASCE 7-16, Table 12.13-3 for a 40-foot span and risk category II, a differential settlement threshold of about 4½ inches is allowable for "*other multi-story structures.*" A structural engineering should review the magnitudes of seismic-induced settlements presented herein and assess if mitigation of liquefaction hazard is necessary and/or if seismic-ties are needed to interconnect isolated spread footings.

Vertical settlements deemed excessive by the structural engineer can be reduced using ground improvement. Ground improvement via aggregate piers densify the ground, increase soil stiffness, and are embedded in dense material that is not expected to liquefy. 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. Foundation recommendations are provided in the **Shallow Foundations** section. Discussion of aggregate piers is provided in the **Ground Improvement** section.

Ground Improvement

Mitigation of excessive settlement from static loading and/or seismic-induced ground motions (e.g., seismic-induced settlement) is generally accomplished through one of two methods below:

- Densification of native soils through ground improvement
- Transfer foundation loads through weaker soils to competent soils using deep foundations

For this site, we estimate that the post-liquefaction settlements may be on the order of 10 and 5 inches for vertical and differential settlements, respectively. This magnitude of differential settlement is marginal with respect to achieving the code's life safety objective. Additionally, risks associated with undocumented existing fill renders the site unsuitable for shallow foundations without implementing mitigation measures such as completely removing the existing fill or ground improvement.

Based on preliminary consultation with a ground improvement contractor, the settlement risks can be mitigated through use of ground improvement via aggregate piers. An aggregate pier consists of a stone-filled column consisting of crushed stone placed in lifts and applying a high degree of compactive effort resulting in stone filled piers. The aggregate pier construction process not only results in a rigid stone-filled column that lends support to structures, it also helps to densify the soils surrounding the pier. In general, shallow spread footings on ground improvement can be designed for bearing pressures on the order of 4,000 psf with total settlements of 1 inch.

Installation of aggregate piers are proprietary methods and should be designed and installed by a ground improvement specialty contractor. Due to the specialty of this soil improvement procedure, we recommend that a performance specification be used for this system with design parameters defined by the structural engineer. Furthermore, we recommend that Terracon review the ground improvement design prior to construction.

The aggregate pier design firm will become the engineer of record for the ground improvement system and the long-term performance of the foundations. As such, the design firm would provide the necessary design parameters for the planned foundation system including, but not limited to, allowable bearing pressure, settlement estimates, and foundation-specific earthwork recommendations.

We recommend that Raising Cane's contract a specialty ground improvement contractor for development of a project-specific ground improvement design, should this be the desired site improvement option. Terracon should observe the ground improvement installation and testing on a full-time basis. The ground improvement information provided herein should be considered preliminary until confirmed by a ground improvement contractor.

Geotechnical Overview

Based on the results of the explorations and our analyses, the site is estimated to experience appreciable seismic-induced vertical and differential displacements for a design-level seismic event. Liquefiable soils exist to 60 feet below the ground surface. Mitigation of seismic induced displacements can be accomplished through ground improvements or use of deep foundations. Assuming the structures can be designed to tolerate some differential displacements while satisfying the seismic performance objective of life-safety, we recommend ground improvement with aggregate piers. Aggregate piers are generally a less expensive ground improvement method and are commonly used to mitigate excessive settlements. Ground improvement should extend outside the building footprint by 10 feet or at least 10 percent of the building footprint, whichever is greater. A specialty contractor should be consulted for design of the ground improvement system and the structural engineer should be consulted to provide tolerable post-liquefaction displacements.

We observed existing fill at all exploration. Undocumented, existing fill presents a risk of erratic and unpredictable settlement in response to foundation loading. Installation of ground improvement would mitigate the risks associated with undocumented existing fills within the building area, could help mitigate liquefaction risks and increase the bearing capacity, and is therefore, recommended. See the **Ground Improvement** section. See **Existing Fill** section of **Earthwork** for mitigating undocumented fills within the pavement areas.

Based on preliminary consultation with a ground improvement specialty contractor, ground improvement via aggregate piers can mitigate the post-liquefaction settlement, risks associated with the undocumented existing fill, and the estimated low bearing capacity of the foundation soils.

The **Shallow Foundations** section addresses support of the building bearing on ground improvement. Floor slabs are recommended to be supported on ground improvement. The **Floor Slabs** section addresses slab-on-grade support for the building.

A combination of rigid and flexible pavement systems is recommended for this site. The **Pavements** section addresses the design of the pavement system.

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. The recommendations contained in this report are based upon the results of field and laboratory testing, engineering analyses, and our current understanding of the proposed project. 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, (M41-10)*.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration 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 demolition, clearing and grubbing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

Demolition

The proposed building will be constructed within the footprint of the existing Red Robin which will need to be demolished. Underground utilities, conduits, and drain lines should be fully removed and the trenches backfilled per the recommendations herein. Any existing buried debris, foundations, slabs, or deleterious material should be removed completely from all proposed development areas.

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.

Mature trees are located within or near the footprint of some of the proposed buildings, which will require removal at the onset of construction. Tree root systems can remove substantial moisture from surrounding soils. Where trees are removed, the full root ball and all associated dry and desiccated soils should be removed. The soil materials which contain less than 5 percent organics can be reused as engineered fill provided the material is moisture conditioned and properly compacted. Voids left by the removal of the tree-root system should be backfilled per the recommendations herein.

Large area subgrades 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 or representative. Areas excessively deflecting under the proof roll should be delineated and subsequently addressed by the Geotechnical Engineer. Excessively wet or dry material should either be removed or moisture conditioned and recompacted.

Existing Fill

As noted in **Geotechnical Characterization**, all borings encountered previously placed fill to depths ranging from about 4 to 6½ feet. Greater depths of existing fill may be present at locations not explored. We have no records to indicate the degree of control, and consequently, the fill is considered unreliable for support of building and canopy foundation loads. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill but can be reduced by following the recommendations as follows:

- Proposed building footprint, appurtenances, and 10 feet beyond: ground improvement via aggregate piers is recommended.
- Pavement areas: it may be desirable to reduce earthwork costs by partially removing the existing fill within these areas. In doing so, the associated risks of unpredictable settlements must be accepted by the owner. Provided the owner is willing to accept this risk, limited removal of the existing fill is feasible. If the unsuitable soils are to be partially removed, we recommend a minimum 1 foot of existing fill to be removed, followed by scarifying and recompacting the underlying 1 foot. The restoring of grades should be performed with compacted structural fill as recommended herein.

The Geotechnical Engineer can aid the contractor in identifying the presence of existing fill during removal, as well as evaluate the excavated materials for potential reuse.

Fill Material Types

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

Import and On-Site Soil: Excavated on-site soil may be selectively reused as fill below pavement and landscaping areas. Portions of the on-site soil have an elevated fines content and will be sensitive to moisture conditions (particularly during seasonally wet periods) and may not be suitable for reuse when above optimum moisture content.

Imported fill materials should meet the following material property requirements. 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.

Material property requirements for on-site soil for use as general fill and structural fill are noted in the table below:

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] On-site Soils (i.e. GeoModel Layers 1) [2, 3]	Beneath and adjacent to structural slabs, foundations, building appurtenances, and pavement subgrades
Common Fill	Section 9-03.14(3) *Common Borrow* [1] On-site Soils (i.e. GeoModel Layers 1) [2, 3]	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]

Fill Type	Recommended Materials	Acceptable Location for Placement

1. WSDOT Standard Specifications
2. Structural and common fill should consist of approved materials free of organic matter and debris. 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 drainpipe 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.

Fill Placement and Compaction Requirements

Structural and common fill should meet the following compaction requirements.

Item	Structural Fill	Common 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]	95% of max. below and adjacent to foundations and within 1 foot of finished pavement subgrade 92% of max. 1 foot or more below finished pavement subgrade	92% of maximum dry density
Water Content Range[1]	Typically within 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 softened/disturbed 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.

All trenches should be wide enough to allow for compaction around the haunches of the pipe. If water is encountered in the excavations, it should be removed prior to fill placement.

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. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

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. Effective drainage will be essential during construction to limit the extent of soil disturbance during the wet season.

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. Gutters and downspouts should be routed into tightline pipes that discharge either directly into a municipal storm drain or to an alternative drainage facility. Splash-blocks should also be considered below hose bibs and water spigots.

Site grades should be established such that surface water is directed away from foundation and pavement subgrades to prevent an increase in the water content of the soils. Adequate positive drainage diverting water from structures, open cuts, and slopes should be established to prevent erosion, ground loss, and instability. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping, final grades should be verified to document effective drainage has been achieved. 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. These remedial actions should be performed under the observation of the geotechnical engineer.

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 proof rolling and mitigation of unsuitable areas delineated by the proof roll.

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. 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 prescribe mitigation options and observe the implementation.

Where ground improvement has been implemented, footing subgrades should be observed by the Geotechnical Engineer for the presence of the aggregate piers to confirm installation has occurred per plan. Any improperly located aggregate piers should be reinstalled, or the footing widened such that the footings bear firm on the aggregate piers or leveling pad. In the event that unanticipated conditions are encountered, the Geotechnical Engineer should recommend mitigation 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 near-surface soils have variable fines content based on our visual observations and lab testing and are considered moisture sensitive. The soils will exhibit moderate erosion potential and may be transported by running water. Silt fences and other best-management practices will be necessary to control erosion and sediment transport during construction.

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.

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.

Shallow Foundations

Provided ground improvement has been installed, and subgrades are prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow foundations.

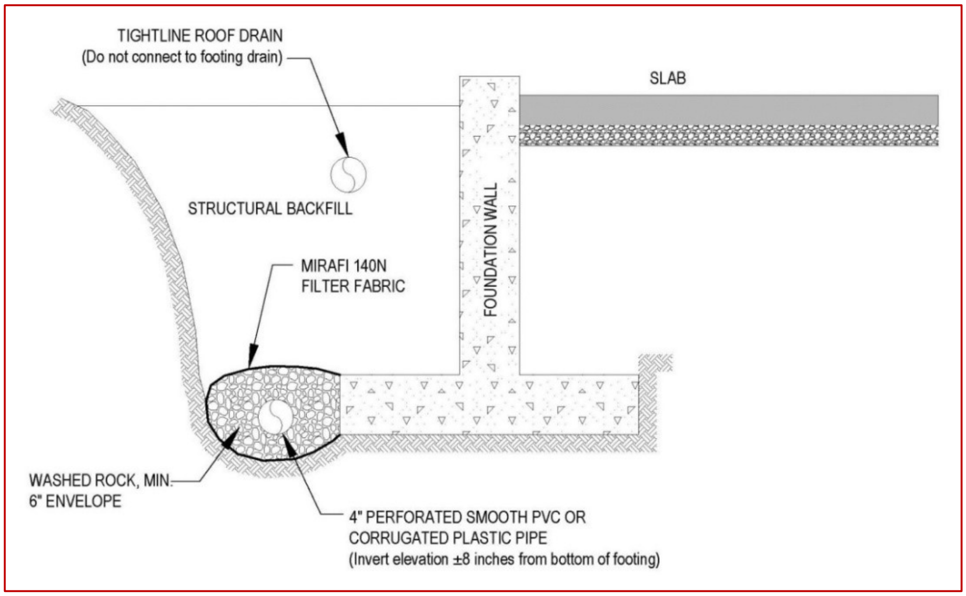
Design Parameters – Compressive Loads

Item	Description
Preliminary Maximum Net Allowable Bearing Pressure [1, 2, 3]	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
Sliding Resistance [5]	0.4 allowable coefficient of friction
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. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2. Values provided are for maximum loads noted in **Project Description**. Additional geotechnical consultation will be necessary if higher loads are anticipated.
3. **Following** recommendations provided in **Ground Improvement**.
4. Passive resistance in the upper 2 feet of the soil profile should be neglected. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
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.
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 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 (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 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. We anticipate that a ground improvement and a shallow foundation system are feasible.

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 Augered and Cast-in-Place (ACIP) piles. The values are presented for allowable side friction and end bearing include a factor of safety of 2 and 3, respectively.

Piles should have a minimum (center-to-center) spacing of six (6) pile diameters. Closer spacing may require a reduction in axial load capacity. If 6B cannot be achieved due to site constraints, please contact Terracon to provide updated lateral design.

A minimum pile diameter of 12 inches should be used. Pile embedment should be no less than 5 feet below ground level.

ACIP Design Summary [1]

Depth (feet)	Stratigraphy [2]		Allowable Side Friction (psf) [3]	Allowable End Bearing Pressure (psf) [4]
	Model No.	Material		
0-5	1	Existing Fill	190	--
>5	2	Native Silt and Sand	300	2,600

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. Piles should extend at least 5 feet into Soil Layer 2.

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
0-5	1	Sand (Reese)	32°	125	75	--
5-10	2	Sand (Reese)	30°	120	30	--

1. See Subsurface Profile in **Geotechnical Characterization** for more details on Stratigraphy.
2. Definition of Terms:

Stratigraphy[1]		L-Pile Soil Model	ϕ [2]	γ' (pcf) [2]	K (pci)	
Depth (ft; bgs)	Model No.				Static	Cyclic

ϕ: Internal friction angle

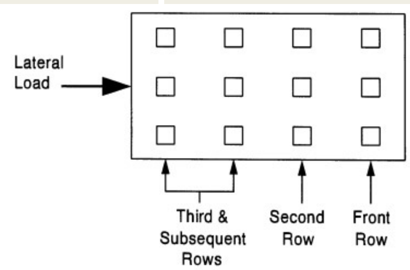
γ': Effective unit weight

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 driving operations. 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/rock and groundwater conditions observed, consistency with expected conditions, and details of the installed pile.

Floor Slabs

Unsuitable soils (e.g. existing fill, loose/soft native soils) may be encountered at the floor slab subgrade level. These soils should be replaced with compacted structural fill, such that 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 have positive drainage away from the structure along with positive drainage of the aggregate base beneath the floor slab. The recommendation presented below do not assume ground improvement is included beneath floor slab areas. If ground improvement is performed, the modulus of subgrade reaction would be higher therefore the values presented below are conservative. A ground improvement contractor should be consulted if a revised estimate of modulus of subgrade reaction is desired.

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]	130 pounds per square inch per inch (psi/in) for point loads 90 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.

Item	Description

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 value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table. It is provided for point loads.

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.

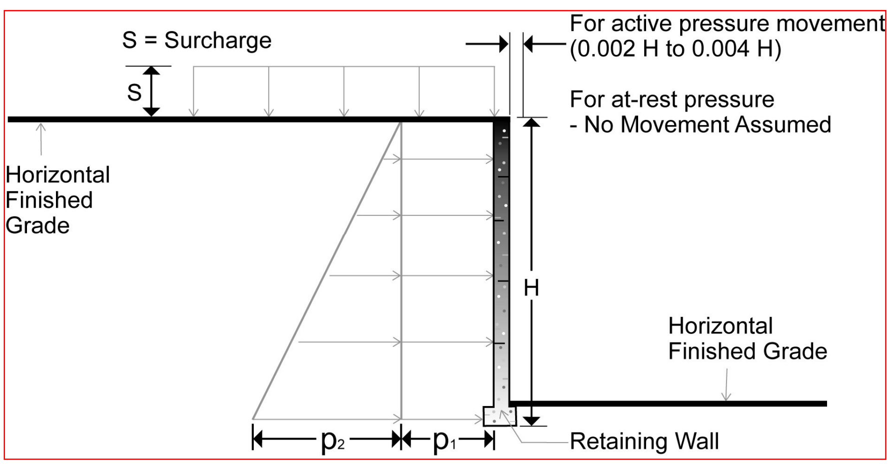
The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches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

Lateral Earth Pressures

Based on the current site plan, retaining walls or loading docks are not anticipated. However, the project is in its early stages and locations of developments features may not yet be known. For planning purposes, lateral earth pressures are provided in the event retaining walls are needed to accommodate grade transitions. If retaining walls are incorporated into the planned development, Terracon should be retained to review the wall locations and confirm the recommendations presented herein are appropriate.

Design Parameters

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3,4,5] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4,5]
Active (Ka)	0.31	(0.31)S	(40)H
At-Rest (Ko)	0.47	(0.47)S	(60)H

Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3,4,5] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4,5]
Passive (Kp)	3.25	---	(400)H
Seismic[6]	---	(7)H – Active (12)H – At-Rest	---

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance. Fat clay or other expansive soils should not be used as backfill behind the wall.
2. Uniform, horizontal backfill, compacted to at least 92 percent of the ASTM D1557 maximum dry density.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. No safety factor is included in these values
6. Values are in addition to static earth pressures

Backfill placed against structures should consist of granular soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively.

Preliminary Stormwater Management Discussion

Due to the destructive nature of infiltration test pits, this testing type is deferred until a later design phase, or until after the property has been acquired (assuming this is required by the local jurisdiction). For planning purposes, we evaluated infiltration potential based on grain size analysis. These should be used for preliminary sizing of the infiltration facility only. Terracon should be retained to conduct pilot infiltration testing (PIT) upon acquiring the site and confirming the site's infiltration testing requirements.

Using the grain-size correlation presented in the Department of Ecology Manual for Western Washington, a design infiltration rate of 0.8 inches per hour is estimated. Reduction factors of 1.0, 0.4, and 0.9 were applied for site variability, test method, and siltation/bio-buildup, respectively.

The groundwater level of roughly 7½ feet bgs may limit infiltration options. In our experience, feasible designs include shallow dispersion trenches or detention ponds. We suspect a groundwater mounding analysis will need to be performed in support of an infiltration system. Please contact Terracon if this analysis is deemed necessary by the project's civil engineer.

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.

If expected traffic volumes exceed the values assumed, Terracon should be notified in order to provide pavement sections designed for higher levels of traffic.

Pavement Design Parameters

A 20-year design life is assumed. A California Bearing Ratio (CBR) of 10 was used for the subgrade for the asphaltic concrete (AC) pavement designs. Any imported or borrow source fill placed below the proposed pavements should have a CBR value of at least 10. 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).

Pavement Section Thicknesses

The design of Asphaltic Concrete (AC) pavements are based on the 1993 AASHTO guidelines. Minimum recommended pavement section thicknesses are presented below:

Asphaltic Concrete (AC) Design

Layer	Light Duty Layer Thickness (inches)	Heavy Duty Layer Thickness (inches)
Compacted Subgrade [1]	12	12
Crushed Aggregate Base [2]	4	6
Asphalt Thickness [3, 4]	3	4

1. May vary based on observations following proof-rolling.
2. Aggregate base meeting WSDOT:9-03.9(3) Base Course specifications, and the requirements specified in the **Earthwork** section.

Asphaltic Concrete (AC) Design

Layer	Light Duty Layer Thickness (inches)	Heavy Duty Layer Thickness (inches)

3. Aggregates for asphalt surface meeting WSDOT: 9-03.8(2) ½-inch HMA requirements.
4. PG58H-22 asphalt binder.

Portland Cement Concrete (PCC) Design

Layer	Layer Thickness (inches)
Compacted Subgrade [1]	12
Crushed Aggregate Base [2]	4
Concrete	5

1. May vary based on observations following proof-rolling.
2. Aggregate base meeting WSDOT:9-03.9(3) Base Course specifications, and the requirements specified in the **Earthwork** section.

We recommend that Portland cement concrete (PCC, rigid) pavement be used where rigid pavements are appropriate. These areas include but are not limited to entrance and exit sections, dumpster pads, or any areas where extensive wheel maneuvering or repeated loading are expected. The rigid pavement pads should be large enough to support the wheels of the truck which will bear the haul load. Adequate reinforcement and number of longitudinal and transverse control joints should be placed in the rigid pavement in accordance with ACI requirements. Although not required for structural support, the base course layer is recommended to help reduce potential for slab curl, shrinkage cracking, subgrade “pumping” through joints, and provide a workable surface. These thicknesses assume the subgrade is properly prepared and compacted as noted above. Proper joint spacing will also be required to prevent excessive slab curling and shrinkage cracking. All joints should be sealed to prevent entry of foreign material and dowelled where necessary for load transfer.

