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REPORT ON

MERCER ISLAND MIXED USE DESIGN
2885 78TH AVE SE
MERCER ISLAND, WASHINGTON

by
Haley & Aldrich, Inc.
Seattle, Washington

for
Xing Hua Group, Ltd.
Bellevue, Washington

File No. 0202744-000
June 2023

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1. Introduction

This report presents our geotechnical engineering design study a mixed-use development project in Mercer Island, Washington (Figure 1). It includes our geotechnical engineering design recommendations and is organized as follows:

- Introduction;
- Project Understanding;
- Purpose, Scope, and Use of This Study;
- Subsurface Conditions;
- Seismic Considerations;
- Geotechnical Engineering Design Recommendations; and
- Recommendations for Continuing Geotechnical Services.

Tables are presented in the text and figures follow the text to illustrate the project area, exploration locations, and geotechnical design recommendations. Appendix A presents field exploration logs. Appendix B presents the laboratory test methods and results for the current study. Appendix C presents historical exploration logs in the project vicinity completed by Haley & Aldrich, Inc. (Haley & Aldrich) and others.

The recommendations presented herein are based on the building design as of the date of this report. If the project/building plans change, these recommendations should be confirmed and/or revised.

2. Project Understanding

The project consists of a four-story, mixed-use building with one level of below-grade parking. The proposed development site is shown on Figures 1 and 2.

We understand that the grading plan is for the underground parking level to have a basement finish floor at approximately an elevation of 77.5 feet. The existing ground surface generally slopes from an elevation of about 90 feet along 78th Avenue SE to about 82 feet along 77th Avenue SE. The bottom of the excavation is expected to be approximately 8 to 15 feet below existing ground surface.

In this report, the elevation datum is North American Vertical Datum 1988 and the horizontal datum is North American Datum 1983/1991.

3. Purpose, Scope, and Use of This Study

The purpose of our work was to assess subsurface information and provide geotechnical engineering recommendations for design of the proposed structure. Our scope of work included:

- Collecting and assessing subsurface conditions from historical explorations;
- Drilling four borings and installing two monitoring wells;
- Conducting six dynamic cone penetration test;
- Preparing logs of the explorations;
- Assessing groundwater conditions including slug testing of new and existing wells;
- Conducting engineering analysis; and
- Preparing this report summarizing our findings and presenting geotechnical recommendations.

We completed this work in general accordance with our contracts and change orders, and recent discussions with the design team on the revised design of the development. Two other reports have been prepared for this site, the most recent being 3 November 2020. This current report provides updated design recommendations based on the current project plans and can be considered to be a stand-alone document.

This report is for the exclusive use of Xing Hua Group, Ltd., and its design consultants for specific application to this project and site. We completed this work in accordance with generally accepted geotechnical engineering practices for the nature and conditions of the work completed in the same or similar locations, at the time the work was performed. We make no other warranty, express or implied.

4. Subsurface Conditions

4.1 SITE CONDITIONS

We visited the site in 2013 to observe the condition of the on-site buildings, nearby buildings, and paved surfaces. The buildings did not show signs of excessive building settlement such as large cracks in the walls or sloping lines. We did observe concrete cracking on the exterior stairway on the north side of the 2885 78th Avenue SE building that houses the Seven Star restaurant and a slight separation of concrete masonry unit (CMU) joints on the southwest corner of the 2864 77th Avenue SE building that houses Terra Bella; however, these observed conditions are not definitively caused by foundation settlement. We have not done a similar walk around the site since then.

According to property records accessed on the City of Mercer Island website, it appears that most of the buildings on or near the site are founded on spread foundations. However, the McDonald's restaurant immediately north of the site and the building immediately north of the McDonald's (2737 78th Avenue SE) were both supported using timber pile foundations up to 25 feet long.

4.2 FILED EXPLORATIONS

Exploration locations by Haley & Aldrich for the current project are shown on Figure 2 and exploration logs are provided in Appendix A. We also observed push probes conducted by Farallon Consulting for environmental sampling and made our own exploration logs for those explorations. We also reviewed geotechnical reports by Terra Associates, Inc. (Terra, 2012) and ABPB Consulting (ABPB, 2012). The locations of historical explorations and Farallon's push probes are also shown on Figure 2 and the logs are provided in Appendix B.

On 12 to 13 November 2014, we performed a subsurface investigation including four hollow-stem auger borings, HC-1 to HC-4, from 36.5 to 41.5 feet below ground surface (bgs) and one dynamic cone penetrometer, HC-5, to 20.5 feet bgs. We installed monitoring wells in borings HC-1 and HC-2. On 14 November 2014, we developed the monitoring wells and on 17 November 2014, we performed slug testing on monitoring wells in borings HC-1, HC-2, APBP M3, and Terra B-1. On 18 April 2022, we performed six Cone Penetration tests (CPT) CPT-01 to CPT-06 from 10 to 40 feet bgs.

Our understanding of the subsurface conditions is based on current and historical explorations at the site. Subsurface conditions interpreted from explorations at discrete locations on the site and soil properties inferred from the field and laboratory tests formed the basis of the geotechnical recommendations in this report. The nature and extent of variations between explorations may not become evident until additional explorations are performed or construction begins. If variations are encountered, it may be necessary to reevaluate the recommendations made in this report. General soil and groundwater conditions are addressed below. Refer to exploration logs for more detailed information at specific locations.

4.3 SOIL CONDITIONS

The subsurface soil conditions are illustrated by generalized subsurface profiles AA' through DD' on Figures 3 through 6. Based on our interpretation of the borings, the regional topography, and our conversations with the current property owners, the site is likely a filled in swamp/marsh lowland area underlain by relatively impermeable glacial silt and clay. On the east side of the property the dense soils are less than 10 feet deep. On the west the dense soils are deeper. This affects foundation support recommendations primarily, and shoring elements secondarily.

As shown on the subsurface profiles, we have divided the lithology into four main soil units:

Unit 1. Loose to medium dense silty granular FILL, soft SILT, and PEAT. This unit is generally not suitable for conventional spread footings.

Unit 2. Medium stiff to hard SILT and silty CLAY. This unit is generally suitable for conventional spread footings with moderate bearing pressures but may require localized overexcavation and replacement with structural fill to provide adequate foundation subgrade.

Unit 3. Medium dense to dense SAND and silty SAND. This unit may be interbedded with Unit 2 and Unit 4 and is expected to be most prominent and most likely to be encountered along the southern end of the site. Excavations into this unit will likely require dewatering.

Unit 4. Hard SILT. This unit generally underlies the other soil units except along the southern end of the site. This unit is suitable for conventional spread footings with moderate to high bearing pressures.

In this report we define “competent soils” as Soil Units 2, 3, and 4.

4.4 GROUNDWATER CONDITIONS

Groundwater was observed during drilling at the site at depths of 7.5 to 35 feet. Groundwater occurs in the predominantly fine-grain soils (Units 1, 2, and 4) as perched water within discontinuous permeable lenses. Saturated groundwater conditions were observed in Unit 3. We have variously noted or measured groundwater between elevations 75 feet and 79 feet. Based on our measurements and observations, the water level varies seasonally. For design purposes, we recommend a groundwater table elevation of 79 feet for the design of below grade structures and for groundwater management planning. But depending on the time of year and the location around the site the water level might actually be at about elevation 75 feet.

Except on the far east side, this puts the groundwater in the upper poor soil.

5. Seismic Considerations

5.1 SEISMIC SETTING

The seismicity of Western Washington is dominated by the Cascadia Subduction Zone (CSZ), where the offshore Juan de Fuca plate subducts beneath the continental North American plate. Three main types of earthquakes are typically associated with subduction zone environments: crustal, intraplate, and interplate earthquakes. Seismic records in the Puget Sound area clearly indicate a distinct shallow zone of crustal seismicity, the Seattle Fault, which may have surficial expressions and can extend to depths of 25 to 30 kilometers (km). A deeper zone is associated with the subducting Juan de Fuca plate and produces intraplate earthquakes at depths of 40 to 70 km beneath the Puget Sound region (e.g., the 1949, 1965, and 2001 earthquakes) and interplate earthquakes at shallow depths near the Washington coast (e.g., the 1700 earthquake with an approximate magnitude of 9.0).

5.2 CODE-BASED SEISMIC DESIGN PARAMETERS

The basis for seismic design for the 2015 International Building Code (IBC) is the risk-targeted maximum considered earthquake (MCE_R) for ground motion response accelerations, and the maximum considered earthquake geometric mean (MCE_G) hazard for the peak ground acceleration (PGA).

The MCE_R ground motion response accelerations are defined for the most severe earthquake considered by IBC 2015, determined for the orientation that results in the largest maximum response to horizontal ground motions, and adjusted for the targeted risk. The geometric mean PGA corresponding to MCE_G is defined for the most severe earthquake, without adjustment for the targeted risk. The most severe earthquake considered by the code has a 2 percent probability of exceedance in 50 years, corresponding to a 2,475-year return period.

The mapped response spectra are based on Site Class B (rock) conditions. Seismic parameters are adjusted according to the actual site conditions. Based on the average soil stiffness in the upper 100 feet of soil, the recommended site class for this project location is Site Class D (stiff soil). IBC 2015 defines the design spectral acceleration parameters at short periods (S_{DS}) and at the one-second period (S_{1D}) as two-thirds of the corresponding site-class-adjusted MCE_R parameters (SMS and SM1). Similarly, American Society of Civil Engineers (ASCE) 7 requires MCE_G peak ground acceleration adjusted for site effects (PGA_M) to be used for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. The seismic design parameters were obtained from the U.S. Geologic Survey (USGS) U.S. Seismic Design Maps web application (https://earthquake.usgs.gov/ws/designmaps/) and the ASCE 7 Hazard Tool web application (https://asce7hazardtool.online/) accessed on 28 October 2020. The resulting seismic design parameters are shown in Table 1.

Table 1 – Seismic Design Parameters (ASCE/SEI 7-10)	
Parameter	**Value**
Latitude	47.58473
Longitude	–122.234008
Site class	D
Risk category	I, II, or III
Peak ground acceleration, PGA	0.568 g
Spectral response acceleration at short periods, S_S	1.38
Spectral response acceleration at the 1-second period, S_1	0.531
Seismic site coefficient, F_{PGA}	1
Site modified peak ground acceleration, PGA_m	0.568 g
Seismic site coefficient, F_a	1
Seismic site coefficient, F_v	1.5
Sds (Ss * Fa)	0.92
Sd1 (S1 * Fv)	0.531

Note:

S_S and S_1 values presented in Table 1 are for the Site Class B/C boundary and should be adjusted to be applicable to Site Class D conditions at the project site using the site coefficients included in this table (i.e., F_a and F_v).

5.3 SEISMICALLY INDUCED GEOTECHNICAL HAZARDS

Surface Rupture. The northernmost splay of the Seattle Fault exists approximately 0.5 miles south of the site. There is a remote potential for surface rupture at the site from a new splay of the Seattle Fault; however, this hazard is very low based on the Seattle Fault's 3,000-year recurrence interval, the large number of possible locations for surface rupture, and the chance that the fault would not produce surface rupture in this segment of the fault.

Lateral Spreading. Lateral spreading is typically associated with lateral movement on sloping ground caused by liquefaction or a reduction of shear strength of soils within or under the slope. Given the low liquefaction hazard at the site, we judge that the potential for lateral spreading is also low.

Landslides. We reviewed the City's Environmentally Critical Area (ECA) Ordinance and found that no critical area issues, such as previous landslide or steep slope, currently exist at the site. The risk of landslide during an earthquake is considered low for this site.

Liquefaction and Subsidence. Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in sudden loss of shear strength in the soil. Granular soils that rely on inter-particle friction for shear strength are susceptible to liquefaction under the excess pore pressure buildup during strong ground shaking. Liquefaction can cause ground settlement, bearing capacity failure, and lateral spreading.

Liquefaction susceptibility on the site was evaluated from standard penetration test (SPT) results using the Idriss and Boulanger (2008) method. The evaluation identified liquefiable soils for four of the 95 SPT samples analyzed. Anticipated post seismic settlement may occur on the order of 1 to 2.5 inches in these discrete areas. The liquefiable samples are located 20 to 35 feet below the existing ground surface. The CPTs results on liquefaction are consistent with analyzed SPT samples. We anticipate that the proposed foundation system will be able to tolerate this discrete settlement and not adversely affect the functionality of the building. As a precaution, if loose soils are observed beneath the footings during construction, they should be over excavated and replaced with well-compacted materials, such as Washington State Department of Transportation (WSDOT) Standard Specification (SS) Section 9-03.9(3) crushed surfacing base course or better.

6. Geotechnical Engineering Design Recommendations

This section of the report presents our geotechnical engineering analysis, conclusions, and design recommendations for the project. Our recommendations are based on our current understanding of the project and the subsurface conditions revealed by relatively recent and historical borings. As noted, if the nature or location of the proposed project facilities changes, Haley & Aldrich should be notified so that we can change or confirm our recommendations.

6.1 EXCAVATION AND SHORING

We recommend a conventional shoring system of soldier piles, tieback anchors, and wood lagging, even though the excavation will extend about to the groundwater level.

Our shoring recommendations assume that the excavation will extend down to only about elevation 75 feet. There are project elements that will require deeper excavation, such as elevator pits and possible foundations for the tower crane. In those cases we assume that open cuts can be used, or if the conditions do not allow open cuts, then temporary and reusable shoring (like steel plates).

Perched groundwater will likely be encountered in sand zones throughout the excavation depth. Excavations below elevation 79 feet will encounter increasing amounts of groundwater, but we expect the flows to still be manageable and not to require wide-spread dewatering. If the conditions encountered are not as expected we will need to relook at the groundwater management requirements, working with the project dewatering designer and consultant.

Shoring should be designed by a professional structural engineer registered in the State of Washington. We also recommend that we be given the opportunity to review the geotechnical aspects of the shoring design before construction. It is generally not the purpose of this report to provide specific criteria for the contractor’s construction means and methods. It should be the responsibility of the shoring contractor to verify actual ground conditions and determine the construction methods and procedures needed to install an appropriate shoring system.

6.2 LATERAL SOIL PRESSURES FOR DESIGN OF TEMPORARY SHORING WALLS

Lateral earth pressures for the shoring design depend on the type of shoring and its ability to deform. If the top of the shoring is allowed to deform on the order of 0.001 to 0.002 times the shoring height, and if no settlement-sensitive structures or utilities are within the zone of deformation, the shoring may be designed using active earth pressures. If settlement-sensitive structures or utilities exist within the potential zone of deformation, or where the shoring system is too stiff to allow sufficient lateral movement to develop an active condition, at-rest earth pressures should be used to design the shoring.

We expect that temporary shoring will consist of soldier piles and timber lagging in either a cantilevered condition or with one level of tieback anchors. Tied-back or braced walls should be designed using a triangular earth pressure distribution, subject to additional discussions between the shoring designer, the structural engineer, and Haley & Aldrich. General earth pressure diagrams and recommendations for temporary shoring are provided on Figure 7.

The lateral earth pressures presented herein for soldier piles are based on non-sloping conditions behind the walls and drained conditions so that hydrostatic water pressure does not act on the walls above the base of the excavation. For design calculations, we recommend adding at least 2 feet to the proposed excavation depth to allow for possible surface pressures near the excavation (e.g., light vehicles, small material stockpiles).

Based on the assumed loading conditions and the applied loads, we expect the shoring system to deflect about 1 inch or less into the excavation. Individual soldier piles may deflect more than 1 inch or deflect away from the excavation.

Haley & Aldrich should review any soldier piles that deflect more than 1/2 inch to try to identify the cause of the deflection and to determine whether remedial measures are required.

6.2.1 Surcharge Pressures on Shoring

Additional lateral pressures due to surcharge loads (e.g., buildings, footings, heavy equipment, large material stockpiles) should be calculated using methods shown on Figure 8. These loads would be added to the loads calculated for the shoring walls. We recommend Haley & Aldrich review or complete the estimated surcharge loads when surcharge loads, footprints, and foundation plans of adjacent structures are available.

6.3 SOLDIER PILE DESIGN

We recommend the following for soldier pile design:

- Soldier piles must be designed by a licensed structural engineer;
- Soldier piles should be designed for bending using a uniform loading equivalent to 80 percent of the design values and analyzed for shear using total load;
- To design against kickout, the lateral resistance should be computed using the passive pressure on Figure 7, acting over 2 times the diameter of the concreted shaft section or the pile spacing, whichever is less;
- The embedded portion of the pile shaft should be at least 2 feet in diameter; and
- Piles should be embedded at least 8 feet below the bottom of the excavation and extend below Soil Unit 1.

These recommendations assume proper installation of the soldier piles as discussed later in this report.

We recommend the allowable axial pile capacity parameters in Table 2 to calculate the vertical resistance of the soldier piles. The values assume that soldier piles are embedded into competent soils. The pile side friction above the bottom of the excavation should be neglected. The soldier piles should be embedded at least 8 feet below the base of the excavation.

Table 2. Axial Capacity Parameters for Drilled Soldier Piles

Soil Unit	Allowable Unit Side Capacity (ksf)	Allowable Unit End Capacity (ksf)
Unit 1	0.5 ksf	NA
Units 2 – 4	2 ksf	10 ksf

Notes:
ksf = kips per square foot

6.4 LAGGING DESIGN

Temporary lagging should be designed in accordance with Federal Highway Administration (FHWA) Geotechnical Engineering Circular 4 (FHWA, 1999), structural engineering guidelines, soil type, and local experience. Table 3 provides recommended lagging thicknesses based on the FHWA recommendations.

Based on our site investigation, we recommend using a Soil Type of "Competent" for the eastern half of the site and "Difficult" for the western half of the site.

Table 3. Recommended Temporary Lagging Thickness

		Clear Span of Lagging (feet)					
		5	6	7	8	9	10
		Minimum Actual Thickness of Rough-Cut Timber Lagging (inches)					
Competent (Type 1) [a]	25 and under	2	3	3	3	4	4
	Over 25 to 60	3	3	3	4	4	5
Difficult (Type 2) [a]	25 and under	3	3	3	4	4	5
	Over 25 to 60	3	3	4	4	5	5
Potentially Dangerous (Type 3) [a]	15 and under	3	3	4	5	See note[b]	See note[b]
	Over 15 to 25	3	4	5	6	See note[b]	See note[b]
	Over 25	4	5	6	See note[b]	See Note[b]	See note[b]

Notes:

a. *Soil type as defined in WSDOT SS Section 6-16.3(6)A.*

b. *For exposed wall heights exceeding the limits in Table 3, or where minimum rough-cut lagging thickness is not provided, the contractor should design the lagging in accordance with structural engineering guidelines and local experience. Soldier pile and lagging shoring may not be appropriate for these cases.*

6.5 TIEBACK DESIGN

We recommend the tentative allowable tieback pullout value in Table 4 for a typical 6-inch-diameter drilled hole with a pressure-grouted bond zone. The allowable transfer load includes a recommended factor of safety of 2.0. The factor of safety should be confirmed by completing at least two successful verification tests in each soil type. Additionally, each tieback should be proof-tested to 133 percent of the design load. We recommend that the shoring contractor and/or designer determine a final design tieback pullout resistance based on their previous experience in Mercer Island or Seattle, which must then be confirmed by field testing.

Table 4. Tentative Pullout Resistance for Tiebacks with Pressure-Grouted Bond Zone

Soil Type	Allowable Transfer Load (kip/ft)
Competent soils – Soil Units 2 through 4	2

We make the following additional recommendations for tieback design:

- Do not install the bond zone within Soil Unit 1 (fill, soft silt and clay, peat).
- Tieback bond zones should be outside of the no-load zone. The no-load zone is shown on Figure7 as a zone bounded by a 60-degree line to the horizontal that starts at a distance of H/4 from the bottom of the excavation, where H is the excavation height.
- Locate anchors at least three tieback diameters apart.
- Design anchor lengths so that they do not conflict with any underground support elements of adjacent structures.
- Identify existing facilities adjacent to the project site including buried utilities and foundations, as these may affect the location and length of the anchors.
- Allow the contractor to select the tieback anchor material and the installation technique. The shoring contractor should be contractually responsible for the design of the tieback anchors, as tieback capacity is largely a function of the means and methods of installation. The selected installation method must be confirmed using verification and proof-testing.
- Haley & Aldrich should review the design for anchor locations, capacities, and related criteria prior to implementation.

6.6 PERMANENT SUBGRADE WALL DESIGN

This section and Figures 8 and 9 provide guidance for determining the permanent subgrade wall loads.

6.6.1 Earth Pressures

Permanent subsurface walls constructed adjacent to soldier pile shoring may be designed using the same earth pressure values and distribution that was used for shoring design. The earth pressure does not include surcharge loads such as loads from adjacent buildings; these must be calculated separately and added to get the total permanent lateral pressure.

Permanent walls that are backfilled and are not adjacent to shoring walls should be designed using a triangular earth pressure distribution. For typical granular fill soil, active and at-rest pressures may be determined using the equivalent fluid unit weights in Table 5. Note that the equivalent fluid density does not include any surface loading conditions or loading due to groundwater hydrostatic pressure; also, the ground surface behind the wall is assumed to be horizontal. Walls without drainage must be designed for full hydrostatic pressure.

The use of active and passive pressure is appropriate if the wall is allowed to yield a minimum of 0.001 times the wall height. For a non-yielding wall, at-rest pressures should be used.

Table 5. Soil Equivalent Fluid Unit Weights for Walls Backfilled with Structural Fill		
Soil Type	**Parameter**	**Value (pcf)**
Structural fill	Active earth pressure	35
	At-rest earth pressure	55
	Passive earth pressure [a]	300
Notes: a. *Include a factor of safety of 1.5*		

6.6.2 Hydrostatic Groundwater Pressure

Subgrade walls and slabs will be waterproofed and designed for hydrostatic lateral and uplift pressures. There will be no wall or underslab drainage installed.

For walls and floors that are not drained, a triangular lateral hydrostatic pressure of $62.4h_w$ per square foot should be added, where h_w is the depth of structure below the design groundwater level. The depth of the basement is expected to be very close to the level of the groundwater table. For undrained walls and slabs we recommend a design water level of 79 feet.

6.6.3 Seismic Earth Pressure on Walls

Lateral earth pressures based on the design earthquake for active and at-rest conditions can be assumed as uniform pressures in pounds per square foot of 8H and 12H (where H is the height of the wall in feet), respectively. The seismic earth pressure should be applied from the top of the wall to the bottom of the excavation, as shown on Figure 9. This seismic earth pressure is calculated using the 2015 IBC design hazard level for the site.

6.6.4 Surcharge Pressures on Walls

The pressures shown on Figures 7 and 9 do not include surcharge loads due to buildings, footings, heavy equipment, large stockpiles, and so forth. These loads must be calculated separately, using the methods shown on Figure 8 or similar, and added to the pressures determined using Figures 7 and 9.