The minimum pavement sections outlined above were determined based on post-construction traffic loading conditions. These pavement sections do not account for heavy construction traffic during development. A partially constructed structural section that is subjected to heavy construction traffic can result in pavement deterioration and premature distress or failure. Our experience indicates that this pavement construction practice can result in pavements that will not perform as intended. Considering this information, several alternatives are available to mitigate the impact of heavy construction traffic prior to pavement construction. These include using thicker sections to account for the construction traffic after paving; using some method of soil stabilization to improve the support characteristics of the pavement subgrade; routing heavy construction traffic around paved areas; or delaying paving operations until as near the end of construction as is feasible.

Areas for the 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.

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.

We recommend drainage be included at the bottom of Aggregate Base (when used) at the storm structures to aid in removing water that may enter this layer. Drainage could consist of small diameter weep holes excavated around the perimeter of the storm structures. The weep holes should be excavated at the elevation of the Aggregate Base and soil interface. The excavation should be covered with Aggregate Base encompassed in Mirafi 140NL, or an approved equivalent, which will aid in reducing the amount of fines that enter the storm system.

Pavement Maintenance

The pavement sections represent the 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.

Geotechnical Engineer of Record

We consider our continuation as the Geotechnical Engineer through construction a vital quality criterion for our recommendations. Terracon must be retained to provide geotechnical observation services through mass grading and earth-related phases of construction to continue as the project's Geotechnical Engineer of Record. If another firm, or entity, other than Terracon is retained to observe the implementation of our recommendations provided herein, the other firm will become the project's Geotechnical Engineer of Record.

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 Explorations	Type of Exploration	Approximate Exploration Depth (feet)	Location
3	Soil Borings	26½	Proposed Building
3	Soil Borings	11½	Proposed Parking Area
2	CPTs	69	Proposed Building

Boring Layout and Elevations: Terracon personnel provided the boring layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. Approximate ground surface elevations were obtained by interpolation from Google Earth Pro. If elevations and a more precise boring layout are desired, we recommend borings be surveyed.

Soil Boring Procedures: We advanced the borings with a track-mounted rotary drill rig using continuous flight hollow stem augers. Four samples were obtained in the upper 10 feet of each boring and 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. For safety purposes, all borings, except where monitoring wells were installed, were backfilled with bentonite chip after their completion in accordance with Washington Department of Ecology requirements related to completion of borings. Pavements were patched with cold-mix asphalt.

We also observed the boreholes while drilling and 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 was 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. 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.

Monitoring Wells: Following advancement of soil boring B-XX, a monitoring well was constructed in the borehole. The well screen interval and construction details are shown on the well logs. A piezometer datalogger was placed in the well to observe water levels every 24 hours. A barometric pressure data logger was installed at the well location to collect barometric pressure values concurrent with the groundwater level.

Cone Penetration Testing (CPT): Advancement of the cone instrument was performed through a porthole in the approximate center of a truck rig. The CPT rig is outfitted with a hydraulic press that continuously advances a standardized and calibrated cone at a constant rate. During advancement, a near-continuous data profile was collected for cone tip and side friction resistance exerted on the cone by the soils as well as the in-situ pore water pressure generated during cone advancement. Tip, side friction, and pore water data were interpreted using empirical correlations to derive soil engineering properties for the full depth of cone advancement. The data collected was used to estimate a soil behavior type which is used to infer the classification of the soils encountered (i.e. sand, silt, clay, etc.). Shear wave velocity measurements were obtained from a 100-foot CPT for purposes of site classification.

Estimates of groundwater level are made through measuring the dissipation of excess pore water pressure that is generated during cone advancement. Multiple dissipations were performed, as necessary, to characterize the groundwater levels across the site.

The CPT subcontractor provided Terracon with the electronic data and interpretations of the explorations. The Terracon project engineer interpreted the data further to perform preliminary geotechnical engineering evaluations. Samples are not obtained during performance of CPTs. Test hole was backfilled with bentonite chips and the pavements were patched with cold asphalt patch.

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 D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D1140 Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing
- ASTM D2974 Standard Test Methods for Determining the Water (Moisture) Content, Ash Content, and Organic Material of Peat and Other Organic Soils

The laboratory testing program often included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Site Location and Exploration Plans

Contents:

Site Location

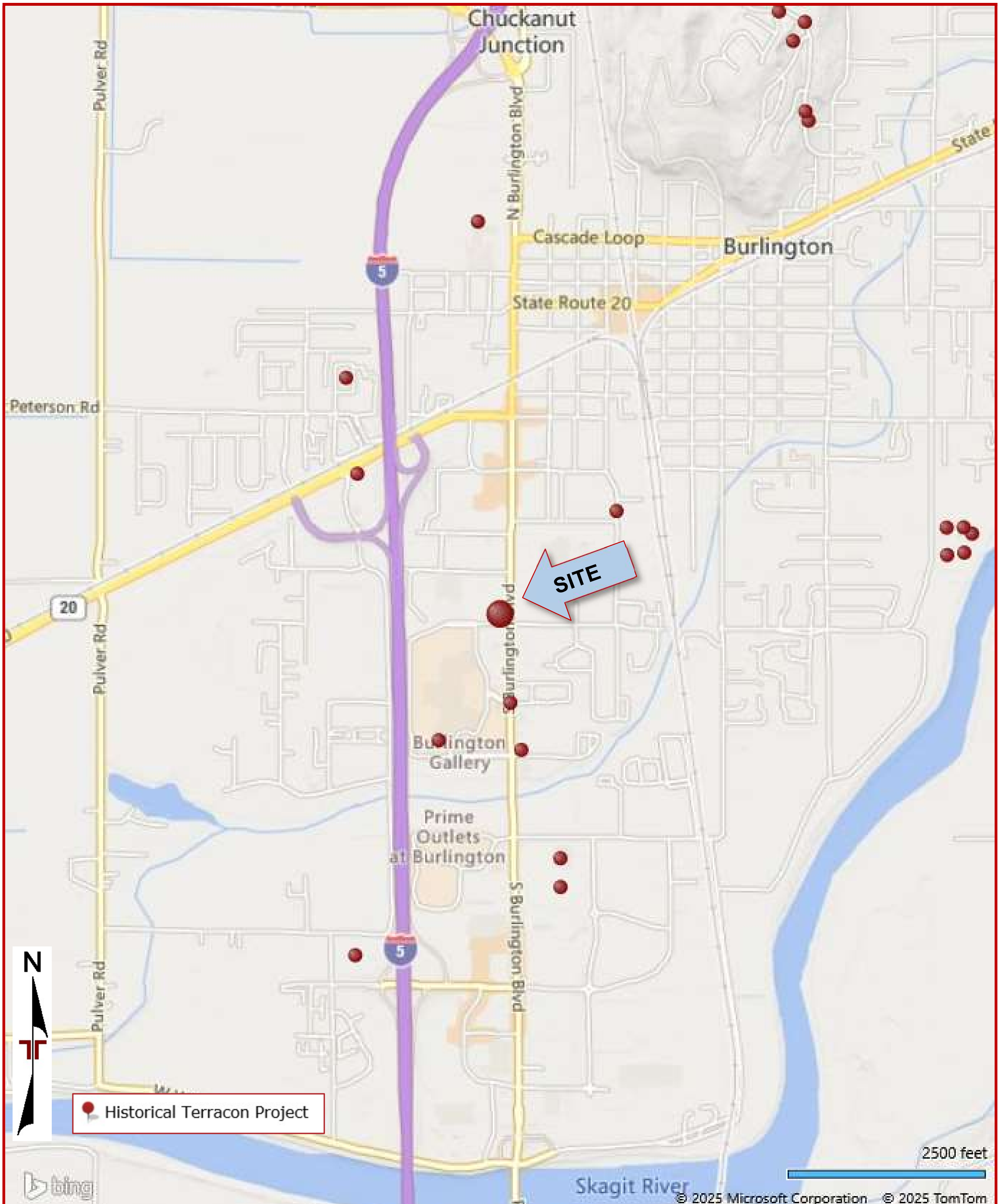


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

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

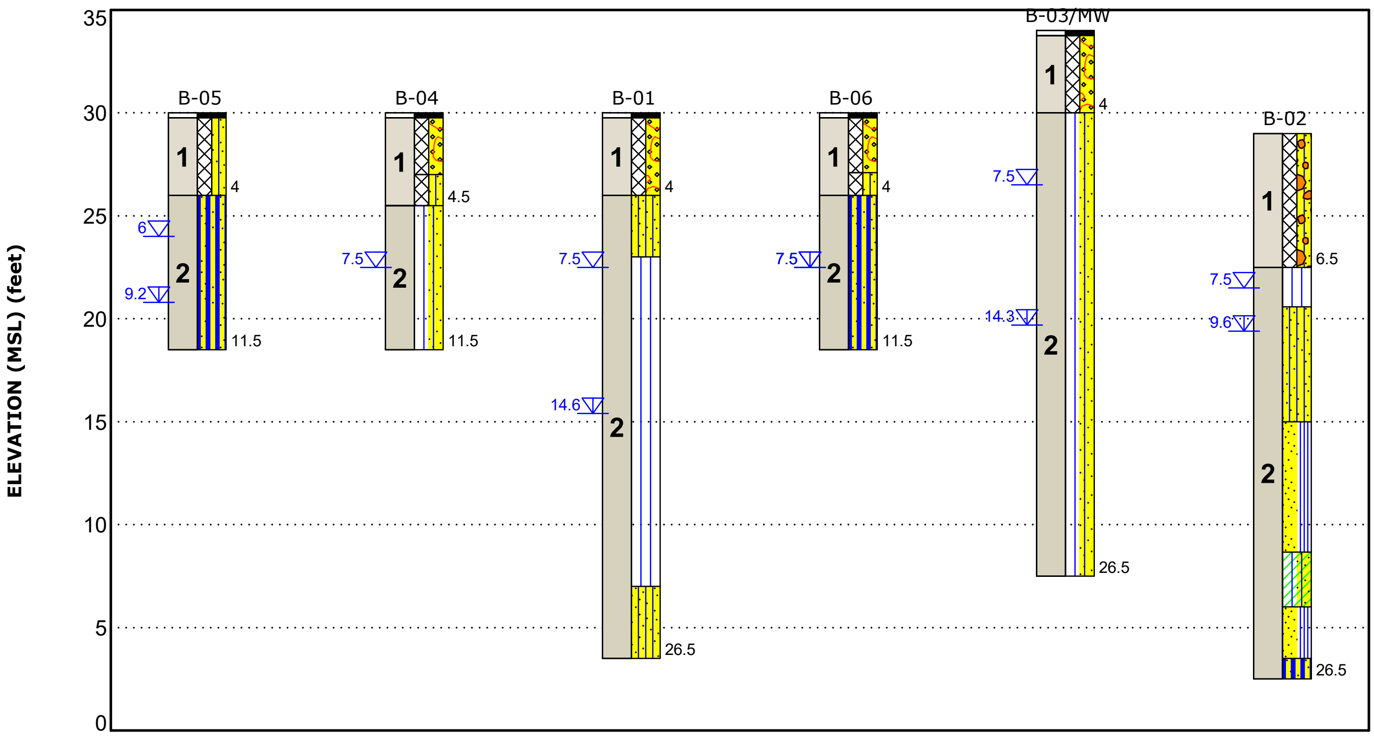
Exploration and Laboratory Results

Contents:

GeoModel
Boring Logs (B-01 through B-06)
Atterberg Limits
Grain Size Distribution
CPT Logs (sCPT-01 and CPT-02)

Terracon
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	Existing Fill	Grayish brown, moist, medium dense, well graded GRAVEL with sand (GW) and silty SAND (SM), fine to coarse
2	Native Silt and Sand	Grayish brown to gray, moist to wet, loose to medium dense silty SAND (SM), soft to stiff SILT with variable sand content (ML)

Legend

- Asphalt
- Well-graded Gravel w/sand
- Silty Sand
- Silt
- Silty Sand with Gravel
- Poorly-graded Sand with Silt
- Silty Clay with Sand
- Sandy Silt
- Silt with Sand

First Water Observation
Second Water Observation

Groundwater levels are temporal. The 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. B-01

Location: See Exploration Plan

Latitude: 48.4650° Longitude: -122.3365°

Model Layer	Depth (Ft.)	Description
1	0.3	**ASPHALT**, ~ 3 inches
1	4.0	**FILL - WELL GRADED GRAVEL WITH SAND (GW)**, fine to coarse grained, subrounded to rounded, brownish gray, moist, medium dense
2	7.0	**SILTY SAND (SM)**, fine to medium grained, gray with orange mottling, moist, medium dense
2	23.0	**SILT (ML)**, fine to medium grained, low plasticity, gray, wet, very soft to medium stiff; increasing sand content, grades to SILT with sand
2	26.5	**SILTY SAND (SM)**, fine to medium grained, gray to black, wet, loose

Boring Terminated at 26.5 Feet

Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
8	12-12-8 N=20	S-1			
16	8-8-7 N=15	S-2	17.1		46.8
18	1-0-1 N=1	S-3	37.9		86.9
16	1-3-2 N=5	S-4			
17	3-4-3 N=7	S-5	33.4		77.8
18	2-2-1 N=3	S-6			
18	2-4-4 N=8	S-7	30.0		18.8

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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
- ▽ Inferred from sample moisture while drilling
- ▽ Water meter reading while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: Asphalt drive aisle

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Boring backfilled with bentonite upon completion. and surface capped with cold mix asphalt

Logged by
DN

Boring Started
05-21-2025

Boring Completed
05-21-2025

RC 1344 - Burlington, WA - Geo
S Burlington Blvd | Burlington, WA
Terracon Project No. 81255069

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

Boring Log No. B-02

Location: See Exploration Plan

Latitude: 48.4647° Longitude: -122.3361°

Model Layer	Depth (Ft.)	Material Description	Water Level Observations	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
1	0.0–6.5	**SILTY SAND WITH GRAVEL (SM)**, trace organics, fine to medium grained, brownish gray, moist, loose, **Possible Fill**							
	2.5–4.0			13	6-3-4 N=7	S-1	13.4		34.9
	5.0–6.5			18	3-3-3 N=6	S-2			
2	6.5–8.4	**SILT (ML)**, low plasticity, gray with orange mottling, wet, medium stiff	▽						
	7.5–9.0			18	2-3-5 N=8	S-3A	35.4	34-26-8	
						S-3B			
	8.4–14.0	**SILTY SAND (SM)**, fine to medium grained, gray, wet, loose, with silt interbeds	▽						
	10.0–11.5			12	3-4-4 N=8	S-4	29.9		19.8
	14.0–20.3	**POORLY GRADED SAND WITH SILT (SP-SM)**, fine to medium grained, gray, wet, loose							
	15.0–16.5			18	5-4-4 N=8	S-5	24.5		7.9
	20.0–21.5			18	1-1-1 N=2	S-6A			
	20.3–23.0	**SILTY CLAY WITH SAND (CL-ML)**, trace organics, low plasticity, gray, wet, very soft				S-6B			
	23.0–25.5	**POORLY GRADED SAND WITH SILT (SP-SM)**, fine to medium grained, gray, wet, loose							
	25.0–26.5			18	4-4-2 N=6	S-7A			
	25.5–26.5	**SANDY SILT (ML)**, low plasticity, gray, wet, soft				S-7B			
		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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
▽ Inferred from sample moisture while drilling
▽ Water meter reading while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: light grass vegetation

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Boring backfilled with bentonite upon completion.

Logged by
DN

Boring Started
05-21-2025

Boring Completed
05-21-2025

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

Boring Log No. B-03/MW

Location: See Exploration Plan

Latitude: 48.4650° Longitude: -122.3361°

Model Layer	Depth (Ft.)	Material Description	Installation Details
1	0.3	**ASPHALT**, ~ 3 inches	Concrete
1	4.0	**FILL - WELL GRADED GRAVEL WITH SAND (GW)**, fine to coarse grained, subrounded to rounded, brownish gray, moist, medium dense	Concrete / Bentonite
2	26.5	**SILT WITH SAND (ML)**, with interbedded black wood fibers, low plasticity, brownish gray to gray, moist to wet, very soft to stiff at ~5 feet: orange mottling at ~10 feet: 6.6% organic content	Bentonite / Sand / Screen

Boring Terminated at 26.5 Feet

Depth (Ft.)	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
2.5-4	1	6-10-11 N=21	S-1	1.9		4
5-6.5	15	4-3-4 N=7	S-2			
7.5-9	18	2-1-1 N=2	S-3	36.9		74
10-11.5	18	1-0-1 N=1	S-4	73.2		
15-16.5	13	3-4-3 N=7	S-5	36.2		82
20-21.5	18	1-0-1 N=1	S-6			
25-26.5	18	2-2-8 N=10	S-7	42.8		85

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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
Inferred from sample moisture while drilling
Water meter reading while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: Asphalt drive aisle

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Groundwater monitoring well installed upon completion of boring

Logged by
DN

Boring Started
05-21-2025

Boring Completed
05-21-2025

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

Boring Log No. B-04

Location: See Exploration Plan

Latitude: 48.4652° Longitude: -122.3369°

Model Layer	Depth (Ft.)	Description	Water Level Observations	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
	0.3	**ASPHALT**, ~ 3 inches							
1	3.0	**FILL - WELL GRADED GRAVEL WITH SAND (GW)**, fine to coarse grained, subrounded to rounded, brownish gray, moist, medium dense		16	10-9-8 N=17	S-1A			
1	4.5	**FILL - SILTY SAND (SM)**, fine to medium grained, brownish gray, moist, with orange mottling				S-1B			
2	11.5	**SILT WITH SAND (ML)**, trace organics, fine grained, low plasticity, brownish gray, moist to wet, soft to stiff, with sand interbeds and orange mottling	▽ (~7.5 ft)	15	3-6-4 N=10	S-2			
				18	2-2-2 N=4	S-3	34.6		74.9
				17	2-2-2 N=4	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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
▽ Inferred from sample moisture while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: Asphalt drive aisle

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Boring backfilled with bentonite upon completion. and surface capped with cold mix asphalt

Logged by
DN

Boring Started
05-21-2025

Boring Completed
05-21-2025

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

Boring Log No. B-05

Location: See Exploration Plan

Latitude: 48.4650° Longitude: -122.3370°

Model Layer	Depth (Ft.)	Description	Water Level Observations	Sample Type	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
	0.3	**ASPHALT**, ~ 3 inches								
1	4.0	**FILL - SILTY SAND (SM)**, fine grained, brownish gray, moist, medium dense			12	7-7-6 N=13	S-1	18.0		46.6
2	11.5	**SANDY SILT (ML)**, low plasticity, brownish gray with orange mottling, moist to wet, soft to stiff			18	3-2-1 N=3	S-2			
					18	3-4-5 N=9	S-3			
					16	4-3-3 N=6	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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
Inferred from sample moisture while drilling
Water meter reading while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: Asphalt drive aisle

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Boring backfilled with bentonite upon completion. and surface capped with cold mix asphalt

Logged by
DN

Boring Started
05-21-2025

Boring Completed
05-21-2025

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

Boring Log No. B-06

Location: See Exploration Plan

Latitude: 48.4652° Longitude: -122.3363°

Model Layer	Depth (Ft.)	Description	Sample Depth (Ft.)	Water Level Observations	Recovery (In.)	Field Test Results	SAMPLE ID	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
1	0.3	**ASPHALT**, ~ 3 inches								
1	2.9	**FILL - WELL GRADED GRAVEL WITH SAND (GW)**, fine to coarse grained, subrounded to rounded, brownish gray, moist, medium dense			9	7-6-5 N=11	S-1A			
1	4.0	**FILL - SILTY SAND (SM)**, trace gravel, trace organics, fine to medium grained, brownish gray, moist, medium dense, with orange mottling					S-1B			
2		**SANDY SILT (ML)**, trace gravel, low plasticity, brownish gray to gray, moist to wet, medium stiff	5		9	4-4-3 N=7	S-2			
2				▽	17	2-3-4 N=7	S-3			
2	11.5		10		13	4-4-3 N=7	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.
Elevation Reference: Elevations were interpolated from Google Earth