We recommend Haley & Aldrich review or complete the estimated surcharge loads when surcharge loads, footprints, and foundation plans of adjacent structures are available.

6.7 FOUNDATION DESIGN RECOMMENDATIONS

Figure 10 provides a contour map of the estimated elevation of the top of competent soils; however, it is important to note that the contours on Figure 10 are only an estimate based on interpolation between the exploration locations. With the lowest finished floor at elevation 77.5 feet, we estimate that spread footing and a mat slab could be founded at about elevation 74 feet to 75 feet. Based on the Figure 10 contours that means approximately the eastern 40 percent of the site will expose competent soils at elevation 75 feet, allowing direct support of spread foundations. For the western 60 percent of the site some improvement of the ground will be needed to support spread foundations. The maximum allowable foundation bearing pressure for either an isolated footing or a mat slab is 5 ksf. If a lower allowable bearing pressure will work for the building that will reduce the post-construction settlement and/or allow a broader spacing on the ground improvement (if used).

Overexcavation. If the competent soils are within 3 to 5 feet of the bottom of the spread foundations it could be most economical to overexcavate the poor material and either found the excavations deeper or backfill the overexcavation with compacted structural fill or lean mix concrete. In this condition, however, excavation extending below elevation 75 feet will encounter increasing amounts of groundwater, thus complicating the entire operation. For that reason we recommend only nominal overexcavation and replacement, and generally in isolated or confined areas that can be drained.

Ground Improvement. Rammed aggregate piers, commonly known by the company trade name as Geopiers, or another method call rigid inclusions, are both common ways to improve poor soil so that spread foundations can be used. Geopiers are designed and installed by a specialty contractor based on criteria provided by the geotechnical engineer. Rigid inclusions are designed by the geotechnical engineer and are installed by contractors capable of installing soldier piles or augercast piles, etc. Both methods take the spread foundation load and transfer it to denser soils below the bearing elevation.

Augercast Piles. If spread foundations are not desired on the western portion of the building, then augercast piles can be used as a deep foundation alternative. In our opinion use of augercast piles will create a potential differential settlement concern and so will require additional modeling and design to predict and mitigate that settlement.

If used, we recommend Geopiers and rigid inclusions be designed for an allowable bearing pressure of 5 ksf with a post-construction settlement of no more than 1 to 1/2 inches. We also recommend the installations be a minimum of 20 feet long and extend at least 10 feet into the dense glacial soil. Depending on the design we may also ask for load testing and confirmation testing using CPTs. The tops of the installations are covered with a load transfer platform consisting of 12 to 24 inches of gravel or crushed rock.

Because of the likely ground conditions at the bearing elevation, it will probably be necessary to install the Geopiers or rigid inclusions from the ground surface or just below the ground surface. Geopiers are installed using a heavy tracked rig supported by a large front loader. A stable working surface for this equipment is needed.

These ground improvement elements should also be installed before the shoring walls.

6.8 GROUNDWATER MANAGEMENT

6.8.1 Slug Results

Water levels and slug testing results are presented in Appendix C and may be used for design of construction dewatering and estimating water flow into a permanent drainage system. Based on the slug test results we recommend average hydraulic conductivities for wells screened in Soil Unit 3, sand and silty sand, 9.0×10^{-5} to 8.3×10^{-4} centimeters per second (0.3 to 2.4 feet per day).

6.8.2 Temporary Construction Dewatering

We set the design groundwater level at elevation 75 feet to better estimate settlements associated with dewatering and the resulting water level drawdown. But since extensive dewatering will not be required, we believe elevation 79 feet is a better planning level for encountering and dealing with groundwater in the excavations. We expect lower levels to occur in the drier summer and early fall months. We think it is important to keep the dewatering designer engaged as part of the team through construction even if a major dewatering program is not expected. We will work with the designer and with the contractors to determine the best methods of controlling groundwater during excavation and construction of the foundations and below-grade elements.

Subject to the dewatering designer's concurrence we believe only nominal and isolated drainage and dewatering will be required for general excavation. A network of ditches and sumps, supplemented by well points where needed, should provide the necessary drainage to allow excavations to be completed and free movement of excavating and other construction equipment to occur.

Note however that the subgrade will be soft and will get softer if it rains. It will probably be necessary some or all of the time in the bottom of the hole to build a working surface of quarry spalls or crushed rock, perhaps with a geotextile. We can work with the contractor on appropriate materials to create a stable working surface.

The amount of water discharged from the site depends on many factors including design and operation of the dewatering system (if applicable), the excavation depth and extent, and the variability in soil and groundwater properties. Rainfall, surface water, and groundwater from adjacent utility trenches can significantly increase short-term water discharge rates. Also, the time of year and nearby construction dewatering activities can affect groundwater flows.

6.8.3 Permanent Drainage

Because the below-grade walls and slabs will be waterproofed there will be no subsurface drainage system required. All below-grade elements located below elevation 79 feet must be designed for the lateral and uplift hydrostatic pressures from the groundwater.

6.8.3.1 Backfilled Walls

Walls with soil backfilled on only one side will require drainage or they must be designed for full hydrostatic pressure. We recommend the following:

- Backfilling should be done with a minimum thickness of 18 inches of free-draining sand or sand and gravel that is well-graded (i.e., that has a wide range in particle size).
- Drains should be installed behind any backfilled subgrade walls. The drains, with cleanouts, should consist of perforated pipe a minimum of 4 inches in diameter placed on a bed of, and surrounded by, at least 6 inches of free-draining sand or sand and gravel. The drains should be sloped to carry the water to a sump or other suitable discharge.
- The backfill should be continuous and should envelop the drainage behind the wall.
- The drainage fill surrounding the pipe should be compatible with the size of the holes in the pipe.

6.8.3.2 Final Site Drainage

- The site and adjacent paved areas should be graded in such a way that surface water will not pond near the structures.
- Roof drains should be sloped and tightlined to a suitable outlet away from the proposed building.

6.8.3.3 Stormwater Detention

The required stormwater detention will be provided in 6-foot diameter pipes outside a portion of the northern wall. The pipe inverts will be at about elevation 78 feet or 79 feet and should be above the groundwater table. Excavation for the pipe installation will be about 10 feet or so below grade. We will work with the contractor as needed on the best installation plan.

7. Geotechnical Recommendations for Construction

7.1 SOLDIER PILE INSTALLATION

- Installation methods should minimize caving soils or loosening of soil at the bottom of the drilled shaft which can reduce the bearing capacity in the zone of disturbed soil. Groundwater increases the chances of soil disturbance.
- Tieback de-tensioning and shoring failure could occur if bearing capacity is inadequate and soldier piles settle under the vertical component of the inclined tieback load. We recommend that a Haley & Aldrich representative closely monitor soldier pile installation for these conditions so construction methods can be adjusted accordingly.
- The contractor should be prepared to case the soldier pile holes where loose soils or groundwater seepage could cause loss of ground. Fill soils can be especially prone to caving and may require casing. The actual need for casing should be determined in the field at the time of installation.
- If the shaft excavation contains water or slurry, the contractor should tremie concrete to the bottom of the hole. Lean mix, concrete, and controlled density fill should not be end-dumped through water or slurry.
- The contractor should be prepared to excavate the soldier piles in a manner that prevents heave or boiling at the bottom of the soldier pile excavation. It may be possible to over-drill the borehole and backfill the bottom of the borehole with structural concrete bearing on undisturbed soil.
- Drilling mud should not be used unless reviewed and approved by Haley & Aldrich and the shoring designer.
- Soldier pile shoring construction may be difficult if cobbles or loose sand and gravel are encountered in the excavation. If these conditions are encountered, substantial soil raveling could occur. If raveling soils are encountered, we recommend shaft construction methods such as slurry or temporary casing be used to minimize raveling and loss of soil.

7.2 LAGGING INSTALLATION

- Prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is important to maintain the integrity of the excavation. The contractor should be prepared to place lagging in small vertical increments and should also be prepared to backfill voids caused by ground loss behind the shoring system. The proper installation should be the responsibility of the shoring contractor to prevent soil failure or sloughing and loss of ground, and to provide safe working conditions.
- Voids greater than 1 inch should be backfilled with sand, pea gravel, or a porous slurry. The void spaces progressively as the excavation deepens. The backfill must not allow potential hydrostatic pressure buildup behind the wall. Drainage behind the wall must be maintained or hydrostatic water pressure should be added to the recommended lateral earth pressures.
- If there is a slope above the wall, extra lagging should be installed above the shoring wall to provide a partial barrier for material that could ravel down from the slope face and fall into the excavation.

7.3 TIEBACK INSTALLATION

- Structural grout should be pumped into the anchor zone using a grout hose or tremie hose placed at the bottom of the anchor.
- The portion of the tieback in the no-load zone should be filled with a non-cohesive mixture of sand-pozzolan-water or equivalent; or a bond breaker such as plastic sheathing or a polyvinyl chloride pipe should be installed around the tie rods within the no-load zone.
- Tiebacks should be grouted and backfilled immediately after placing the anchor. To prevent collapse of the holes, ground loss, and surface subsidence, anchor holes should not be left open overnight.
- Care should be taken not to mine out large cavities in granular soil.
- Continuous cutting return should be maintained if pneumatic drilling techniques are used, so that air pressure is not channeled to nearby utility vaults, corridors, or subgrade slabs, which may be damaged by air pressure.
- Anchors should be installed to minimize ground loss and previously installed anchors should not be disturbed. During tieback drilling, wet or saturated zones may be encountered and caving or blow-in could occur. Drilling with a casing may reduce the potential for these conditions and ground loss.
- Tiebacks should be tested to confirm the appropriateness of the anchor design values and to verify that a suitable installation is achieved. The recommended procedures for verification and
- proof-testing are provided below.

7.4 RECOMMENDATIONS FOR TIEBACK TESTING

The tieback anchor testing program should include verification testing of select tiebacks and proof testing of all production tiebacks. We recommend that tieback testing be done in general accordance with the recommendations in the publication Recommendations for Prestressed Rock and Soil Anchors by the Post Tensioning Institute (PTI, 2004) and the recommendations below.

7.4.1 Verification Tests

We recommend a minimum of two verification tests per soil type before installation of production anchors to validate the design pullout value. Haley & Aldrich will select the testing locations with input from the shoring subcontractor. Haley & Aldrich or the shoring designer may require additional verification tests when creep susceptibility is suspected or when varying ground conditions are encountered.

Verification tiebacks should be installed by the same methods and personnel, using the same material and equipment, as the production tiebacks; Haley & Aldrich will determine whether deviations require additional verification testing. At least two successful verification tests should be performed for each installation method and each soil type.

Verification tests load the tieback to 200 percent of the deciliter (DL) and include a 60-minute hold time at 150 percent of the DL. The tieback DLs will be on the shoring drawings. The tieback load should not exceed 80 percent of the steel's ultimate tensile strength. Verification test tiebacks should be incrementally loaded and unloaded using the schedule in Table 6.

Table 6– Tieback Verification Test Incremental Load and Hold Time	
Load Level	**Hold Time**
Alignment Load (AL)	Until stable
0.25DL	10 minutes
0.5DL	10 minutes
0.75DL	10 minutes
1.0DL	10 minutes
1.25DL	10 minutes
1.5DL	**60 minutes**
1.75DL	10 minutes
2.0DL	10 minutes

The alignment load should be the minimum load required to align the testing assembly and should be less than 5 percent of the DL. The dial gauge should be zeroed after the alignment load has stabilized. Perform a creep test at 1.5DL by holding the load constant to within 50 per square inch and recording deflections at 1, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes.