Water Level Observations
▽ Inferred from sample moisture while drilling
▽ Water meter reading while drilling

Drill Rig
EC 95

Hammer Type
Rope and Cathead

Driller
Boretec

Notes
Surface Conditions: Asphalt drive aisle

Advancement Method
Hollow Stem Auger (8-inch O.D., 4 1/4-inch I.D.)
Sampling perfromed using a 2-inch O.D., 1 3/8-inch I.D. split spoon

Abandonment Method
Boring backfilled with bentonite upon completion. and surface capped with cold mix asphalt

Logged by
DN

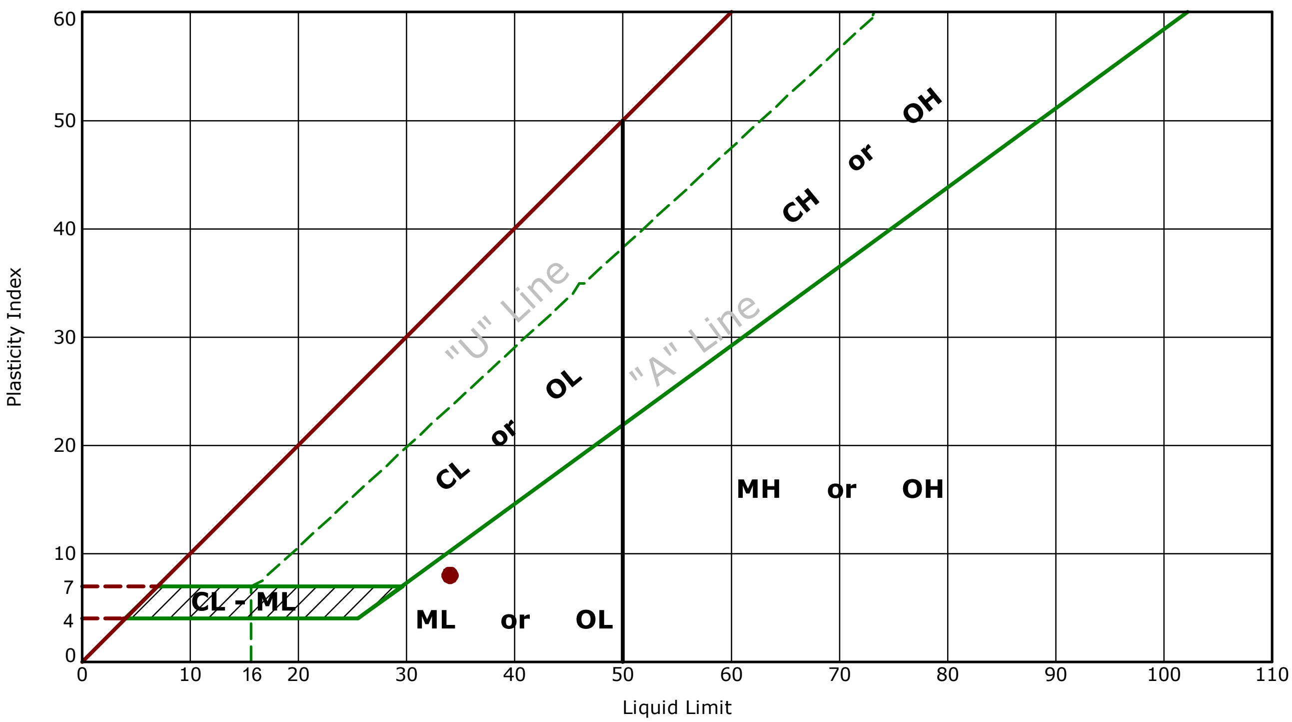
Boring Started
05-21-2025

Boring Completed
05-21-2025

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

Atterberg Limit Results

ASTM D4318

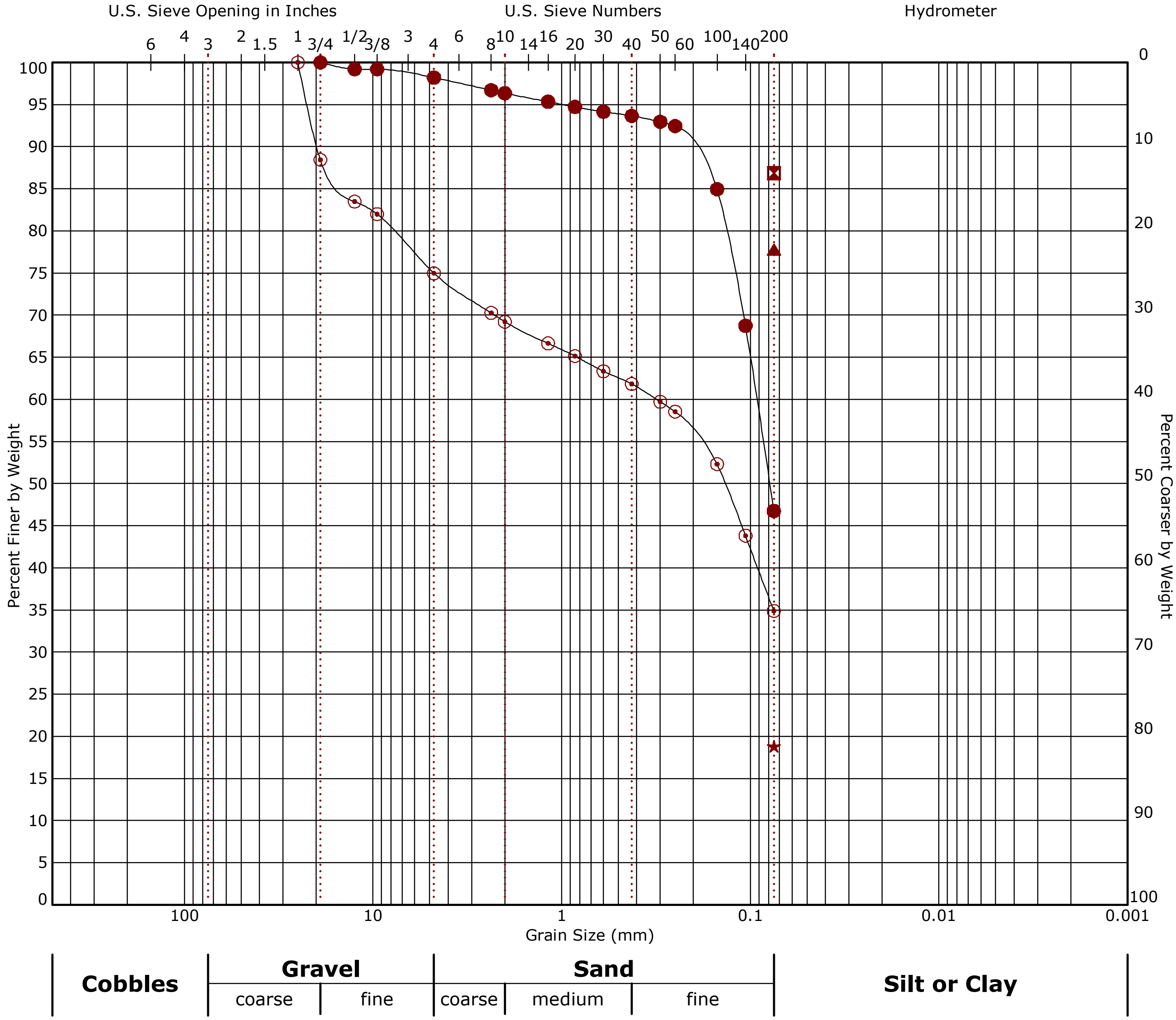


	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	B-02	7.5 - 8.4	34	26	8		ML	SILT

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

Grain Size Distribution

ASTM D422 / ASTM C136 / AASHTO T27



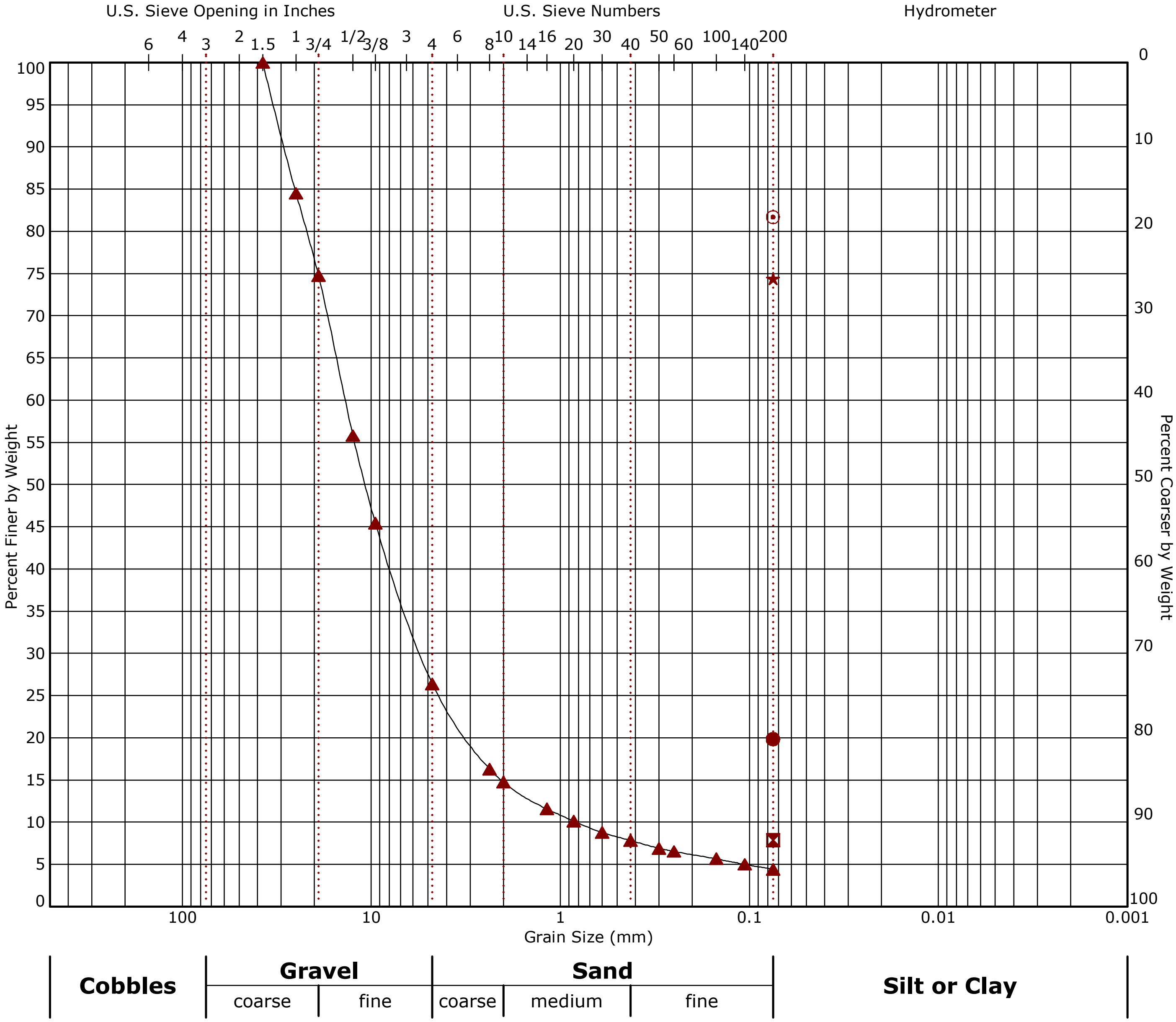
	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	B-01	5 - 6.5	silty SAND	SM					
⊠	B-01	7.5 - 9	SILT	ML					
▲	B-01	15 - 16.5	SILT with sand	ML					
★	B-01	25 - 26.5	silty SAND	SM					
⊙	B-02	2.5 - 4	silty SAND with gravel	SM					

	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-01	5 - 6.5	19	0.092			0.0	1.8	51.4	46.8		
⊠	B-01	7.5 - 9	0.075							86.9		
▲	B-01	15 - 16.5	0.075							77.8		
★	B-01	25 - 26.5	0.075							18.8		
⊙	B-02	2.5 - 4	25	0.314			0.0	25.0	40.1	34.9		

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

Grain Size Distribution

ASTM D422 / ASTM C136 / AASHTO T27



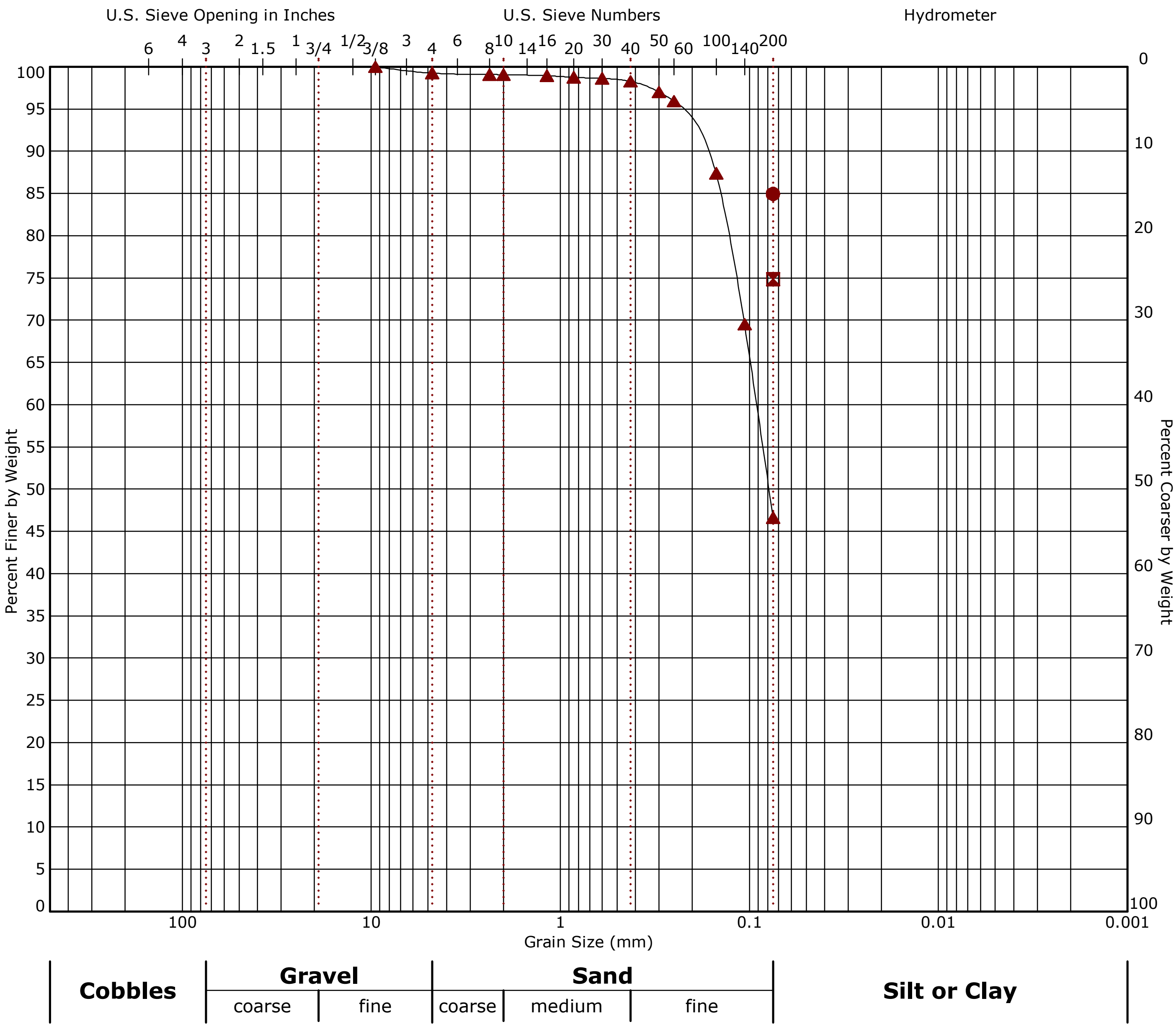
	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	B-02	10 - 11.5	silty SAND	SM					
⊠	B-02	15 - 16.5	poorly graded SAND with silt	SP-SM					
▲	B-03/MW	2.5 - 4	well graded GRAVEL with sand	GW				2.60	16.66
★	B-03/MW	7.5 - 9	SILT with sand	ML					
⊙	B-03/MW	15 - 16.5	SILT with sand	ML					

	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-02	10 - 11.5	0.075							19.8		
⊠	B-02	15 - 16.5	0.075							7.9		
▲	B-03/MW	2.5 - 4	37.5	13.712	5.415	0.823	0.0	73.6	22.0	4.4		
★	B-03/MW	7.5 - 9	0.075							74.4		
⊙	B-03/MW	15 - 16.5	0.075							81.7		

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

Grain Size Distribution

ASTM D422 / ASTM C136 / AASHTO T27



Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
● B-03/MW	25 - 26.5	SILT with sand	ML					
⧗ B-04	7.5 - 9	SILT with sand	ML					
▲ B-05	2.5 - 4	silty SAND	SM					

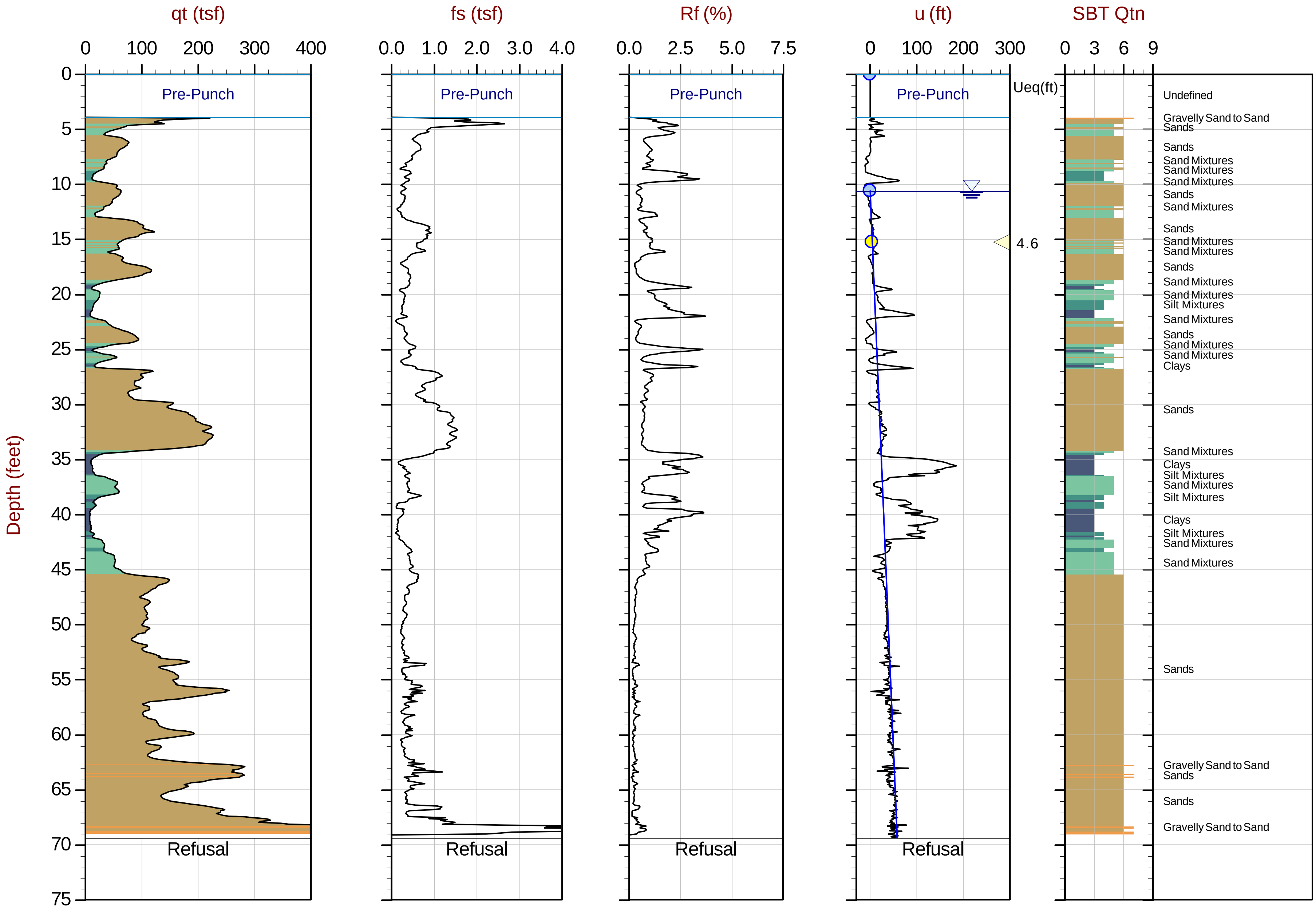
Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
● B-03/MW	25 - 26.5	0.075							84.9		
⧗ B-04	7.5 - 9	0.075							74.9		
▲ B-05	2.5 - 4	9.5	0.092			0.0	0.8	52.6	46.6		

ConeTec

Terracon

Job No: 25-59-29595
Date: 2025-05-21 08:09
Site: RC Burlington CPT

Sounding: SCPT-01
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.150 m / 69.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46508 Long: -122.33634

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

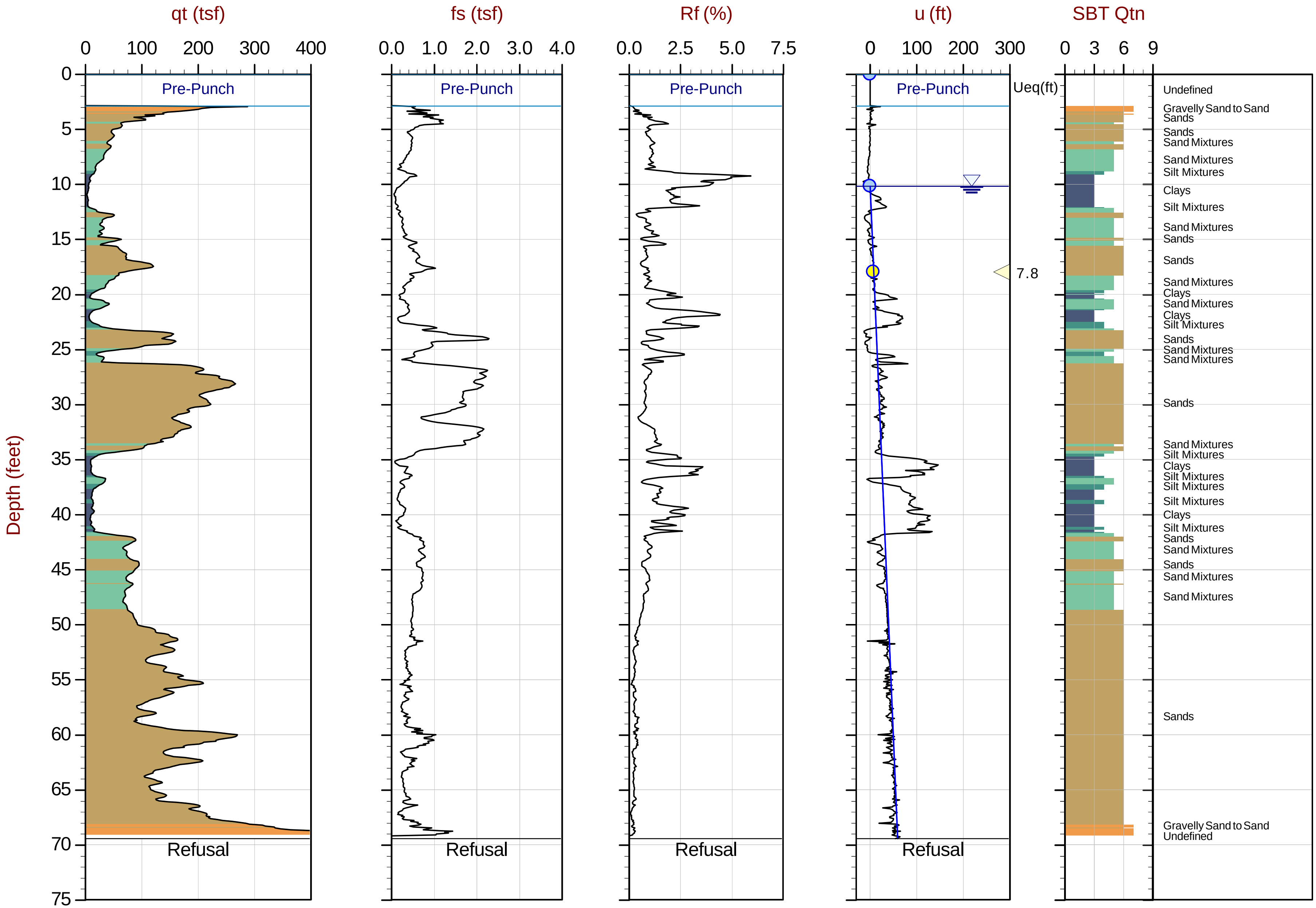
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 09:58
Site: RC Burlington CPT

Sounding: CPT-02
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.175 m / 69.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46480 Long: -122.33666

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.

Supporting Information

Contents:

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

General Notes

Sampling	Water Level	Field Tests
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	5 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	9 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	16 - 30
		Hard	> 4.00	> 30

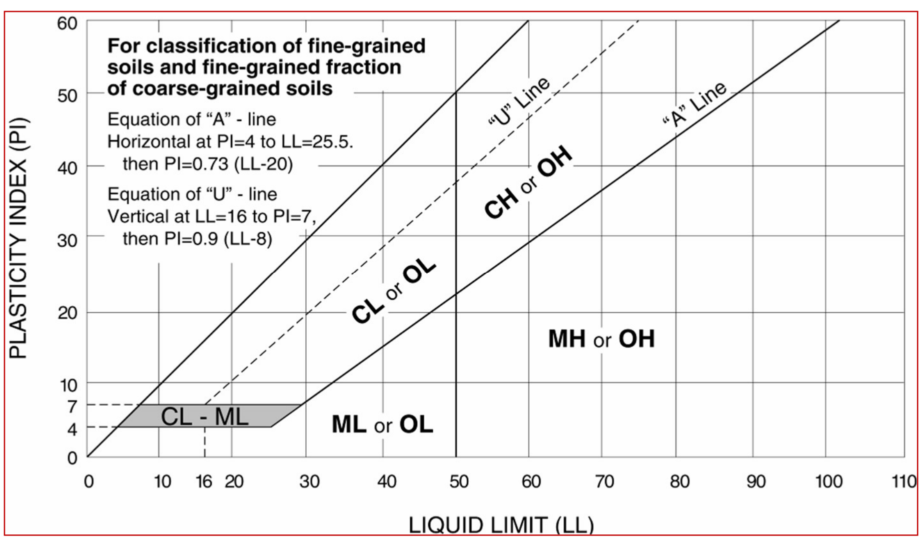
Relevance of Exploration and Laboratory Test Results

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]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [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]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [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

RC Burlington CPT

Prepared for:
Terracon

ConeTec Job No:	**25-59-29595**
Project Start Date:	2025-05-21
Project End Date:	2025-05-21
Release Date:	2025-06-06

Report prepared by:
ConeTec, Inc.

1237 S Director Street, Seattle, WA 98108
Tel: (253) 397-4861

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

ABOUT THIS REPORT

The attached report presents the findings of the site investigation program.

At the request of: Terracon.

Conducted by: ConeTec, Inc.

Please be advised that this report, along with all associated data, is subject to the Third-Party Disclaimer and the Client Disclaimer contained in the 'Limitations' section of this report. For further reference, please consult the list of attached documents following the main body of the report.

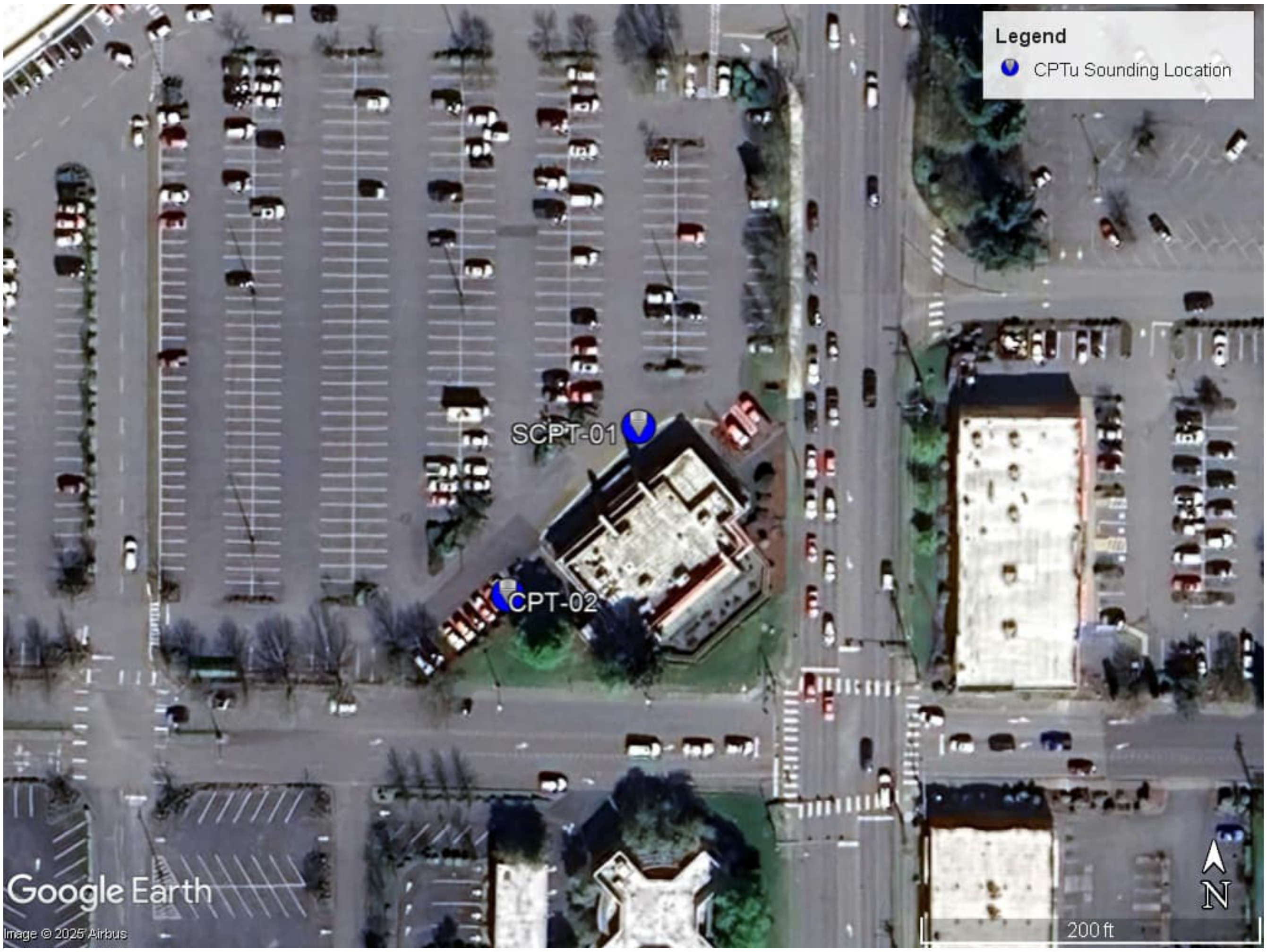
PROJECT	
Client Name	Terracon
Project Name	RC Burlington CPT
Test Types	SCPTu, CPTu
ConeTec Project Number	25-59-29595
Additional Comments	None

CONTENTS

The following are included in the body of the report:

- Site Map
- Limitations and Closure
- Project Information
- Test Summaries and Plots
- Supporting Documents and Materials

SITE MAP



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

ConeTec Job Number: 25-59-29595

Client: Terracon

Project: RC Burlington CPT

Date: 2025-06-06

LIMITATIONS

Third-Party Disclaimer

‘Report’ refers to this document titled: RC Burlington CPT

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. Third parties who gain access to the Report do not acquire any rights by virtue 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

‘Report’ refers to this document titled: RC Burlington CPT

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 warranties, either expressed or implied, with respect to the Data is made by ConeTec. To fully understand the Data included in the Report, reference must be made to the supporting documents 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. 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 contribute to this project. The equipment utilized, as well as the field procedures followed, fully complied with currently accepted best practice standards.

Report prepared by: Jesse Martinez

PROJECT INFORMATION

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

Coordinates			
Test Type	Number of Locations	GPS Collection Method	EPSG Number
CPTu	1	Consumer Grade GPS	4326 (WGS84 / LatLong)
SCPTu	1	Consumer Grade GPS	4326 (WGS84 / LatLong)

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC859:T1500F15U35	859	15	225	1500	15	35
The CPTu summary indicates which cone was used for each sounding.						

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).

REPORT APPENDICES

The appendices listed below are included in the report:

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

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No: 25-59-29595
Client: Terracon
Project: RC Burlington CPT
Start Date: 2025-05-21
End Date: 2025-05-21

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]
SCPT-01	25-59-29595_SP01	2025-05-21	859:T1500F15U35	15	10.6	69.39	22	48.46508	-122.33635
CPT-02	25-59-29595_CP02	2025-05-21	859:T1500F15U35	15	10.2	69.47		48.46481	-122.33666
Totals	2 Soundings					138.86 ft	22		

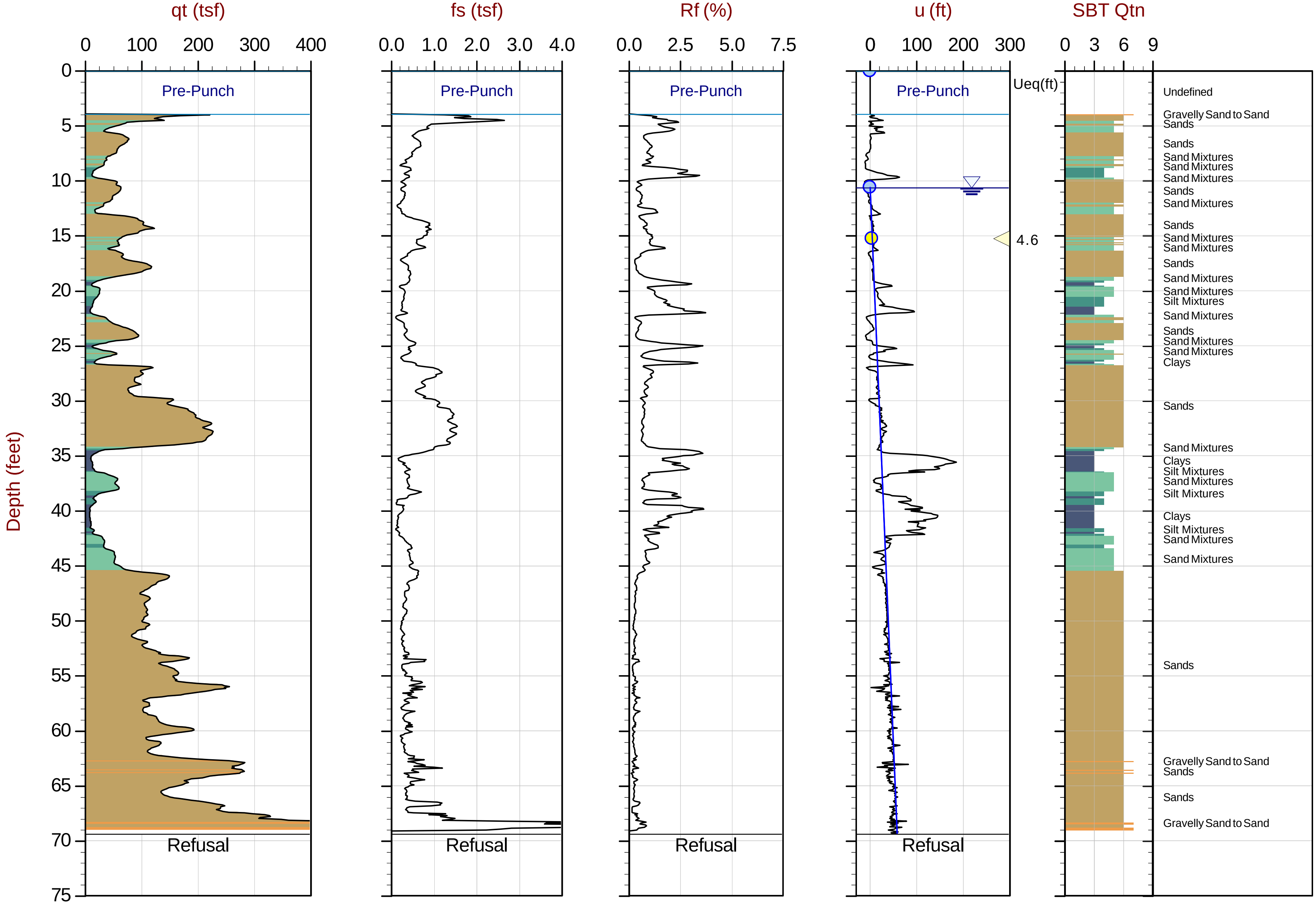
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 08:09
Site: RC Burlington CPT

Sounding: SCPT-01
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.150 m / 69.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46508 Long: -122.33634

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

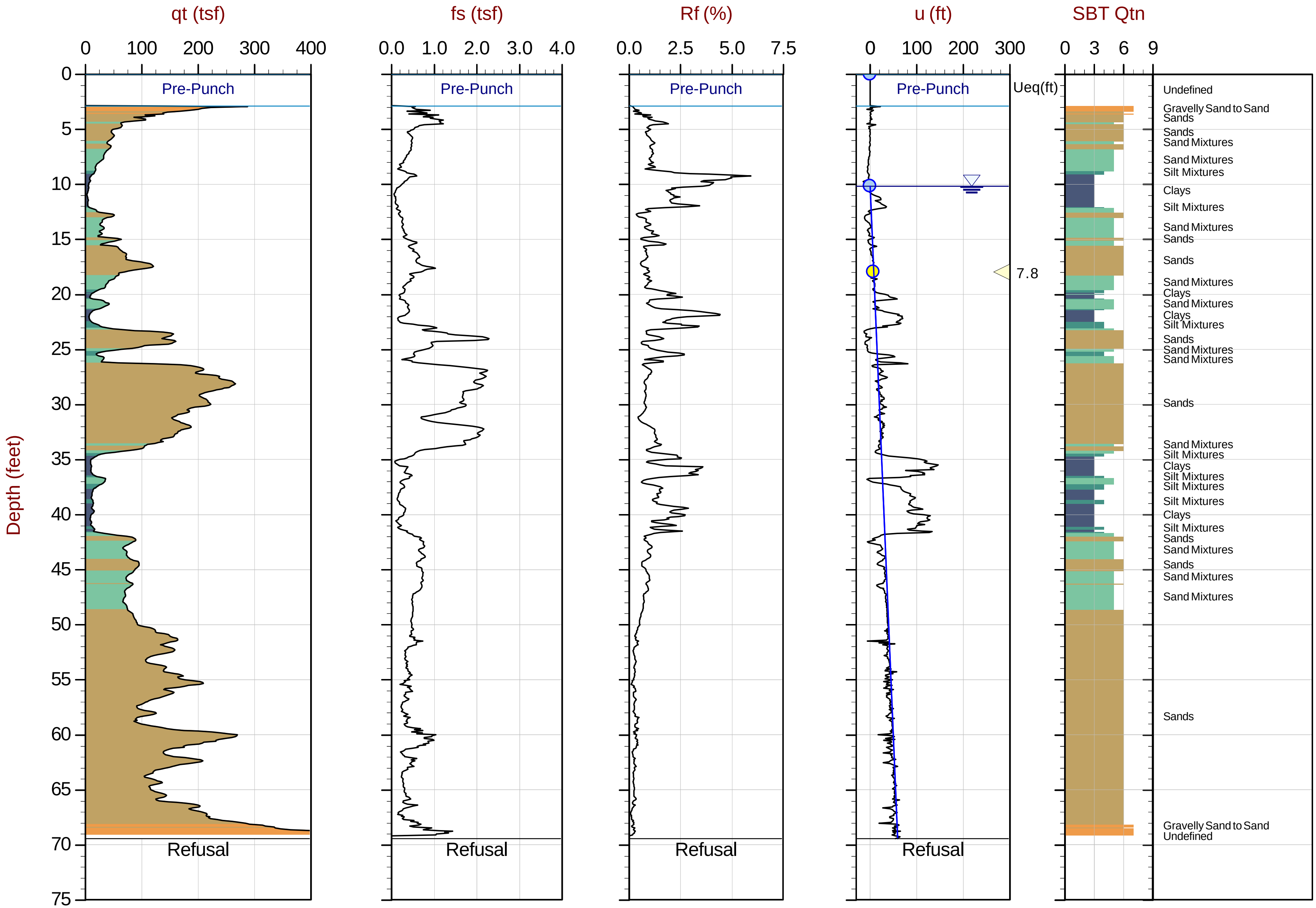
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 09:58
Site: RC Burlington CPT

Sounding: CPT-02
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.175 m / 69.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46480 Long: -122.33666

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.