The acceptance criteria for a verification test are:

- The creep rate at 1.5DL is less than 0.08 inches between 6 and 60 minutes and the creep rate is linear or decreasing during the creep test;
- The total tieback displacement is greater than 80 percent of the theoretical elastic elongation of the design unbonded length plus the jack length; and
- The anchor does not pull out under repeated loading.

7.4.2 Proof Tests

Proof tests load the tieback to 1.33DL and include a 10-minute hold time at 1.33DL. The tieback DLs should be on the shoring drawings. The tieback load should not exceed 80 percent of the steel's ultimate tensile strength. Proof tests should be incrementally loaded and unloaded using the schedule in Table 7.

Table 7 – Tieback Proof Test Schedule	
Load Level	**Hold Time**
AL	Until stable
0.25DL	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.33DL	10 minutes

The alignment load should be the minimum load required to align the testing assembly and should be less than 5 percent of the design load. The dial gauge should be zeroed after the alignment load has stabilized.

The load should be held constant to within 50 psi and deflections recorded at 1, 2, 3, 5, 6 and 10 minutes. If the tieback deflection between 1 and 10 minutes at 1.33DL exceeds 0.04 inches, the load should be held for an additional 50 minutes and deflections recorded at 20, 30, 50, and 60 minutes.

The acceptance criteria for a proof test are:

- The creep rate at 1.33DL is less than 0.04 inches between 1 and 10 minutes or less than 0.08 inches between 6 and 60 minutes and the creep rate is linear or decreasing during the creep test;
- The total tieback displacement is greater than 80 percent of the theoretical elastic elongation of the design unbonded length plus the jack length; and
- The anchor does not pull out under repeated loading.

7.5 SHORING MONITORING

A shoring monitoring program provides early warning if the shoring does not perform as expected. The monitoring program should include a preconstruction survey, periodic surveys during construction, and a post-construction survey.

7.5.1 Preconstruction Survey

A preconstruction survey documents the condition of existing streets, utilities, and buildings. The survey should include video and/or photographic documentation. The size and location of existing cracks in streets and buildings should receive special attention and may be monitored with a crack gauge.

7.5.2 Construction Survey

We recommend adjacent building surveys and optical surveys be included in the shoring monitoring program during construction. If there are sensitive structures/utility vaults adjacent to the excavation, an inclinometer survey may also be a prudent addition to the monitoring program.

All monitoring data should be submitted to Haley & Aldrich for weekly review. The data will be included in our field transmittals to the project team during construction. Details of our expectations for shoring monitoring are included below.

Adjacent Building Surveys. We recommend that adjacent buildings be surveyed before, during, and after construction. The pre-construction survey will establish the baseline of existing conditions (e.g., identifying the size and locations of any cracks). The surveys should consist of a videotape and/or photographs of the interior and exterior of adjacent buildings and detailed mapping of all cracks. Any existing cracks could be monitored with a crack gauge.

Optical Surveying. We recommend optical surveys of horizontal and vertical movements of: (1) the surface of the adjacent streets, (2) buildings on and adjacent to the site, and (3) the shoring system itself. The contractor, in coordination with the geotechnical engineer, should establish two reference lines adjacent to the excavation at horizontal distances back from the excavation face of about 1/3 H and H, where H is the final excavation height. Typically, these lines will be established near the curb line and across the street from the excavation face. The points on the adjacent buildings can be set either at the base or on the roof of the buildings.

Shoring system monitoring should include measuring vertical and horizontal movement at the top of every other soldier pile, and any geotechnical instrumentation (e.g., inclinometers) used for the project.

The measuring system for the shoring monitoring should have an accuracy of at least 0.01 foot. All reference points on the ground surface should be installed and read before excavation begins. The frequency of readings will depend on the results of previous readings and the rate of construction. At a minimum, readings on the external points should be taken twice a week through construction until below-grade structural elements (floors, decks, columns, etc.) are completed, or as specified by the structural and geotechnical engineers. Readings on the top of soldier piles and the face of existing buildings on or adjacent to the property should be taken at least twice a week during this time. We recommend that an independent surveyor hired by the owner to record the data at least once per week with the other reading taken by the surveyor or contractor.

7.5.3 Post-Construction Survey

A post-construction survey includes reviewing the preconstruction survey and comparing it to post-construction conditions. The survey should include video and/or photographic documentation. Changes in the number, size, and location of cracks in streets and buildings should be given special attention.

7.6 FOUNDATION CONSTRUCTION

Haley & Aldrich should observe exposed subgrades before footing, mat, or slab construction begins to confirm design assumptions about subsurface conditions and subgrade preparation. Exposed subgrade soil that is not firm and unyielding, or that is otherwise considered inadequate by Haley & Aldrich, will need to be over-excavated and replaced with structural fill or CDF or lean mix concrete, depending on the extent and the foundation loading.

Haley & Aldrich should observe any ground improvement placement (overexcavation and replacement and/or aggregate pier installation). Footings or mat slab areas or slab-on-grade areas located over ground improvement must have a load transfer platform 12 to 18 inches thick of gravel or crushed rock.

The exposed subgrade should be carefully prepared and protected before foundation or slab concrete placement. Any loosening of the materials during construction could result in more settlement. It is important that foundation excavations be cleaned of loose or disturbed soil before placing any concrete and that there is no standing water in any foundation excavation. These conditions should be observed by our representative.

Maintain groundwater levels below the base grade of the footing excavation at all times to prevent the risk of heave, piping, boiling, and other loss or disturbance of subgrade material. This groundwater level should be maintained until after the footing steel and concrete are placed.

Any loose or soft soils that occur naturally or are disturbed during construction should be overexcavated and replaced with compacted structural fill or lean mix concrete. Any visible organic and other unsuitable material should be removed from the exposed subgrade.

It may be necessary to place a 2-inch to 4-inch-thick lean or structural concrete mat in footing excavations to protect competent subgrade soil from being softened by water or construction activities after it is exposed. Concrete may only be placed after the geotechnical engineer has checked the subgrade. Lean mix concrete should be in accordance with WSDOT SS Section 6-02.3(2)D. If softer soils are exposed a more substantial working surface of crushed rock, quarry spalls, and geotextile may be needed to provide a stable surface for construction equipment and personnel.

7.7 EARTHWORK

7.7.1 Site Preparation and Grading

We recommend conducting all site grading, paving, and any utility trenching during relatively dry weather conditions.

It may be necessary to relocate or abandon some utilities. Excavation of these utility lines will probably occur through backfill. Abandoned underground utilities should be removed or completely grouted. Ends of remaining abandoned utility lines should be sealed to prevent piping of soil or water into the pipe. Soft or loose backfill should be removed, and excavations should be backfilled with structural fill. Coordination with the utility agency is generally required.

7.7.2 Structural Fill

Backfill placed within the building area or below paved areas should be considered structural fill. We recommend the following for structural fill:

- For imported soil to be used as structural fill, a clean, well-graded sand or sand and gravel with less than 5 percent by weight passing the No. 200 mesh sieve (based on the minus 3/4-inch fraction) should be used. Compaction of soil containing more than approximately 5 percent fines may be difficult if the material is wet or becomes wet during rainy weather.
- All structural fill should be placed and compacted in lifts with a loose thickness no greater than 10 inches. For hand-operated “jumping jack” compactors, loose lifts should not exceed 6 inches. For small vibrating plate/sled compactors, loose lifts should not exceed 3 inches.
- All structural fill should be compacted to at least 95 percent of the modified Proctor maximum dry density (as determined by ASTM D1557 test procedure).
- The moisture content of the fill should be controlled to within 2 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the maximum Proctor dry density.
- In wet subgrade areas, clean material with a gravel content of at least 30 to 35 percent may be necessary. Gravel is material coarser than a US No. 4 sieve.
- Before filling begins, samples of the structural and drainage fill should be provided for laboratory testing. Laboratory testing will include a Proctor test and gradation for structural fill and a gradation for drainage fill. Field testing with a nuclear density gauge uses the maximum dry density determined from a Proctor test so it is important to complete the laboratory testing as soon as possible so backfilling is not delayed.

7.7.3 Use of On-Site Soil as Structural Fill

Our explorations indicated that the near-surface site soil includes silty sand, silt, and clay; we do not recommend using these soils for structural fill.

7.7.4 Temporary Cuts

Because of the variables involved, actual slope grades required for stability in temporary cut areas can only be estimated before construction. We recommend that stability of the temporary slopes used for construction be the sole responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. Excavations should be made in accordance with all local, state, and federal safety requirements.

For planning purposes, the soils across the site are likely Occupational Safety Health Administration Soil Classification Type C; however, the soil classification must be reevaluated at the time of construction.

The stability and safety of open trenches and cut slopes depend on a number of factors, including:

- Type and density of the soil;
- Presence and amount of any seepage;
- Depth of cut;
- Proximity of the cut to any surcharge loads near the top of the cut, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

Considering these factors, we recommend:

- Using plastic sheeting to protect slopes from erosion; and
- Limiting the duration of open excavations as much as possible.

8. Recommendations for Continuing Geotechnical Services

Before construction begins, we recommend that Haley & Aldrich continue to meet with the design team as needed to address geotechnical questions that may arise throughout the remainder of the design and permitting process. We also recommend that Haley & Aldrich review the project plans and specifications to confirm that the geotechnical engineering recommendations have been properly interpreted.

During construction, we recommend that Haley & Aldrich be retained to perform the following tasks:

- Review contractor submittals;
- Observe shoring installation;
- Observe general excavation, over-excavation, all backfill and testing, ground improvement, foundation and slab installations;
- Perform other observations as required by the City of Mercer Island Planning Department and the building permit conditions;
- Attend meetings, as needed; and
- Provide geotechnical engineering support that may arise during construction.

References

1. Federal Highway Administration. 1999. Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems. FHWA-IF-99-015. June 1999

2. Hart Crowser. 2020. Geotechnical Engineering Design Report for Multi-Family Development, Mercer Island, WA. November 2020.

3. International Building Code. 2015. International Building Code. International Code Council.

4. Idriss, I.M. and R.W. Boulanger. 2008. *Soil Liquefaction during Earthquakes* by Earthquake Engineering Research Institute MNO-12

5. Post Tensioning Institute. 2004. Recommendations for Prestressed Rock and Soil Anchors, Third Edition. Post Tensioning Institute.