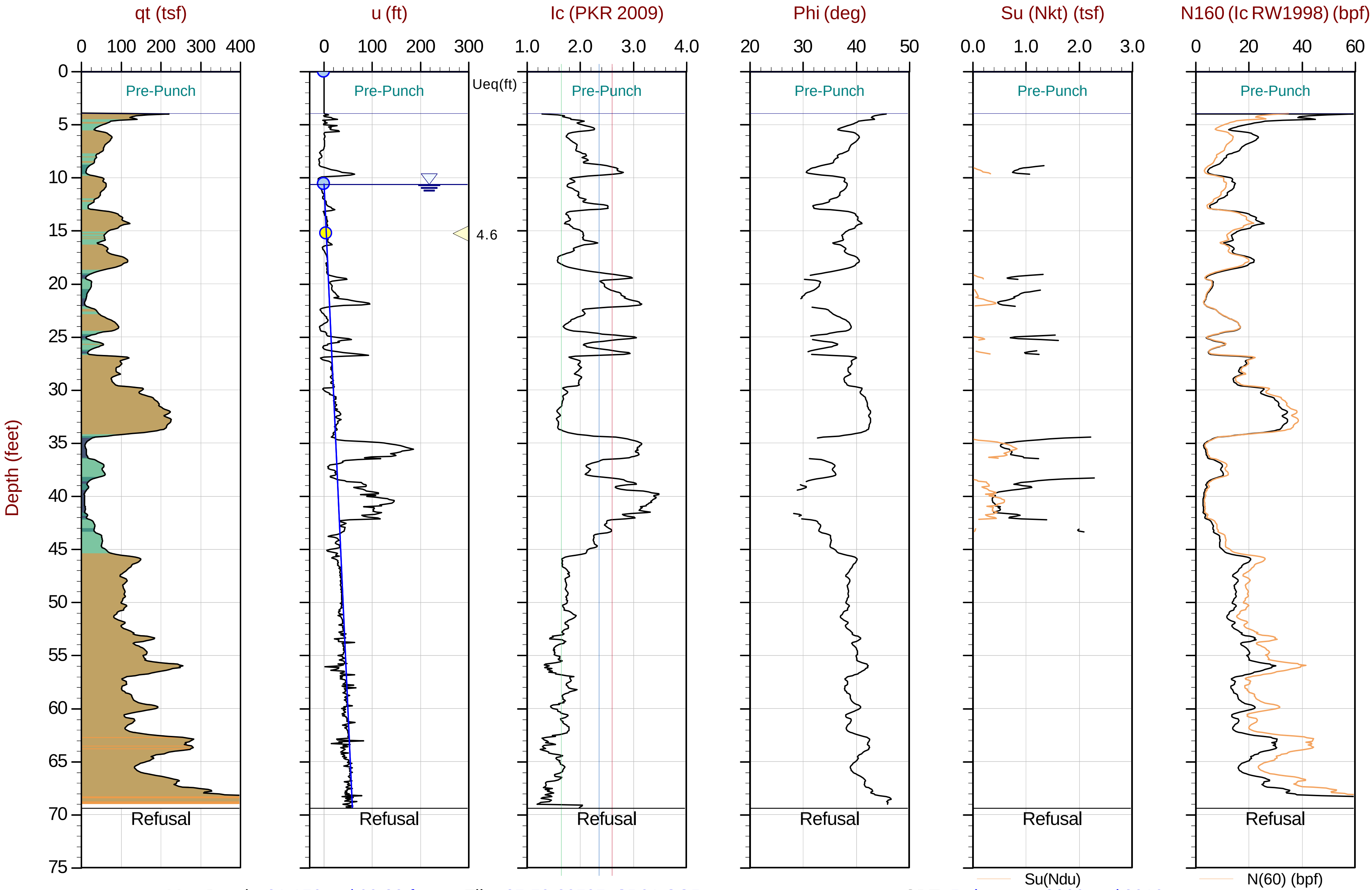
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 08:09
Site: RC Burlington CPT

Sounding: SCPT-01
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.150 m / 69.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46508 Long: -122.33634

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

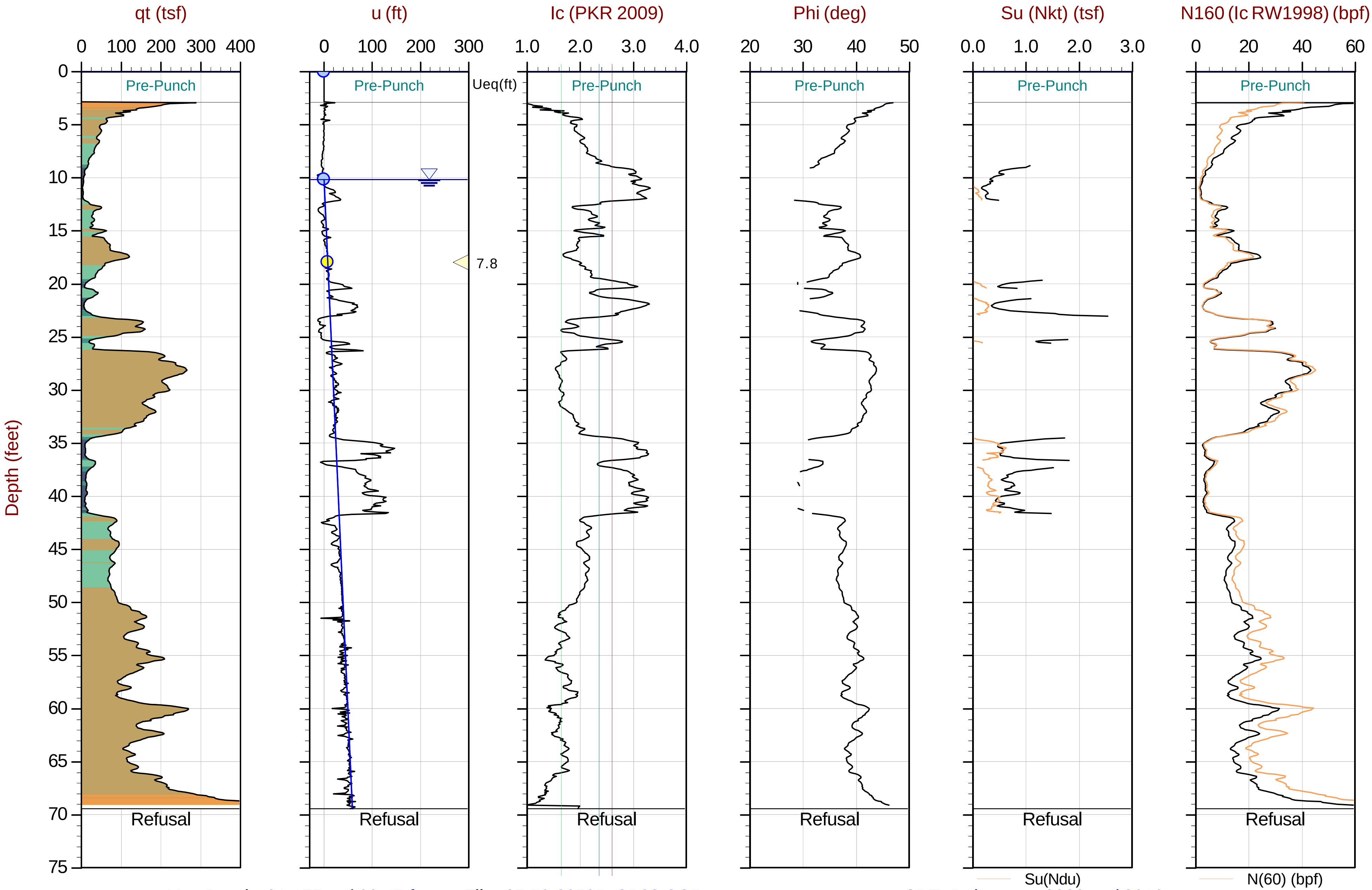
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 09:58
Site: RC Burlington CPT

Sounding: CPT-02
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.175 m / 69.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46480 Long: -122.33666

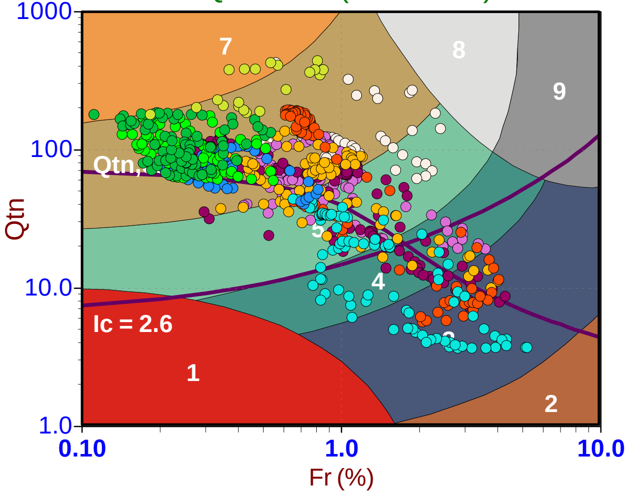
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

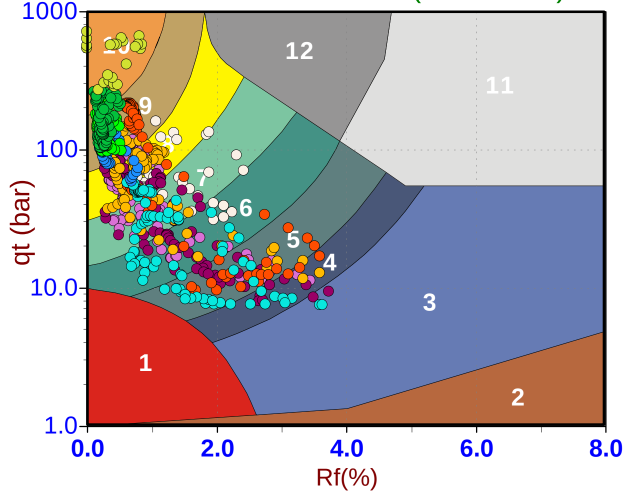
Soil Behavior Type (SBT) Scatter Plots

CONETEC / *Terracon*	Job No: 25-59-29595 Date: 2025-05-21 08:09 Site: RC Burlington CPT	Sounding: SCPT-01 Cone: 859:T1500F15U35 Area=15 cm²

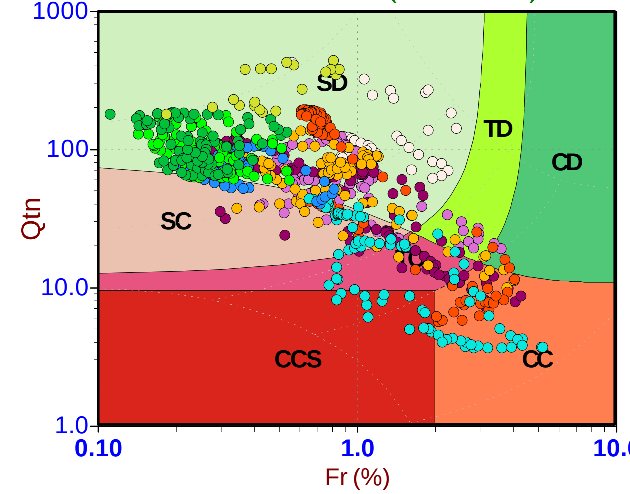
Qtn Chart (PKR 2009)



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

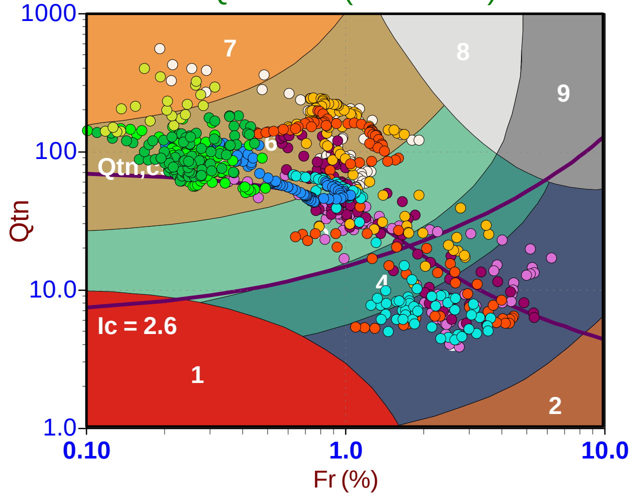
Legend

- 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)

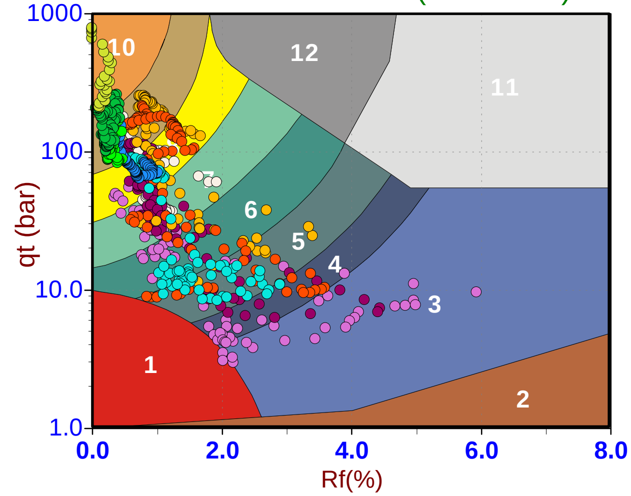
CONETEC

Terracon	Job No: 25-59-29595	Sounding: CPT-02
	Date: 2025-05-21 09:58	Cone: 859:T1500F15U35 Area=15 cm²
	Site: RC Burlington CPT	

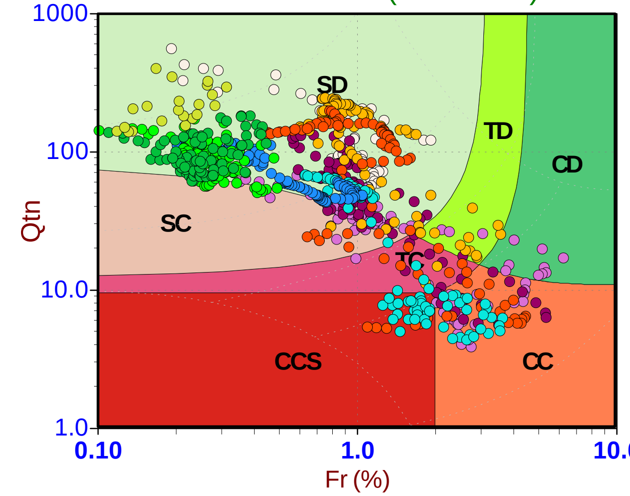
Qtn Chart (PKR 2009)



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 (PPD) Summary and PPD Plots

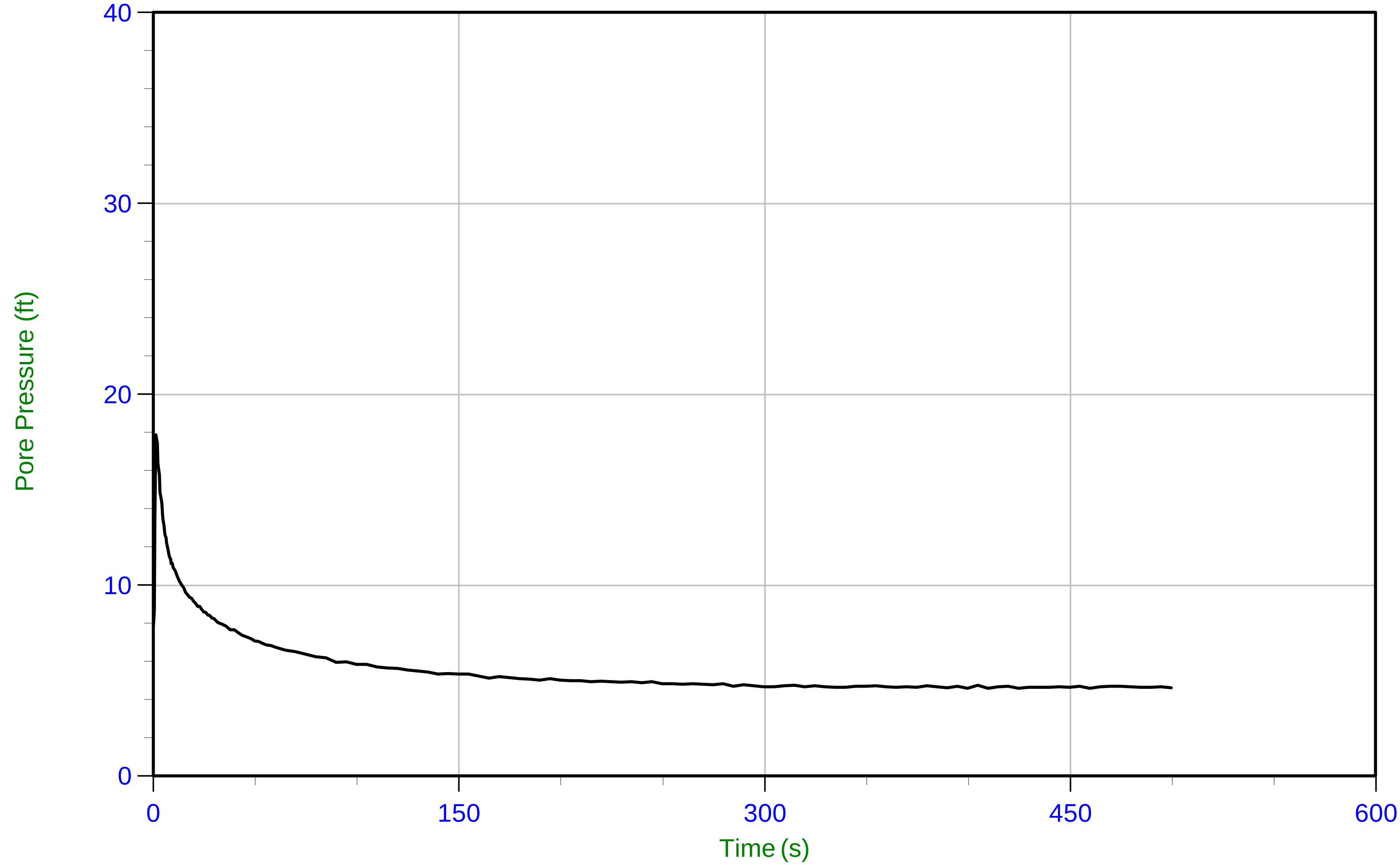
CONETEC

Job No:	25-59-29595
Client:	Terracon
Project:	RC Burlington CPT
Start Date:	2025-05-21
End Date:	2025-05-21

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
SCPT-01	25-59-29595_SP01	15	500	15.26	4.6	10.6	
CPT-02	25-59-29595_CP02	15	400	17.96	7.8	10.2	
Totals			15 min				

Terracon	Job No: 25-59-29595 Date: 2025-05-21 08:09 Site: RC Burlington CPT	Sounding: SCPT-01 Cone: 859:T1500F15U35 Area=15 cm²



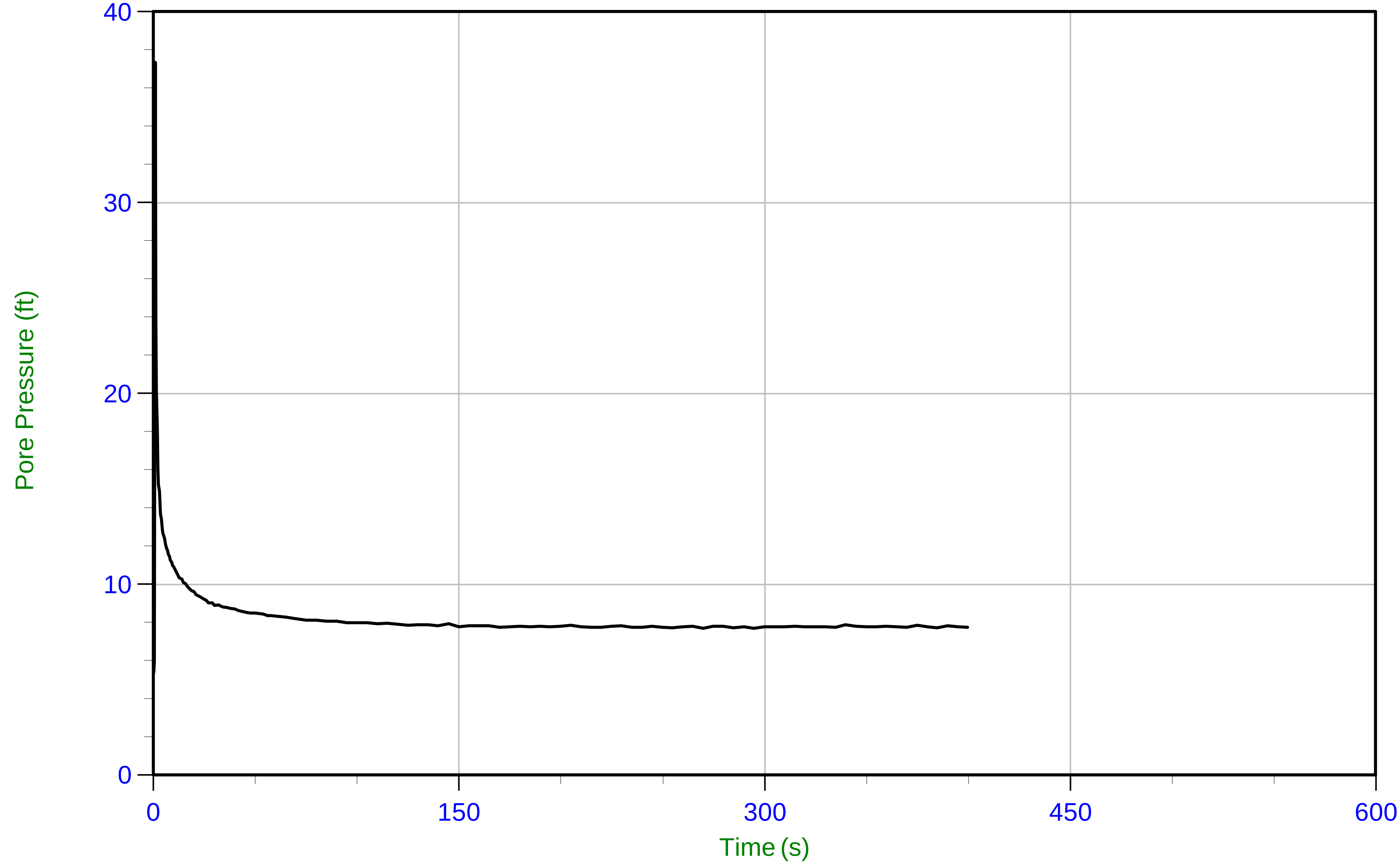
Trace Summary:			
Filename: 25-59-29595_SP01.PPF	u Min: 4.6 ft	WT: 3.24 m / 10.62 ft
Depth: 4.650 m / 15.256 ft	u Max: 17.9 ft	Ueq: 4.6 ft
Duration: 500.0 s	u Final: 4.6 ft	

ConeTec

Terracon

Job No: 25-59-29595
Date: 2025-05-21 09:58
Site: RC Burlington CPT

Sounding: CPT-02
Cone: 859:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 25-59-29595_CP02.PPF
Depth: 5.475 m / 17.962 ft
Duration: 400.0 s

u Min: 5.3 ft
u Max: 37.3 ft
u Final: 7.8 ft

WT: 3.10 m / 10.17 ft
Ueq: 7.8 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

CONETEC	Job No:	25-59-29595
	Client:	Terracon
	Project:	RC Burlington CPT
	Sounding ID:	SCPT-01
	Date:	2025-05-21

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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Sensor Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.92	4.27	4.61			
8.14	7.48	7.68	3.07	4.62	666
11.42	10.76	10.90	3.22	6.25	516
14.76	14.11	14.22	3.31	7.15	463
17.98	17.32	17.41	3.20	6.58	486
21.26	20.60	20.68	3.27	7.79	420
24.44	23.79	23.85	3.17	6.68	475
27.82	27.17	27.22	3.37	5.78	584
31.10	30.45	30.50	3.28	5.62	583
34.38	33.73	33.77	3.28	5.15	636
37.66	37.01	37.05	3.28	5.81	564
40.94	40.29	40.33	3.28	6.41	511
44.23	43.57	43.60	3.28	5.74	571
47.57	46.92	46.95	3.34	4.94	677
50.79	50.13	50.16	3.21	4.75	676
54.07	53.41	53.44	3.28	4.94	665
57.35	56.69	56.72	3.28	4.53	724
60.63	59.97	60.00	3.28	4.00	820
63.98	63.32	63.34	3.35	4.61	725
67.19	66.54	66.56	3.21	3.73	863
69.39	68.73	68.76	2.20	2.24	983

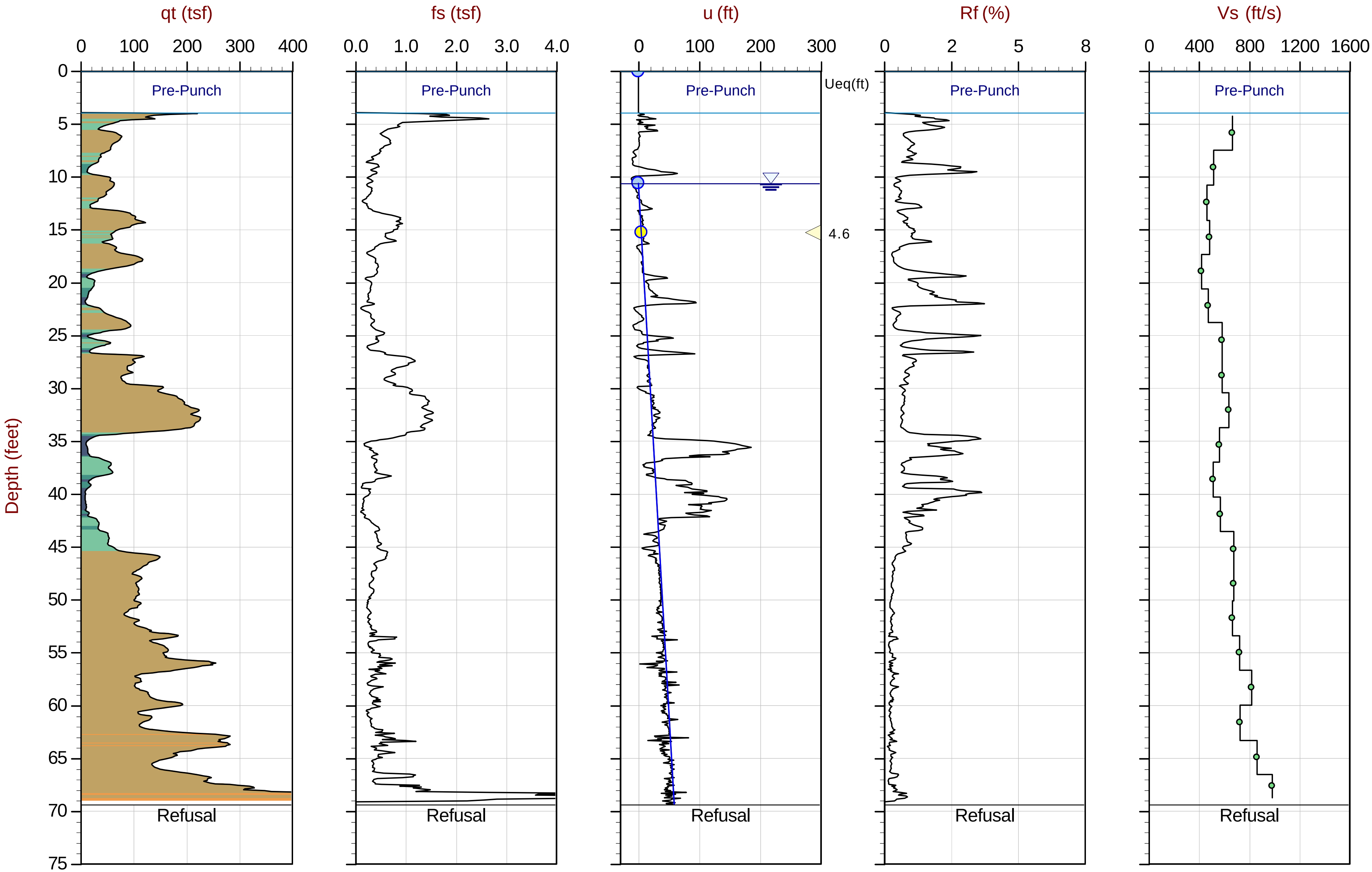
SCPTu Test Plots

CONETEC

Terracon

Job No: 25-59-29595
Date: 2025-05-21 08:09
Site: RC Burlington CPT

Sounding: SCPT-01
Cone: 859:T1500F15U35 Area=15 cm²



Max Depth: 21.150 m / 69.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-59-29595_SP01.COR
Unit Wt: SBTQtn (PKR2009)

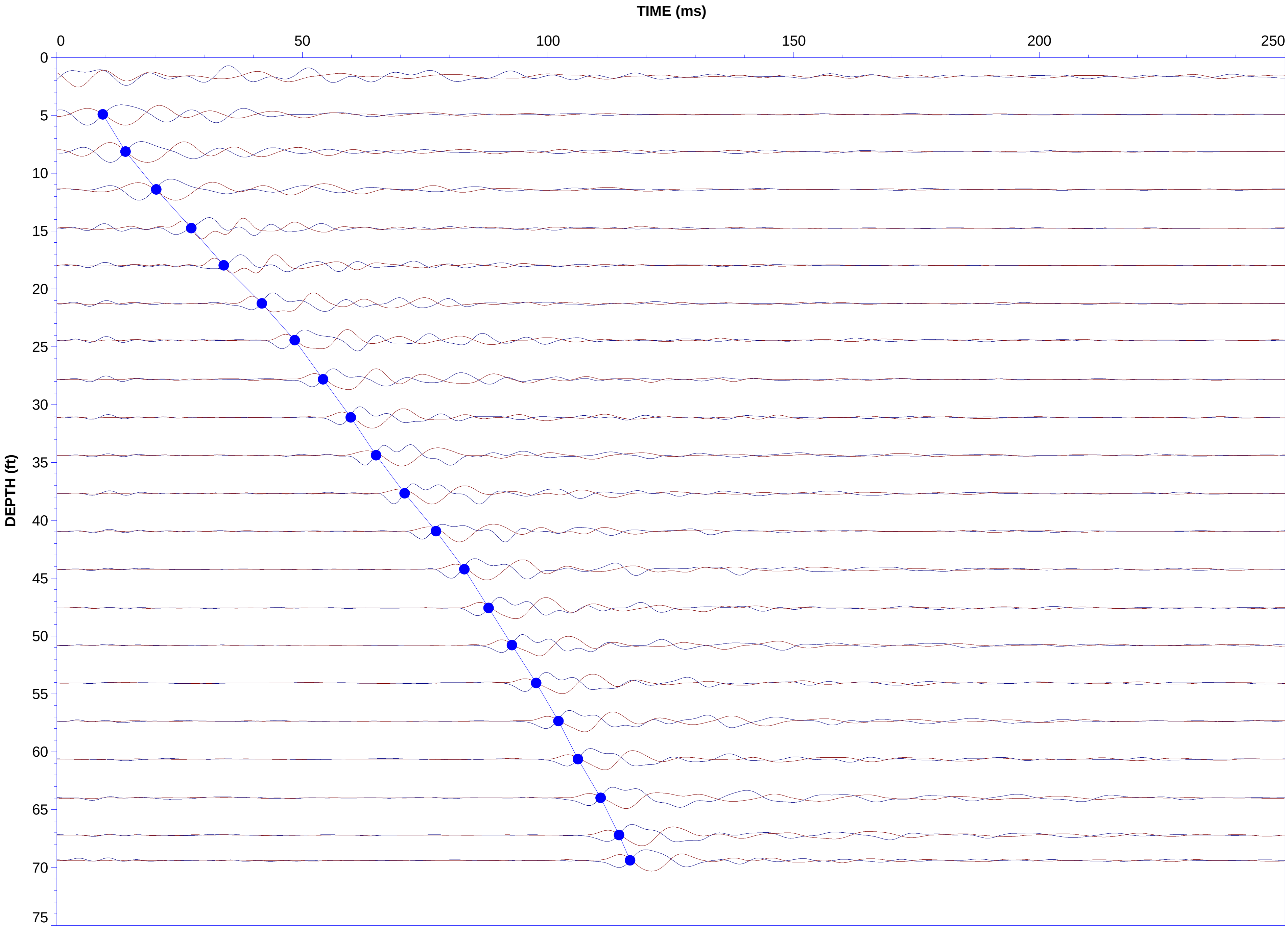
SBT: Robertson, 2009 and 2010
Coords: Lat: 48.46508 Long: -122.33634

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

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

SCPTu Velocity Wave Traces

Job No: 25-59-29595	Client: Terracon	Project: RC Burlington CPT	Analysis: Shear Wave, Geo-X	Sounding: SCPT-01
Filter: BP 0 - 200 HZ	Date: 2025-05-21	Cone: 859:T1500F15U35		



SUPPORTING DOCUMENTS AND MATERIALS

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

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

Methodology Statements

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

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

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

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

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

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

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

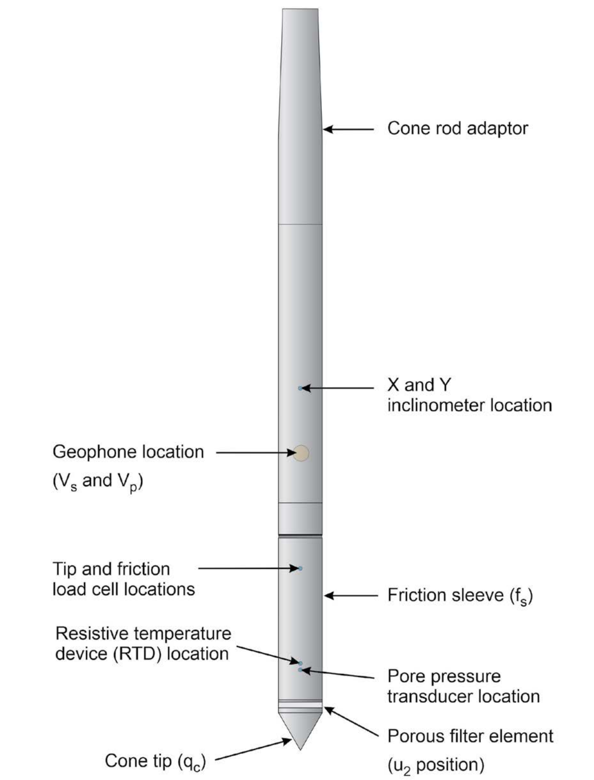


Figure CPTu. Piezocone Penetrometer (15 cm²)

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.