6. Washington State Department of Transportation. 2020. Standard Specifications for Road, Bridge, and Municipal Construction

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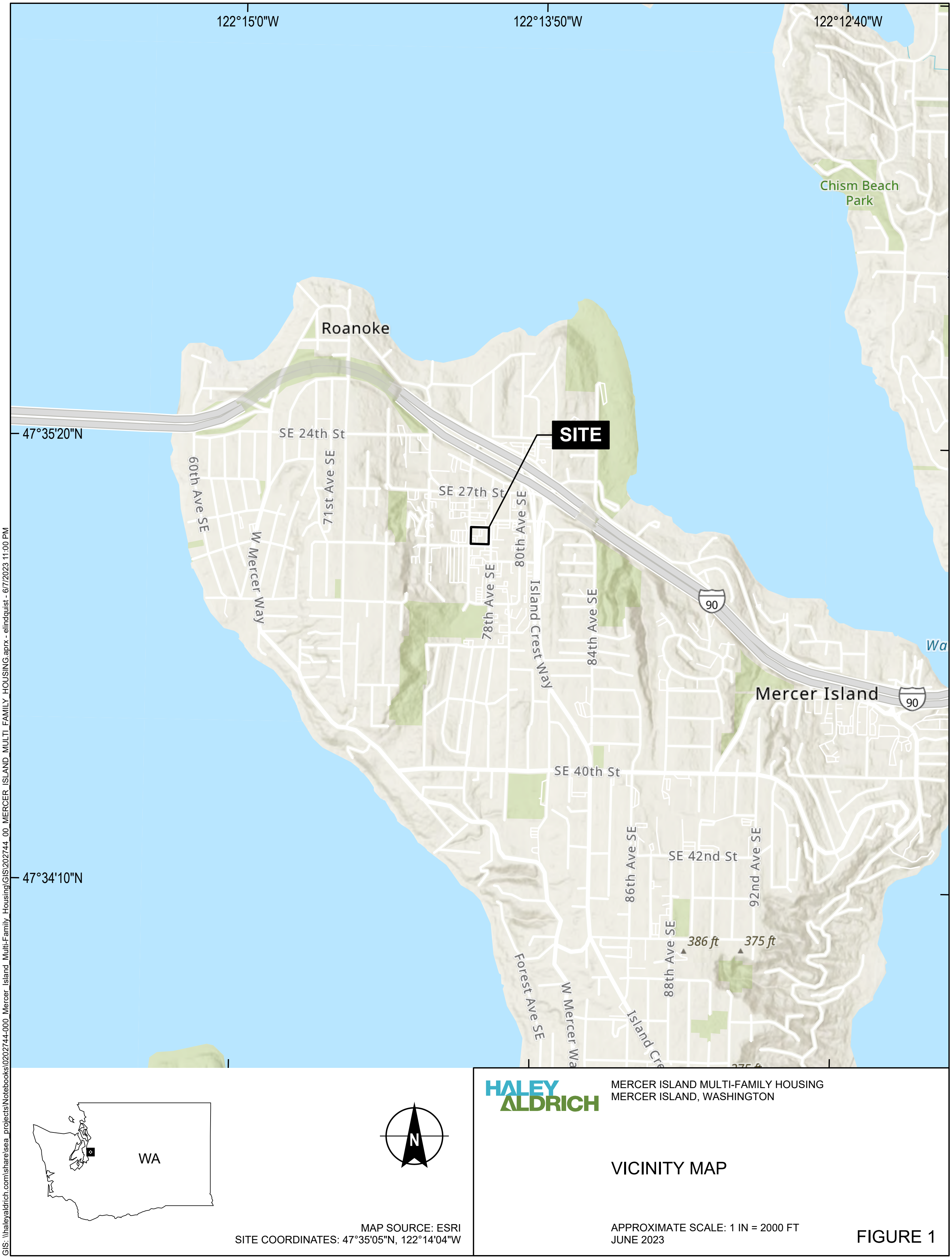
FIGURES



MAP SOURCE: ESRI
SITE COORDINATES: 47°35'05"N, 122°14'04"W

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

VICINITY MAP

APPROXIMATE SCALE: 1 IN = 2000 FT
JUNE 2023

FIGURE 1

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LEGEND

- HC-3 ● BORING (HART CROWSER)
- HC-5 ▲ HAND PROBE (HART CROWSER)
- HC-1 MONITORING WELL (HART CROWSER)
- CPT-01 CONE PENETROMETER TESTING (HART CROWSER)
- B-1 ■ BORING (ABPB CONSULTING)
- B-6 ◑ PUSH PROBE (FARALLON)
- M-1 MONITORING WELL (ABPB CONSULTING)
- B-1 ⊙ BORING (TERRA)
- CROSS SECTION

NOTES

1. FEATURE LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. SURVEY BASE SOURCE: BUSH, ROED & HITCHINGS 14 OCTOBER 2014.

N

0 40 80

SCALE IN FEET

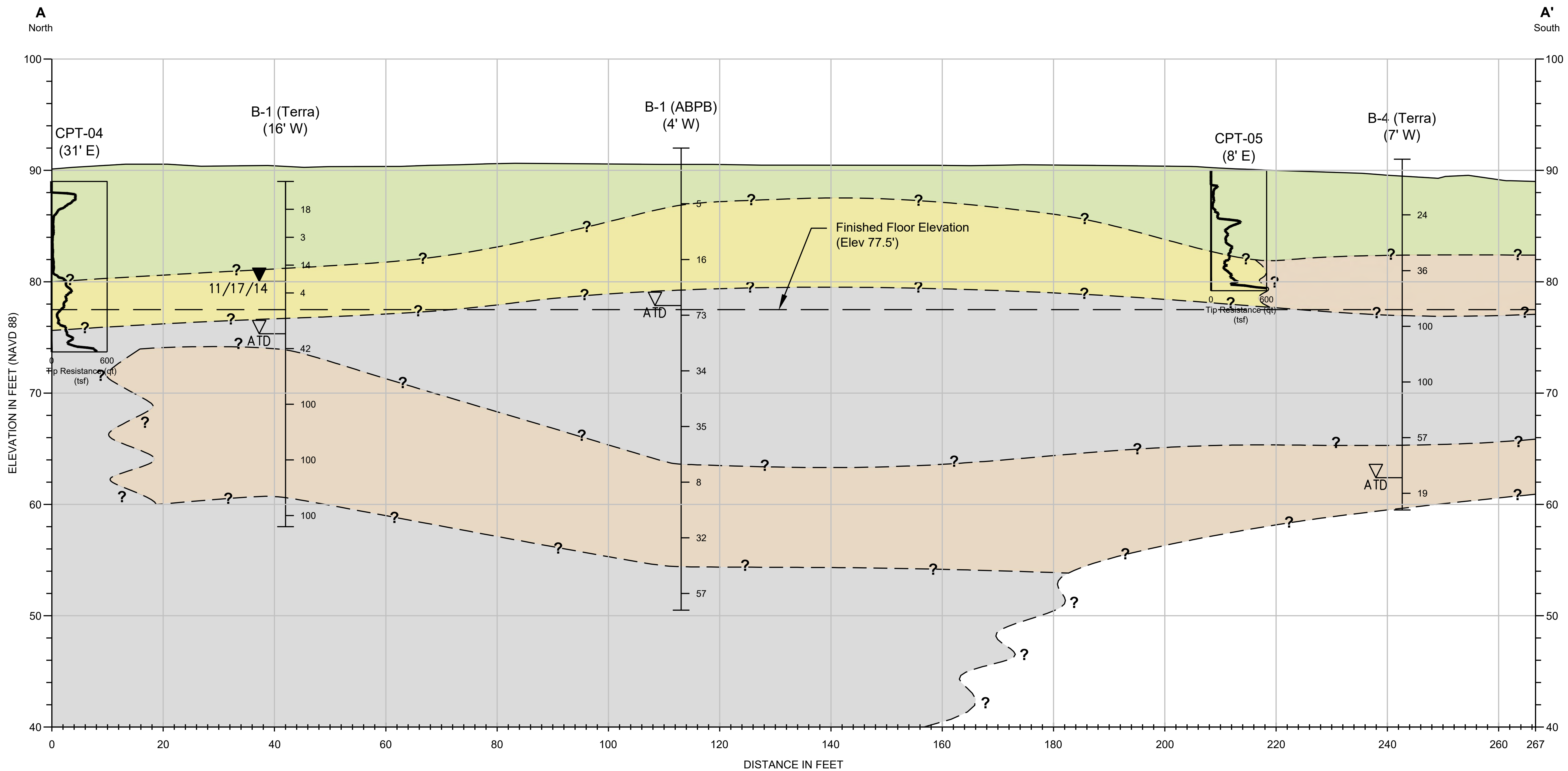
HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

SITE AND EXPLORATION PLAN

JUNE 2023

FIGURE 2



LEGEND

B-1 (Terra)
(16' W) — EXPLORATION NUMBER (OFFSET DISTANCE AND DIRECTION)

EXPLORATION LOCATION

ATD — WATER LEVEL

9 — STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

FINISHED FLOOR ELEVATION JOHNSTON ARCHITECTS, LLC PLANS DATED 10/1/2020

CPT-05
(8' E) — CONE PENETROMETER NUMBER (OFFSET DISTANCE AND DIRECTION)

CONE PENTROMETER LOCATION

0 600
Tip Resistance (qt)
(tsf)

UNIT 1: LOOSE TO MEDIUM DENSE GRANULAR FILL, SOFT SILT, AND PEAT

UNIT 2: MEDIUM STIFF TO HARD SILT AND SILTY CLAY

UNIT 3: MEDIUM DENSE TO DENSE SAND AND SILTY SAND

UNIT 4: HARD SILT

NOTE
THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.

HORIZ. 0 20 40
VERT. 0 10 20
SCALE IN FEET

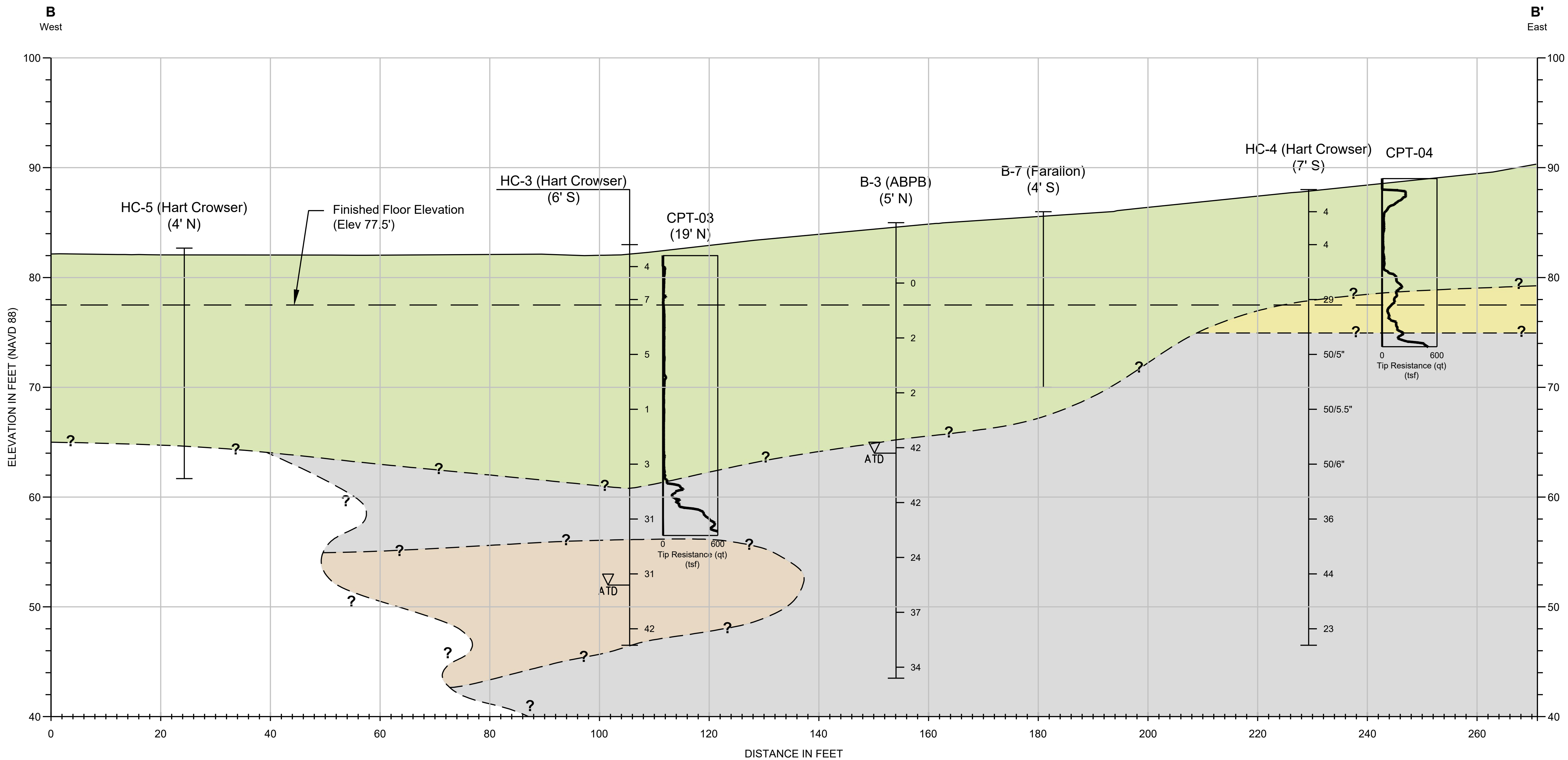
HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

GENERALIZED SUBSURFACE CROSS SECTION A-A'

JUNE 2023

FIGURE 3



LEGEND

B-1 (Terra) (16' W) — EXPLORATION NUMBER (OFFSET DISTANCE AND DIRECTION)

EXPLORATION LOCATION

WATER LEVEL (ATD)

9 — STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

FINISHED FLOOR ELEVATION JOHNSTON ARCHITECTS, LLC PLANS DATED 10/1/2020

CPT-05 (8' E) — CONE PENETROMETER NUMBER (OFFSET DISTANCE AND DIRECTION)

CONE PENTROMETER LOCATION

0 600 Tip Resistance (qt) (tsf)

UNIT 1: LOOSE TO MEDIUM DENSE GRANULAR FILL, SOFT SILT, AND PEAT

UNIT 2: MEDIUM STIFF TO HARD SILT AND SILTY CLAY

UNIT 3: MEDIUM DENSE TO DENSE SAND AND SILTY SAND

UNIT 4: HARD SILT

NOTE
THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.