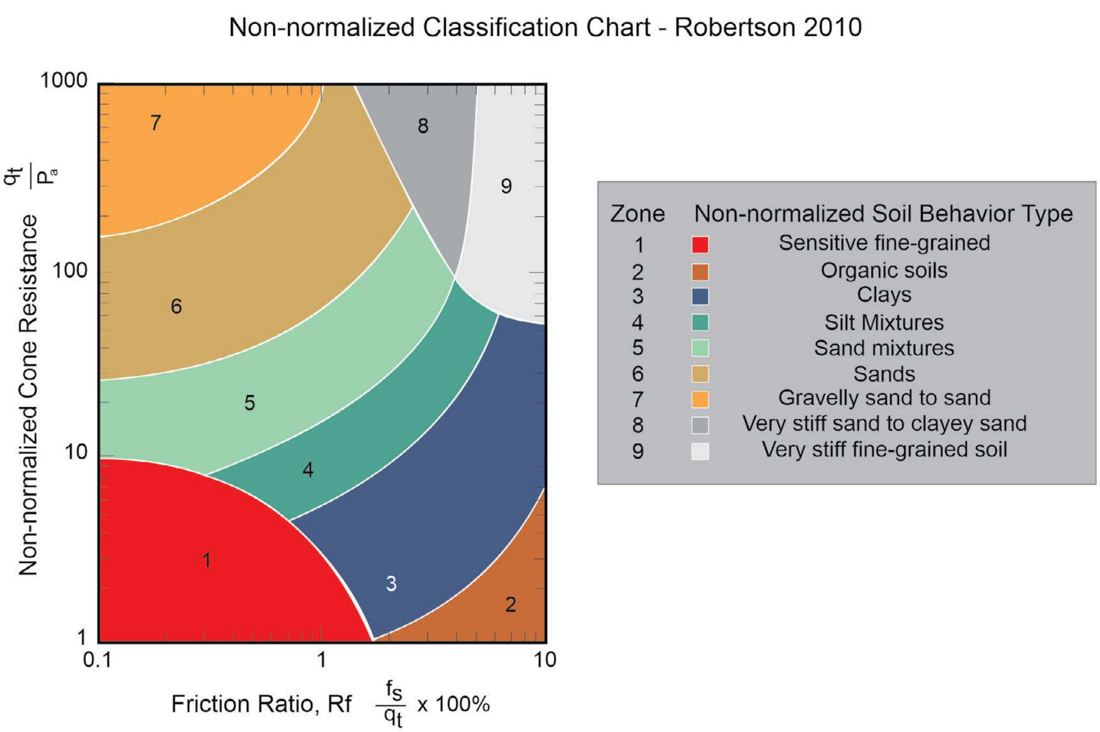


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

CONETEC

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

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

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

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

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

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

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

REFERENCES

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

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

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

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

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

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

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

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

PORE PRESSURE DISSIPATION TEST

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

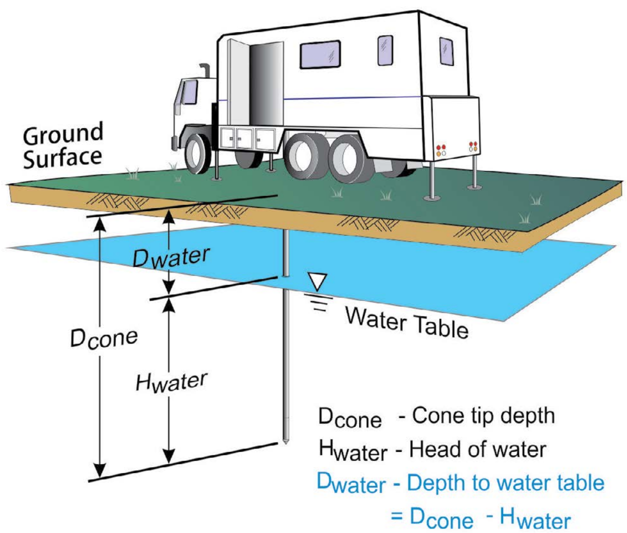


Figure PPD-1. Pore pressure dissipation test setup

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

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

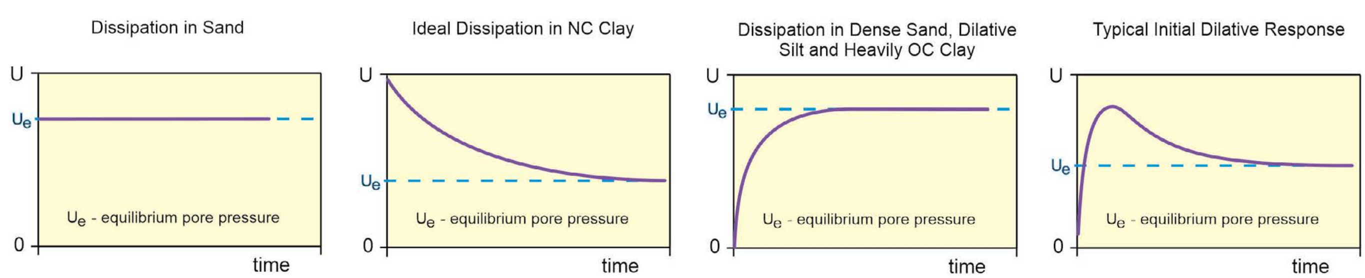


Figure PPD-2. Pore pressure dissipation curve examples

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

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

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

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

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

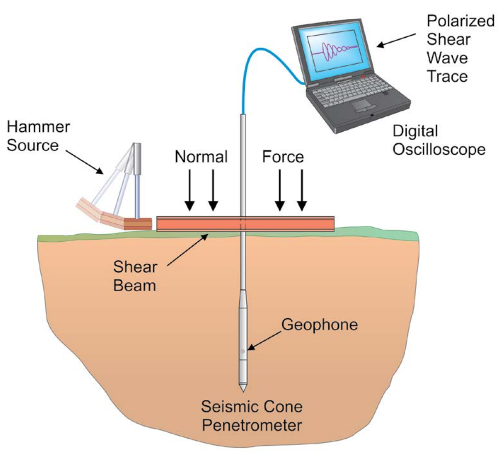


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

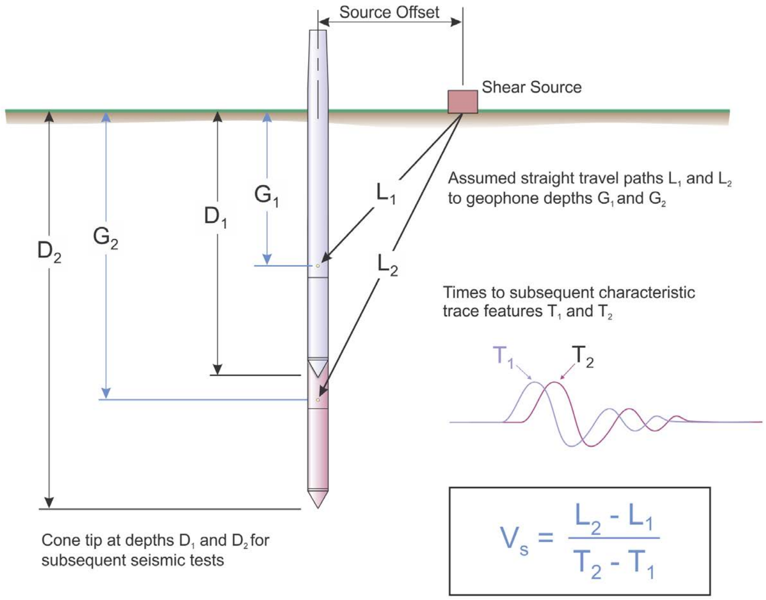


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

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

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

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

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

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

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_s100 or V_s30.

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

REFERENCES

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

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

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

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

Cone Penetration Digital File Formats

ConeTec

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

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

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

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

Header Lines

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

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

Data Records

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

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

End of Data Marker

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

Units Information

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

CPT Data Files (XLS Extension)

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

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

CPT Dissipation Files (XLS Extension)

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

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

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

Data Records

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

Cone Type Designations

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

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

ConeTec

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023

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

CONETEC

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: $q_t = q_c + (1-a) \cdot u_2$ (consistent units are required)

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

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

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

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

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

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

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

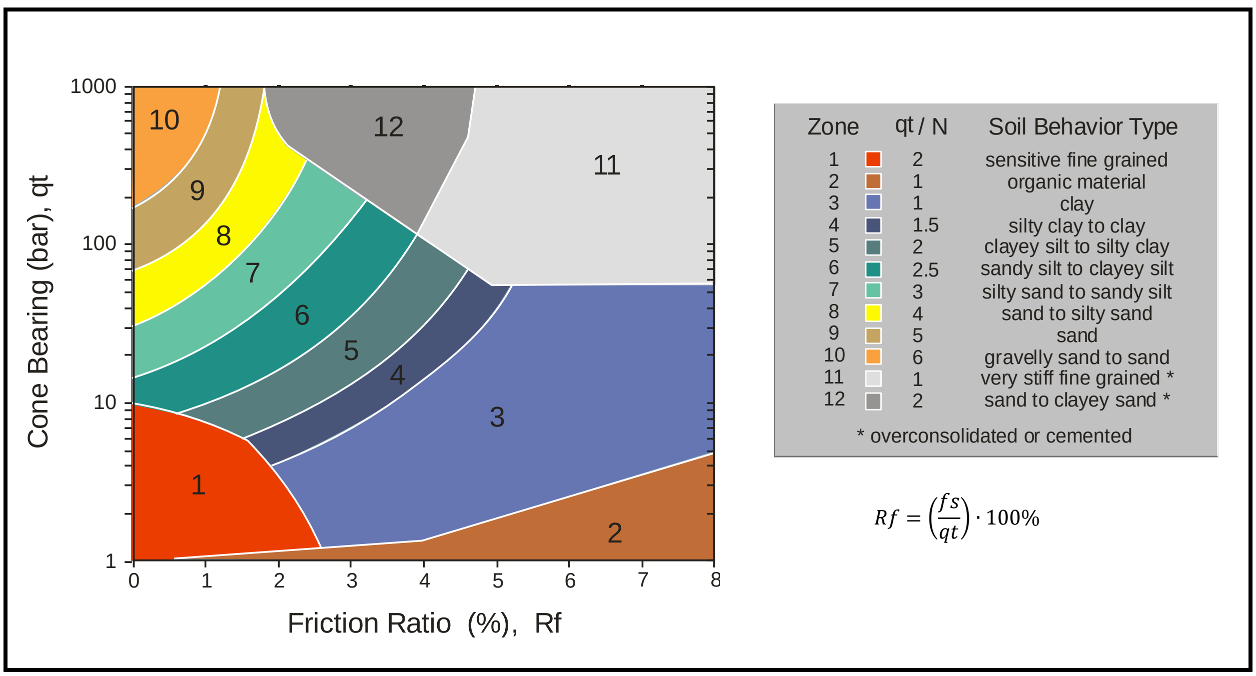


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

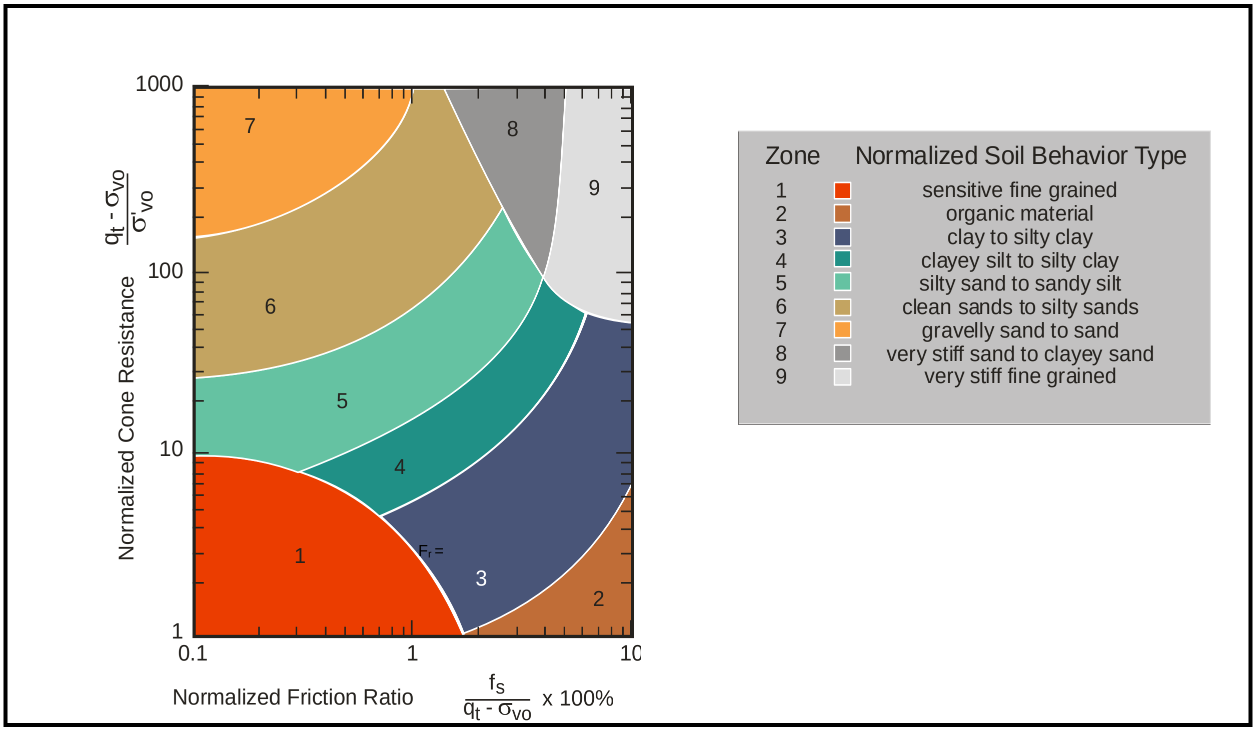


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

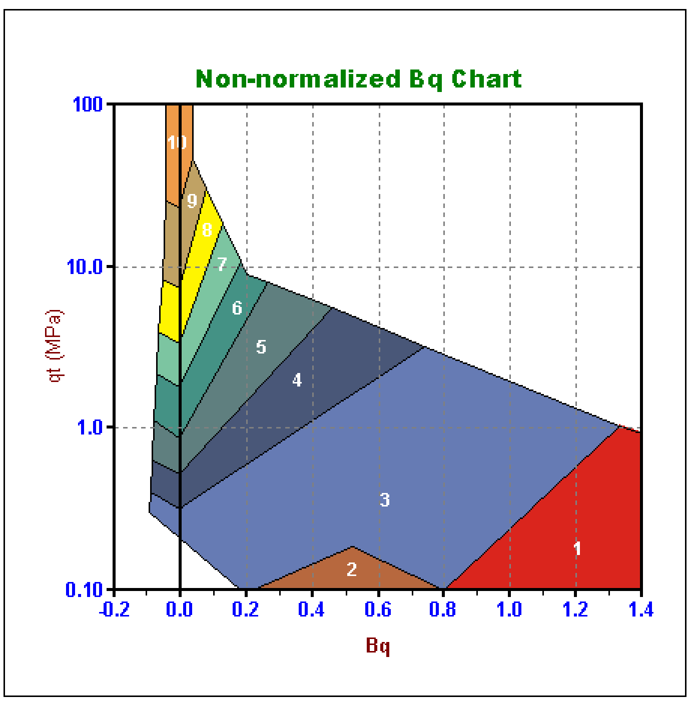


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

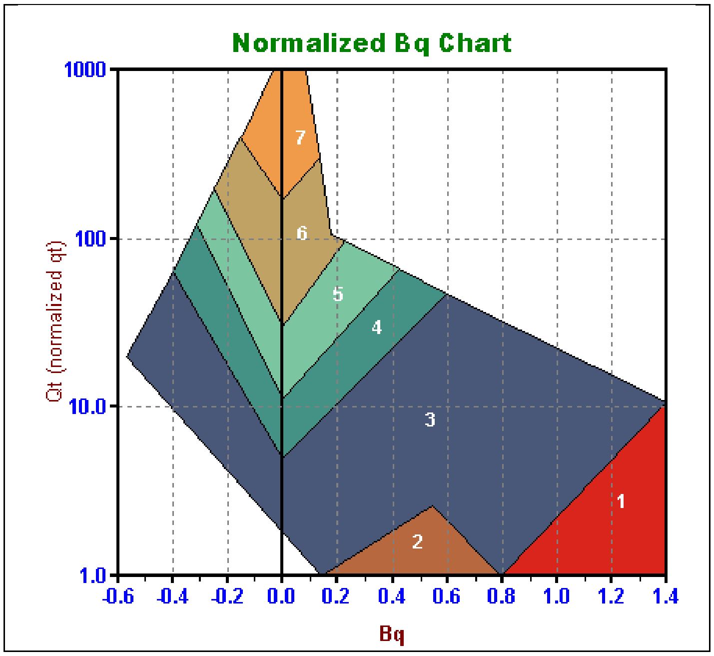


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

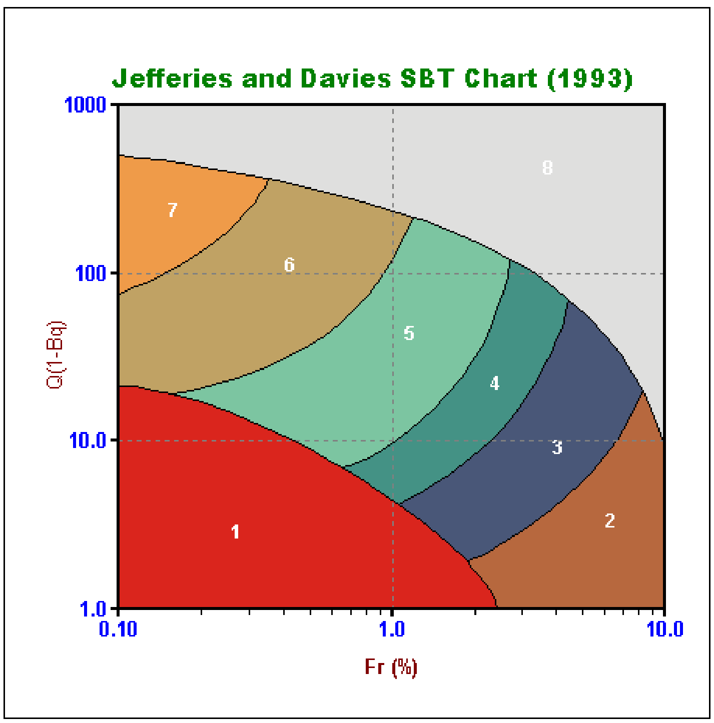


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

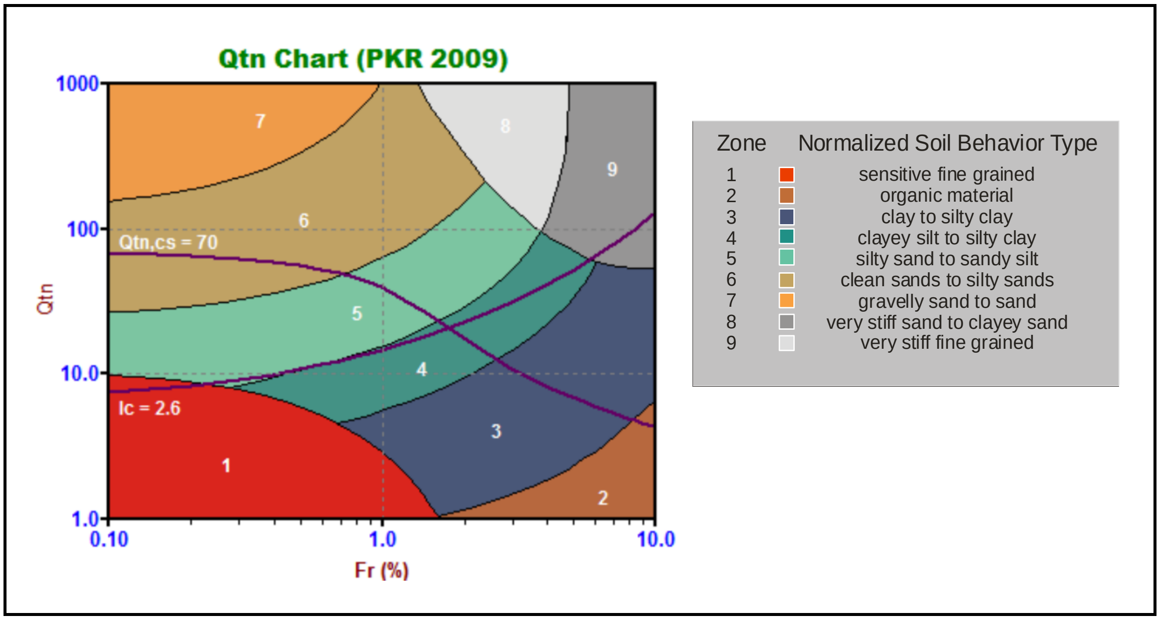


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

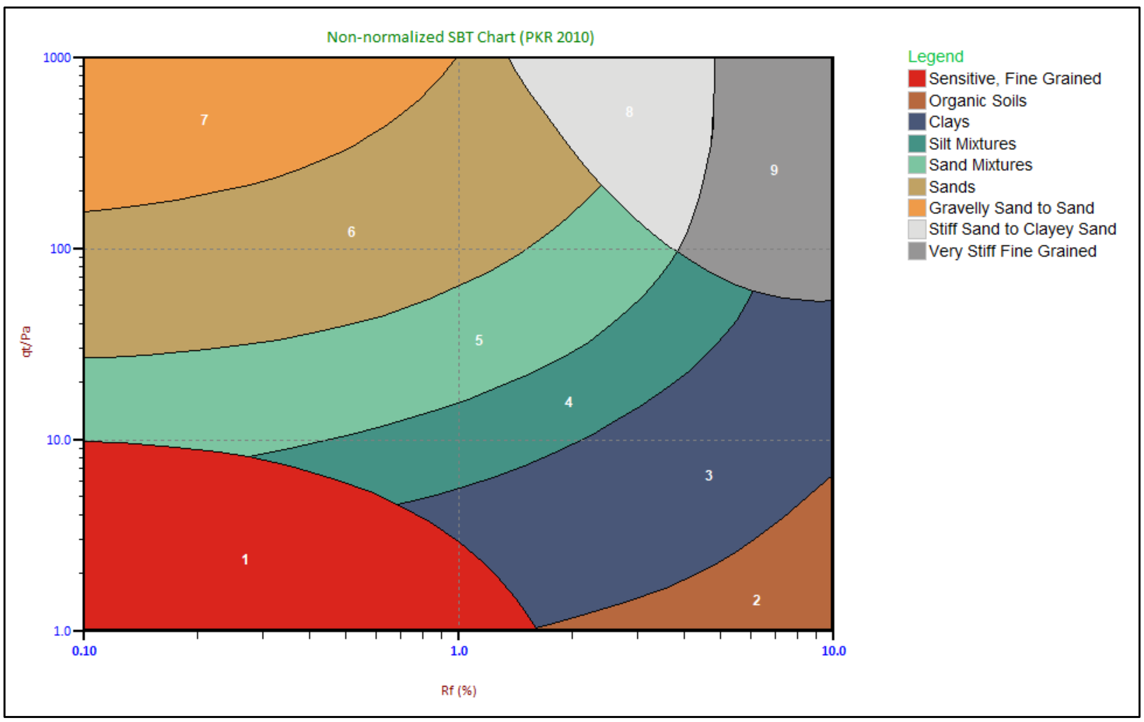


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

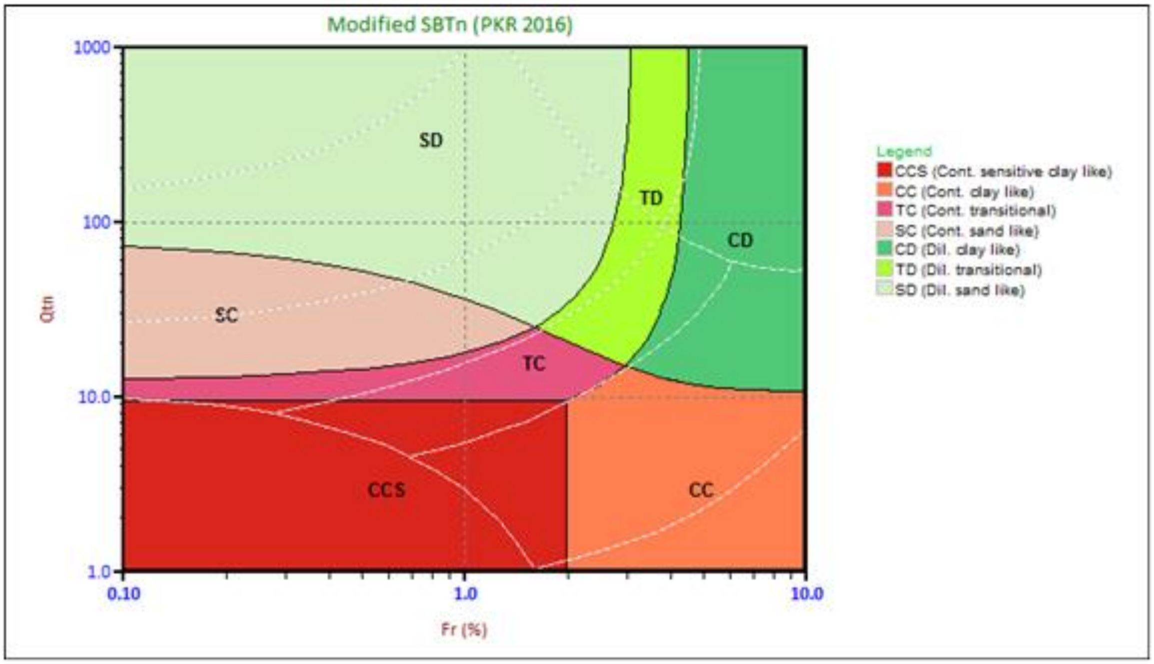


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

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

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

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

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

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

<table>
<tr><th>Calculated Parameter</th><th>Description</th><th>Equation</th><th>Ref</th></tr>
<tr><td>TStress

σ_v</td><td>Total vertical overburden stress at Mid Layer Depth

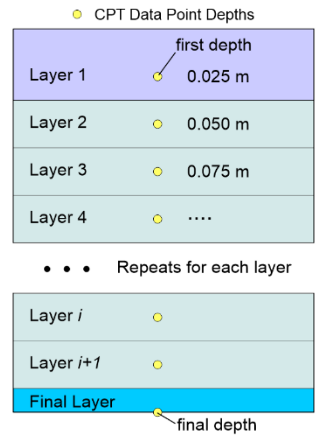
A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.

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.</td><td>$TStress = \sum_{i=1}^{n} \gamma_i h_i$

where γ_i is layer unit weight
h_i is layer thickness


</td><td>CK*</td></tr>
<tr><td>EStress

σ_v'</td><td>Effective vertical overburden stress at mid-layer depth.</td><td>$\sigma_v' = \sigma_v - u_{eq}$</td><td>CK*</td></tr>
<tr><td>Equil u

u_{eq} or u_0</td><td>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.</td><td>For the hydrostatic option:

$u_{eq} = \gamma_w \cdot (D - D_{wt})$

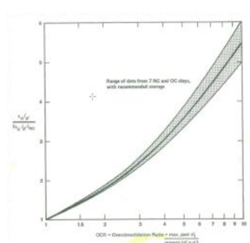
where u_{eq} is equilibrium pore pressure
γ_w is the unit weight of water
D is the current depth
D_{wt} is the depth to the water table</td><td>CK*</td></tr>
<tr><td>K_0</td><td>Coefficient of earth pressure at rest, K_0.</td><td>$K_0 = (1 - \sin\Phi') \, OCR^{\sin\Phi'}$</td><td>17</td></tr>
<tr><td>C_n</td><td>Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.</td><td>$C_n = (P_a/\sigma_v')^{0.5}$

where *$0.0 < C_n < 2.0$ (user adjustable, typically ranging from 1.7 to 2.0)*
P_a is atmospheric pressure (100 kPa)</td><td>4, 12</td></tr>
</table>

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

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

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

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

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

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

Table 2. References

No.	Reference
1	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
2	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, "Evaluating cyclic liquefaction potential using the cone penetration test", Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, " Cone Penetration Testing in Geotechnical Practice," Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, "Use of CPTu to Estimate equivalent N_{60}", Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, "A state parameter for sands", Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, "Guidelines for Cone Penetration Test Performance and Design", Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800", Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, "Liquefied strength ratio from liquefied flow failure case histories", Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, "Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT", Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, 46: 1337-1355.
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.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

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.
20	Mayne, P.W., 2015, “Geocharacterization by In-Situ Testing”, Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, “Liquefaction of sands and its evaluation”, Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo ‘95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, “Soil unit weight estimation from CPTs”, Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, “NCHRP Synthesis 368 on Cone Penetration Test”, Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests.”, Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT’14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, “CPTu-Based State Characterization of Tailings Liquefaction Susceptibility”, Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, “CPTu-Based State Characterization of Tailings Liquefaction Susceptibility”, Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, “Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands”, Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, “The Seismic Cone Penetrometer”, M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
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.
29	Robertson, P.K and Cabal, K.L., 2010, “Estimating soil unit weight from CPT”, Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Huntington Beach, California.
30	Robertson, P.K., 2016, “Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update”, Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, “Interpretation of in-situ tests – some insights”, Mitchell Lecture, ISC’4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, “Guide to Cone Penetration Testing for Geotechnical Engineering”, 6th Edition.
33	Robertson, P.K., 2010b, “Soil behaviour type from CPT: an update”, Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Huntington Beach, California.
34	Been, K., Romero, S., Obermeyer, J. and Hebeler, G., 2012, “Determining in situ state of sand and silt tailings from the CPT”, Tailings and Mine Waster 2012, 325-333.
35	Robertson, P.K., 2010, “Estimating in-situ soil permeability from CPT & CPTu”, Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Huntington Beach, California.
36	Mayne, P.W., Cargill, E. and Greig, J., 2023, “The Cone Penetration Test: A CPT Design Parameter Manual”, ConeTec Group
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
38	Crow, H.L, Hunter, J.A. and Bobrowsky, P.T., 2012, “National shear wave measurement guidelines for Canadian seismic site assessment”, Proceedings of GeoManitoba 2012, the 65th Canadian Geotechnical Conference.
39	Campanella, R.G., Robertson, P.K., Gillespie, D., 1982, “Cone penetration testing in deltaic soils”, Canadian Geotechnical Journal, 20: 23-35.

Calibration Records

ADARA

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

CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC859	Model	A15 T1500 F15 U35
Calibration Date (YYYY-MM-DD)	2025-04-10		
Calibration Due (YYYY-MM-DD)	2026-04-10		
Calibration Performed By	Thanh Nguyen	Signature	Digitally signed by Thanh Nguyen Date: 2025-04-14
Calibration Approved By	Diane Eden	Signature	Digitally signed by Diane Eden Date: 2025-04-14

Lab Condition	As Found	As Left		
Lab Temperature	23 ˚C	23˚C		
Lab Humidity	22%	22%	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	N/A	0.295
Sleeve	bar	N/A	-0.022
Pressure	bar	N/A	0.997
X-Inclinometer	degrees	N/A	-0.025
Y-Inclinometer	degrees	N/A	0.000
Temperature	˚C	N/A	23.081

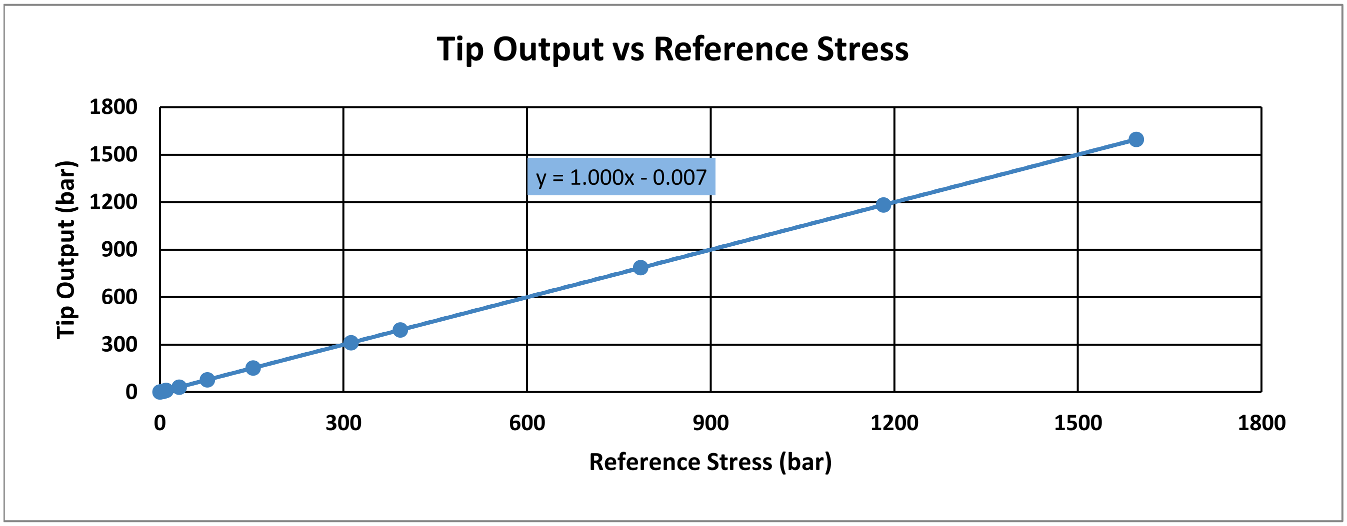
Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

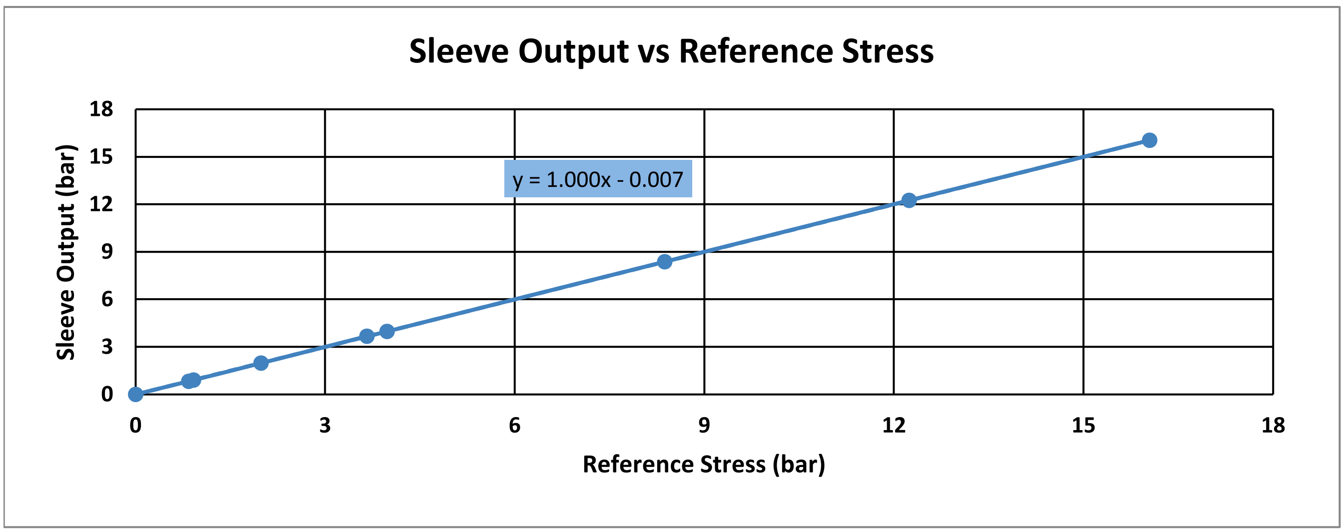
Calibrated in general accordance with the ASTM D5778-20 and D7400-19 standards

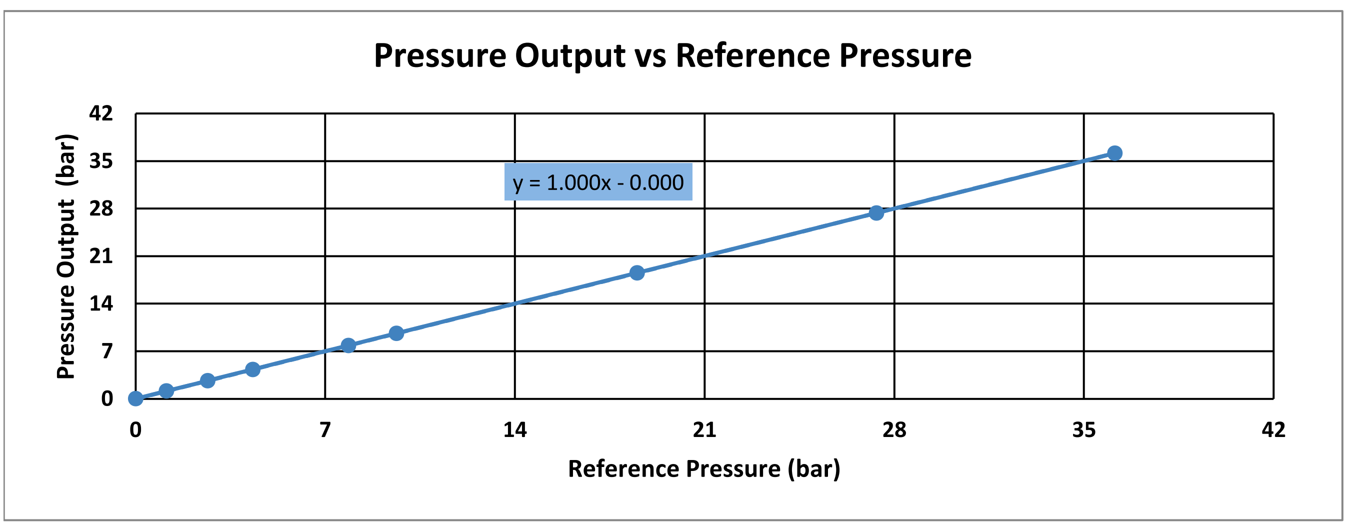
Calibrated with Adara calibration procedure EC_CPTCAL-2.3

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	0.01%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.05%	PASS

Sleeve Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.11%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.25%	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	0.05%	PASS

X-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

Y-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.42%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.83%	PASS

Seismic Calibration					
As Found			As Left		
Trigger Delay Error	0.04%	PASS	Trigger Delay Error	0.04%	PASS

Temperature Calibration			
Full Scale Error	0.13%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	2.1	24.0	42.7	˚C
Tip	-1.210	0.374	2.383	bar
Sleeve	0.075	-0.018	-0.028	bar
Pressure	1.090	1.042	0.996	bar
Temperature	2.075	23.949	42.811	˚C

Tip Temperature Coefficient	0.088	bar/˚C	PASS
Sleeve Temperature Coefficient	-0.003	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	101141	2025-07-03
Sleeve Load Cell	Precision	P-11313	101214	2025-10-21
Digital Loadcell Indicator	9840-200-1-T	88364	101098	2025-06-07
Fluke Reference Pressure Monitor	RPM4 A10Ms	3061	101258	2025-12-11
Tektronix Function Generator	AFG1022	1820013	101200	2025-09-30
Thermometer	THS-222-555	D23255834	101116	2025-06-27
Thermometer	THS-222-555	D23255829	101116	2025-06-27
Thermometer	THS-222-555	D20345575	101116	2025-06-27

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Adara Error Definitions

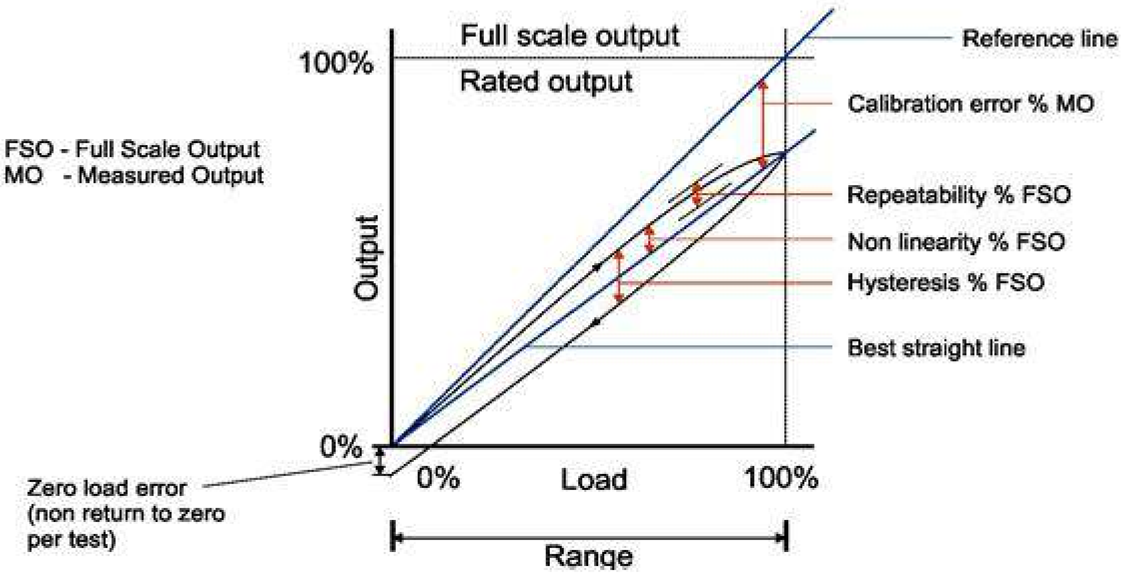

Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

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

Temperature Check Passing Criteria

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

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ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

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

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

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

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

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

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

References

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

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

ASTM D7400-19. "Standard Test Methods for Downhole Seismic Testing". ASTM, West Conshohocken, PA, USA.