HORIZ. 0 20 40
VERT. 0 10 20
SCALE IN FEET

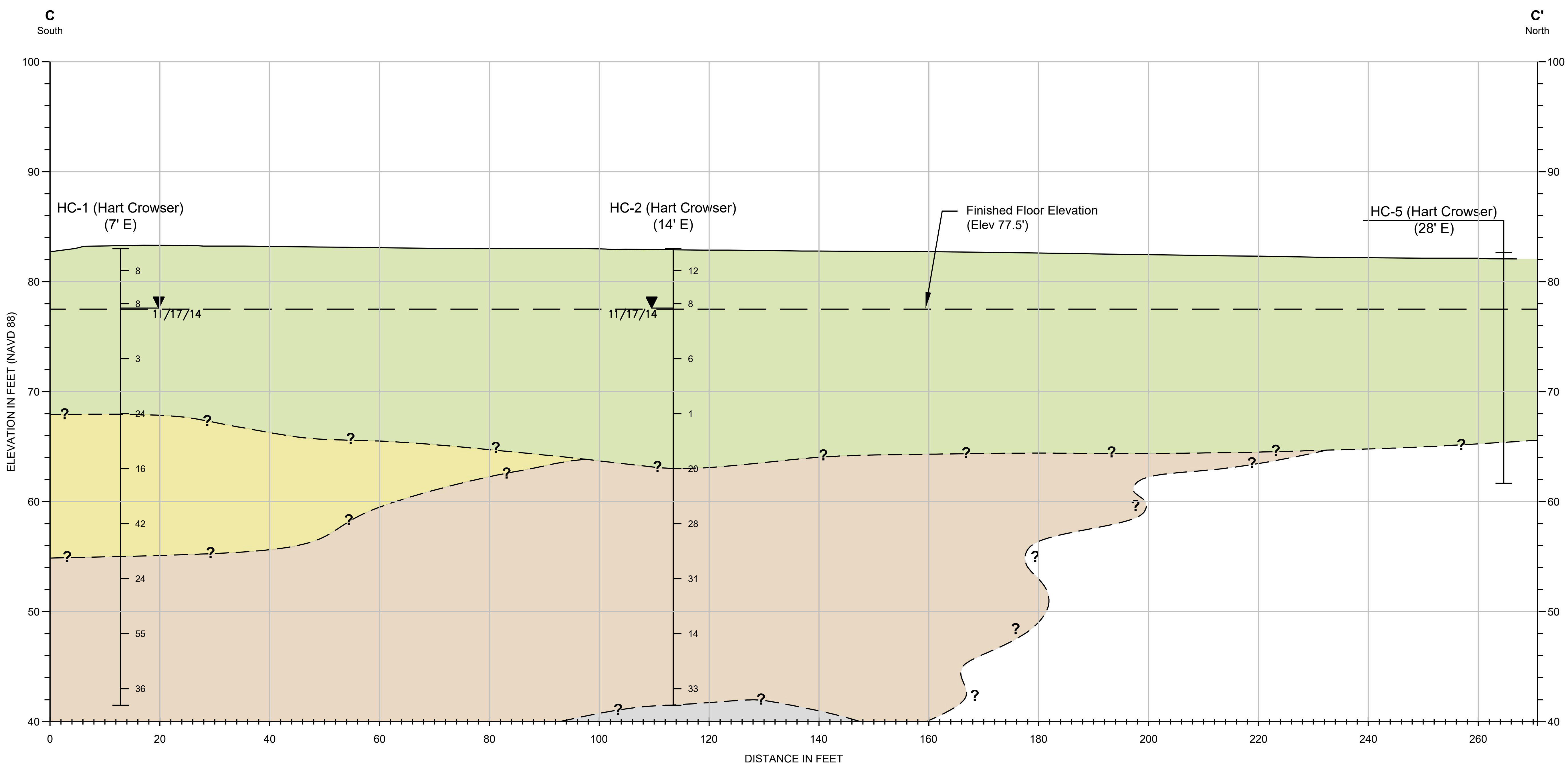
HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

GENERALIZED SUBSURFACE CROSS SECTION B-B'

JUNE 2023

FIGURE 4



LEGEND

B-1 (Terra) (16' W) — EXPLORATION NUMBER (OFFSET DISTANCE AND DIRECTION)

EXPLORATION LOCATION

ATD — WATER LEVEL

9 — STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

FINISHED FLOOR ELEVATION JOHNSTON ARCHITECTS, LLC PLANS DATED 10/1/2020

UNIT 1: LOOSE TO MEDIUM DENSE GRANULAR FILL, SOFT SILT, AND PEAT

UNIT 2: MEDIUM STIFF TO HARD SILT AND SILTY CLAY

UNIT 3: MEDIUM DENSE TO DENSE SAND AND SILTY SAND

UNIT 4: HARD SILT

NOTE
THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.

HORIZ. 0 20 40
VERT. 0 10 20
SCALE IN FEET

HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

GENERALIZED SUBSURFACE CROSS SECTION C-C'

JUNE 2023

FIGURE 5

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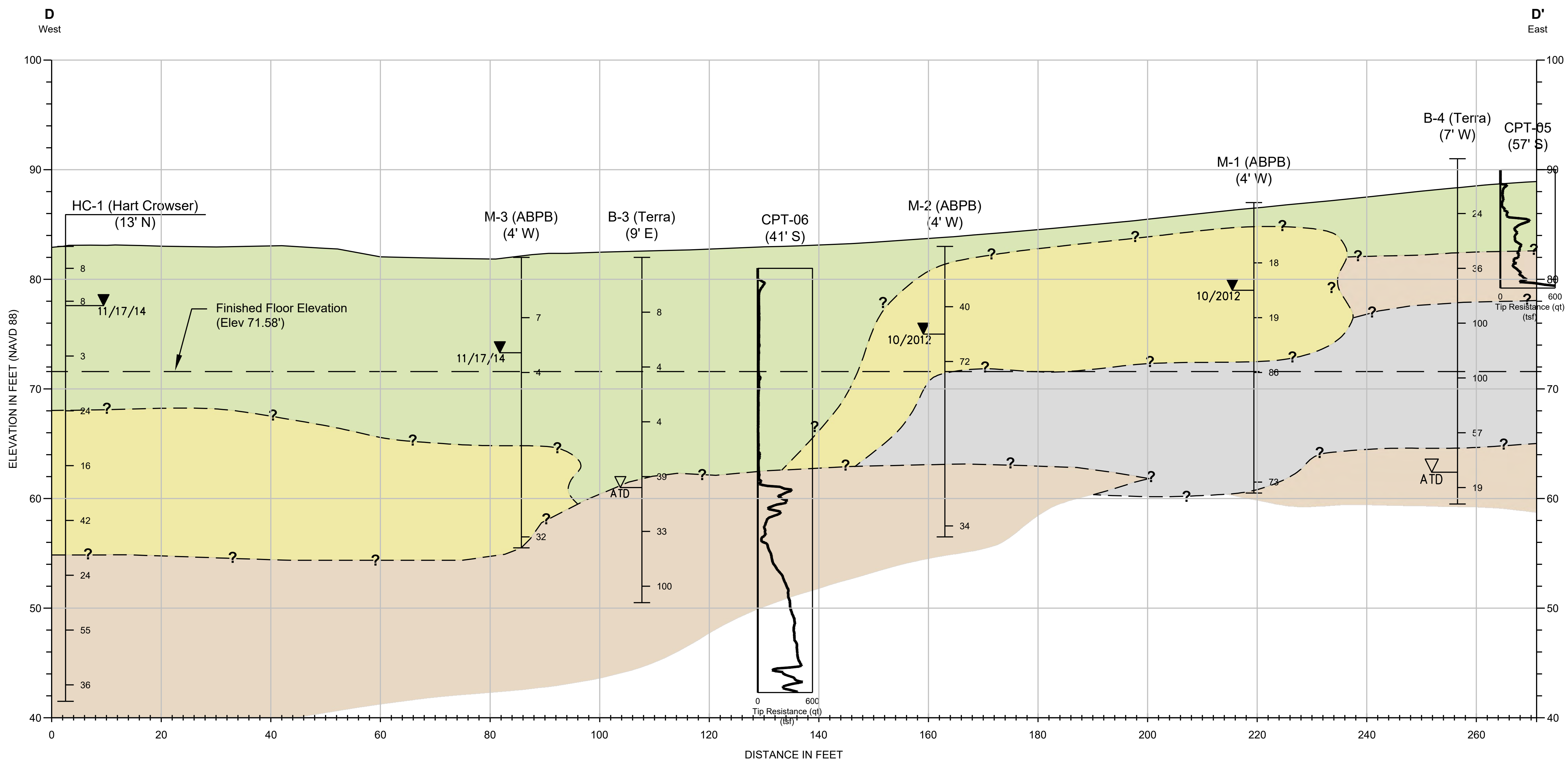


LEGEND

B-1 (Terra)
(16' W)
EXPLORATION NUMBER
(OFFSET DISTANCE AND DIRECTION)

EXPLORATION LOCATION

ATD WATER LEVEL

9 STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

FINISHED FLOOR ELEVATION
JOHNSTON ARCHITECTS, LLC PLANS
DATED 10/1/2020

CPT-05
(8' E)
CONE PENETROMETER NUMBER
(OFFSET DISTANCE AND DIRECTION)

CONE PENTROMETER LOCATION

0 600
Tip Resistance (qt)
(tsf)

UNIT 1: LOOSE TO MEDIUM DENSE GRANULAR FILL, SOFT SILT, AND PEAT

UNIT 2: MEDIUM STIFF TO HARD SILT AND SILTY CLAY

UNIT 3: MEDIUM DENSE TO DENSE SAND AND SILTY SAND

UNIT 4: HARD SILT

NOTE
THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.

HORIZ. 0 20 40
VERT. 0 10 20
SCALE IN FEET

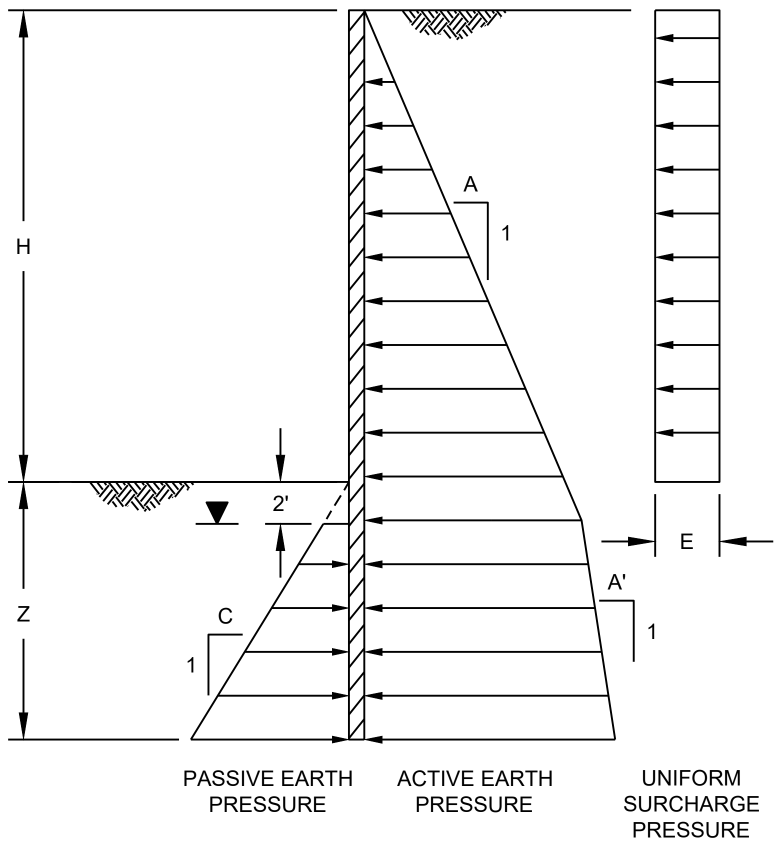
HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

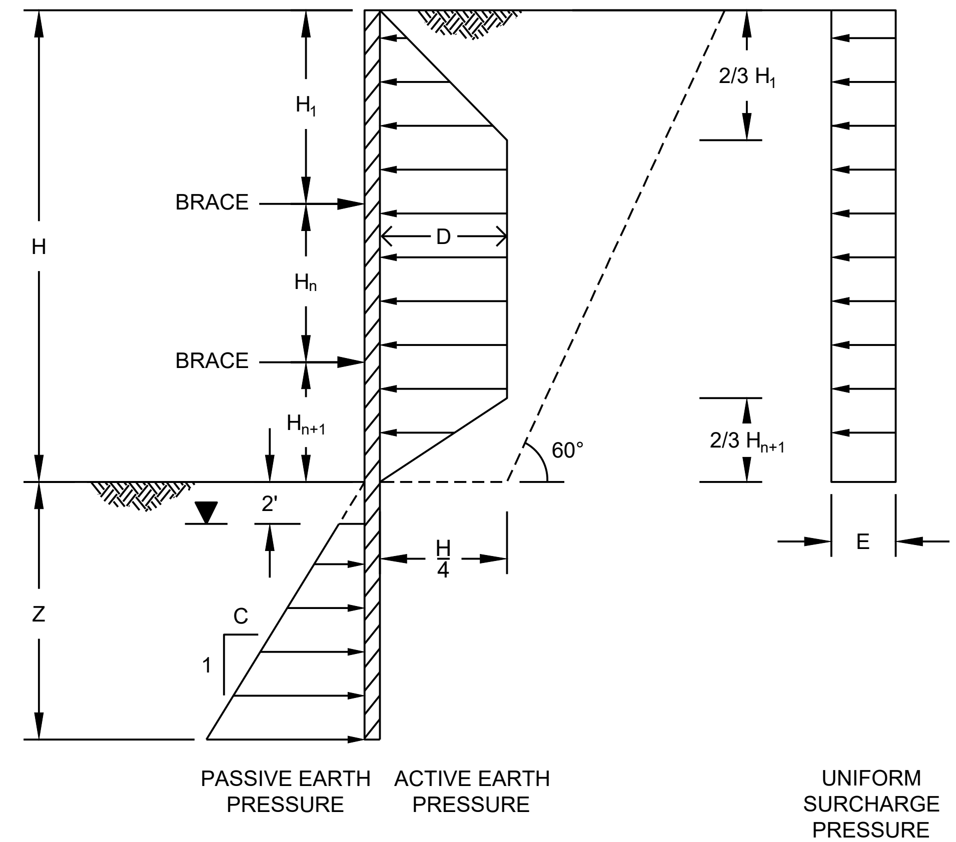
GENERALIZED SUBSURFACE
CROSS SECTION D-D'

JUNE 2023

FIGURE 6



CANTILEVER AND SINGLE BRACED WALL



MULTI BRACED WALL

Recommended Lateral Earth Pressures

	A (Above GWT)	A' (Below GWT)	C (Above GWT)	C (Below GWT)	D	E
ACTIVE	42 pcf	21 pcf	-	-	30H psf	85 psf
AT-REST	60 pcf	30 pcf	-	-	45H psf	125 psf
PASSIVE	-	-	300 pcf	175 pcf	-	-

NOTES

1. ALL EARTH PRESSURES ARE IN UNITS OF POUNDS PER SQUARE FOOT.
2. MINIMUM RECOMMENDED EMBEDMENT (Z) IS 8 FEET.
3. PASSIVE PRESSURES ARE ALLOWABLE VALUES AND INCLUDE A 1.5 FACTOR OF SAFETY.
4. PASSIVE PRESSURE ACTS OVER 2.5 TIMES THE CONCRETED DIAMETER OF THE SOLDIER PILE OR THE THE PILE SPACING, WHICHEVER IS LESS.
5. APPARENT EARTH PRESSURE, ACTIVE EARTH PRESSURE, AND SURCHARGE ACT OVER THE PILE SPACING ABOVE THE BASE OF THE EXCAVATION.
6. ACTIVE PRESSURE ACTS OVER THE PILE DIAMETER BELOW THE EXCAVATION.
7. ADDITIONAL SURCHARGE FROM FOOTINGS, LARGE STOCKPILES, HEAVY EQUIPMENT, ETC., MUST BE ADDED TO THESE PRESSURES.
8. ALL DIMENSIONS ARE IN FEET.
9. DIAGRAMS ARE NOT TO SCALE.

LEGEND

H	TOTAL HEIGHT OF EXCAVATION, FEET
H_1	DEPTH TO UPPERMOST TIEBACK, FEET
H_N	HEIGHT BETWEEN TIEBACKS, FEET
H_{n+1}	DISTANCE FROM BASE OF EXCAVATION TO LOWERMOST TIEBACK, FEET
Z	EMBEDMENT DEPTH, FEET
A,B,C, ...	EARTH PRESSURE FACTORS, SEE TABLE
- - - - - - -	NO-LOAD ZONE
▼	GROUNDWATER TABLE (GWT)

HALEY ALDRICH

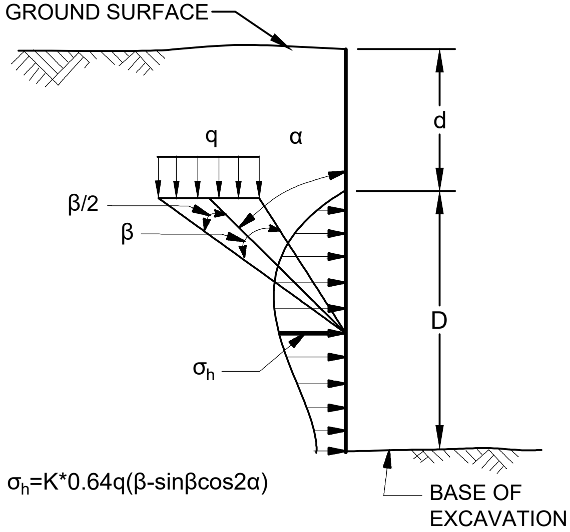
MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

LATERAL EARTH PRESSURE
TEMPORARY SHORING

NOT TO SCALE
JUNE 2023

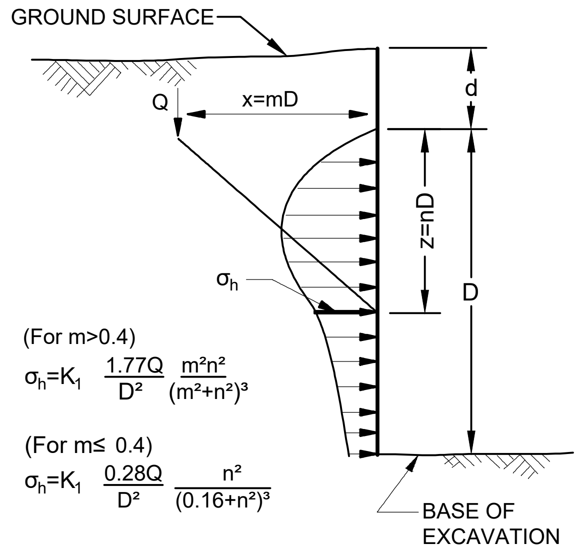
FIGURE 7

A. STRIP FOOTING CROSS SECTION VIEW



σ_h=K*0.64q(β-sinβcos2α)

B(1). SMALL ISOLATED FOOTING CROSS SECTION VIEW



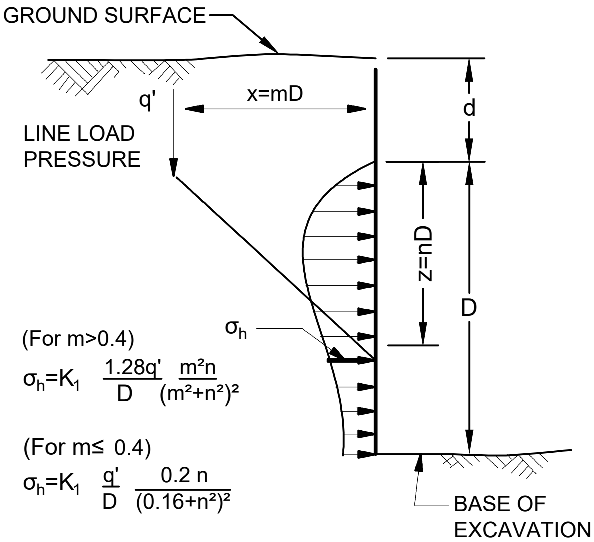
(For m>0.4)

$$\sigma_h = K_1 \frac{1.77Q}{D^2} \frac{m^2n^2}{(m^2+n^2)^3}$$

(For m≤ 0.4)

$$\sigma_h = K_1 \frac{0.28Q}{D^2} \frac{n^2}{(0.16+n^2)^3}$$

C. CONTINUOUS WALL FOOTING PARALLEL TO EXCAVATION CROSS SECTION VIEW



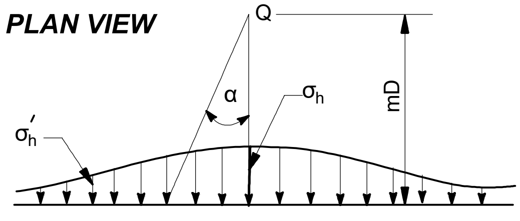
(For m>0.4)

$$\sigma_h = K_1 \frac{1.28q'}{D} \frac{m^2n}{(m^2+n^2)^2}$$

(For m≤ 0.4)

$$\sigma_h = K_1 \frac{q'}{D} \frac{0.2\,n}{(0.16+n^2)^2}$$

B(2). PLAN VIEW



$\sigma_h' = \sigma_h \cos^2(1.1\alpha)$

K_1	CONDITIONS
0.35	ACTIVE EARTH PRESSURE ON A FLEXIBLE WALL (E.G., SHORING)
0.5	AT-REST CONDITONS, WHERE SURCHARGE LOADS EXIST PRIOR TO EXCAVATION
1.0	AT-REST CONDITIONS, WHERE SURCHARGE LOADS ARE APPLIED AFTER CONSTRUCTION ON PERMANENT WALL

DEFINITION AND UNITS

Q	FOOTING LOAD IN POUNDS
D	EXCAVATION DEPTH BELOW FOOTING IN FEET
d	DEPTH TO BASE OF FOOTING IN FEET
σ_h	LATERAL SOIL PRESSURE IN PSF
q	UNIT LOADING PRESSURE IN PSF
q'	FOOTING LOAD IN POUNDS PER FOOT
α, β	RADIANS

NOTES:

1. LATERAL PRESSURES FROM ADJACENT STRUCTURES SHOULD BE ADDED TO LATERAL PRESSURES ON FIGURES 7 AND 9.
2. WALL FOOTINGS ACTING OTHER THAN PARALLEL TO THE EXCAVATION CAN BE TREATED AS SERIES OF DISCRETE POINT LOADS, USING DIAGRAM B.
3. CONTACT HALEY & ALDRICH FOR SURCHARGE RECOMMENDATIONS, IF NECESSARY.

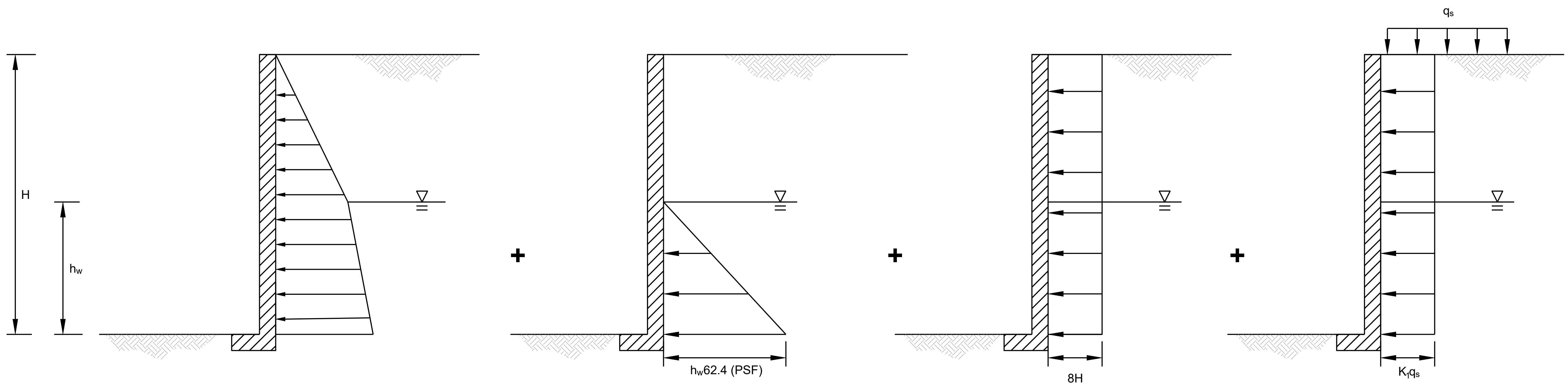
HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

SURCHARGE PRESSURES DETERMINATION OF LATERAL PRESSURE ACTING ON ADJACENT SHORING

NOT TO SCALE
JUNE 2023

FIGURE 8



EARTH PRESSURE*

* THE SAME EARTH PRESSURE DISTRIBUTIONS DETERMINED FOR TEMPORARY SHORING SHOULD BE USED FOR PERMANENT WALLS CONSTRUCTED AGAINST SHORING (SEE FIGURE 7).

NET HYDROSTATIC WATER PRESSURE**

** NEGLECT WATER PRESSURE IF PERMANENTLY DRAINED

DYNAMIC INERTIAL INCREMENT

UNIFORM SURCHARGE***

*** SEE FIGURE 8 FOR K_1

LEGEND

H	HEIGHT FROM BOTTOM OF EXCAVATION TO GROUND SURFACE IN FEET
q_s	TRAFFIC SURCHARGE
h_w	DEPTH OF EXCAVATION BELOW GROUNDWATER TABLE
▽	GROUNDWATER TABLE

NOTES

1. ALL PRESSURES ARE IN UNITS OF POUNDS PER SQUARE FOOT.
2. DIAGRAMS DO NOT INCLUDE SURCHARGE LOADING DUE TO ADJACENT STRUCTURES; SEE FIGURE 8.
3. DIAGRAMS NOT TO SCALE.

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

LATERAL PRESSURES FOR PERMANENT WALLS CONSTRUCTED AGAINST SHORING

NOT TO SCALE
JUNE 2023

FIGURE 9

LEGEND

HC-3 ●	BORING (HART CROWSER)
HC-5 ▲	HAND PROBE (HART CROWSER)
HC-1	MONITORING WELL (HART CROWSER)
CPT-01 ◐	CONE PENETROMETER TESTING (HART CROWSER)
B-1 ■	BORING (ABPB CONSULTING)
B-6 ◐	PUSH PROBE (FARALLON)
M-1	MONITORING WELL (ABPB CONSULTING)
B-1 ⊙	BORING (TERRA)
- - -	TOP OF COMPETENT SOILS (CONTOUR ELEVATION IN FEET)
▨	APPROXIMATE ZONE OF GROUND IMPROVEMENT

NOTES

1. FEATURE LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. SURVEY BASE SOURCE: BUSH, ROED & HITCHINGS 14 OCTOBER 2014.

HALEY ALDRICH

MERCER ISLAND MULTI-FAMILY HOUSING
MERCER ISLAND, WASHINGTON

ELEVATION OF TOP OF COMPETENT SOILS

JUNE 2023

FIGURE 10

APPENDIX A
Field Exploration Methods and Analysis

Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

Sampling Test Symbols

- 1.5" I.D. Split Spoon
- Grab (Jar)
- 3.0" I.D. Split Spoon
- Shelby Tube (Pushed)
- Bag
- Cuttings
- Core Run

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Moisture

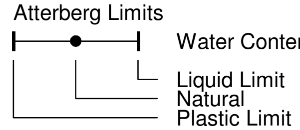
Dry	Little perceptible moisture
Damp	Some perceptible moisture, likely below optimum
Moist	Likely near optimum moisture content
Wet	Much perceptible moisture, likely above optimum

Minor Constituents

Minor Constituents	Estimated Percentage
Trace	<5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Laboratory Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits

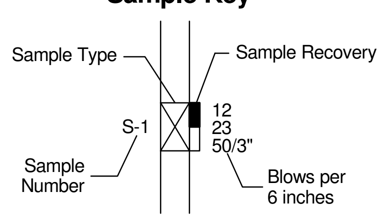


PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density in PCF
OT	Tests by Others

Groundwater Indicators

- Groundwater Level on Date or (ATD) At Time of Drilling
- Groundwater Seepage (Test Pits)

Sample Key



HARTCROWSER

17984-01 11/14

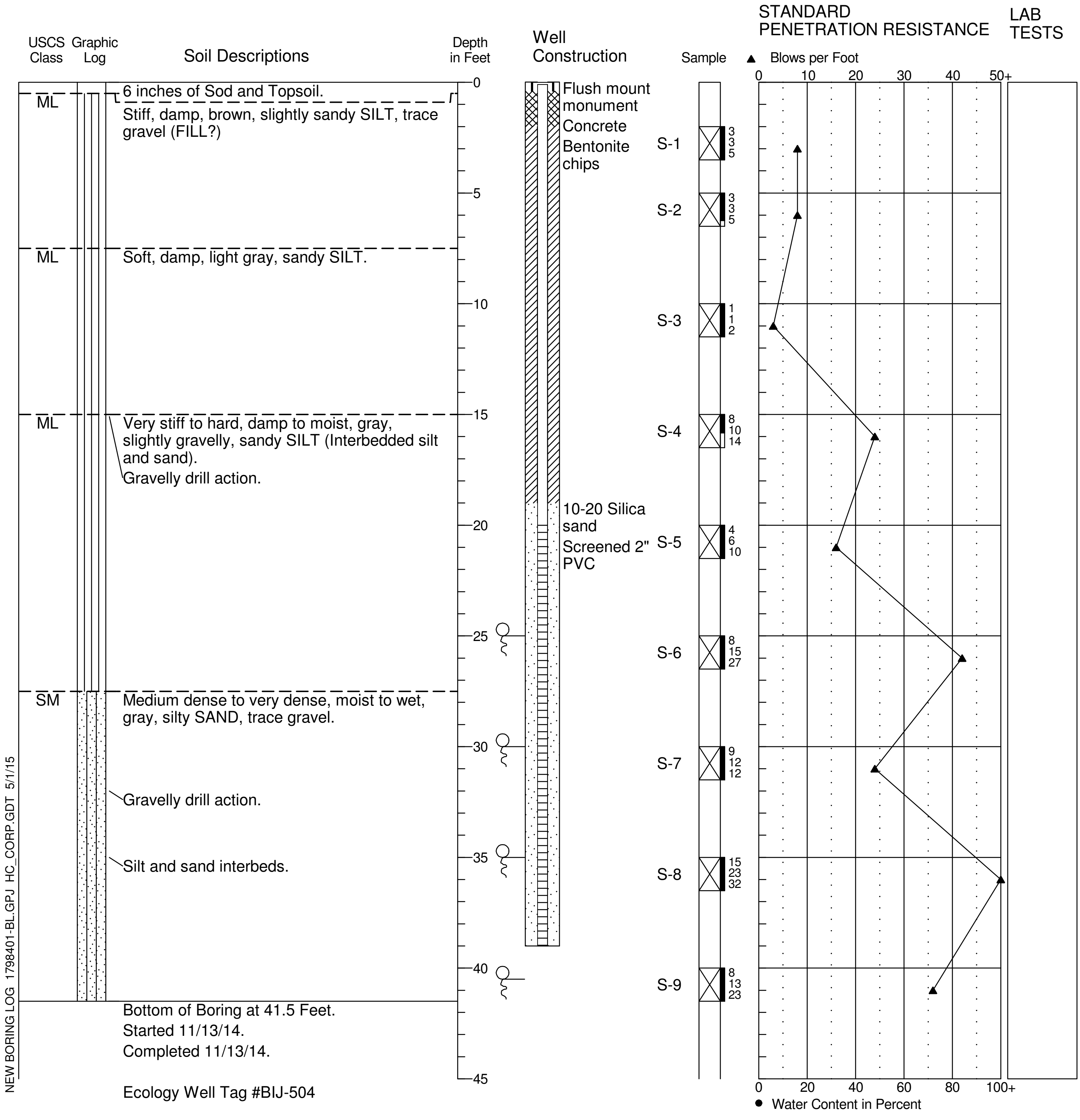
Figure A-1

KEY SHEET 1798401-BL.GPJ HC_CORP.GDT 5/1/15

Boring Log HC-1

Location: 47.584459, -122.234890
Approximate Ground Surface Elevation: 83 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: HSA (Diedrich D50)
Hammer Type: Auto-Trip
Hole Diameter: 8 inches
Logged By: M. Smith   Reviewed By: M. Veenstra



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

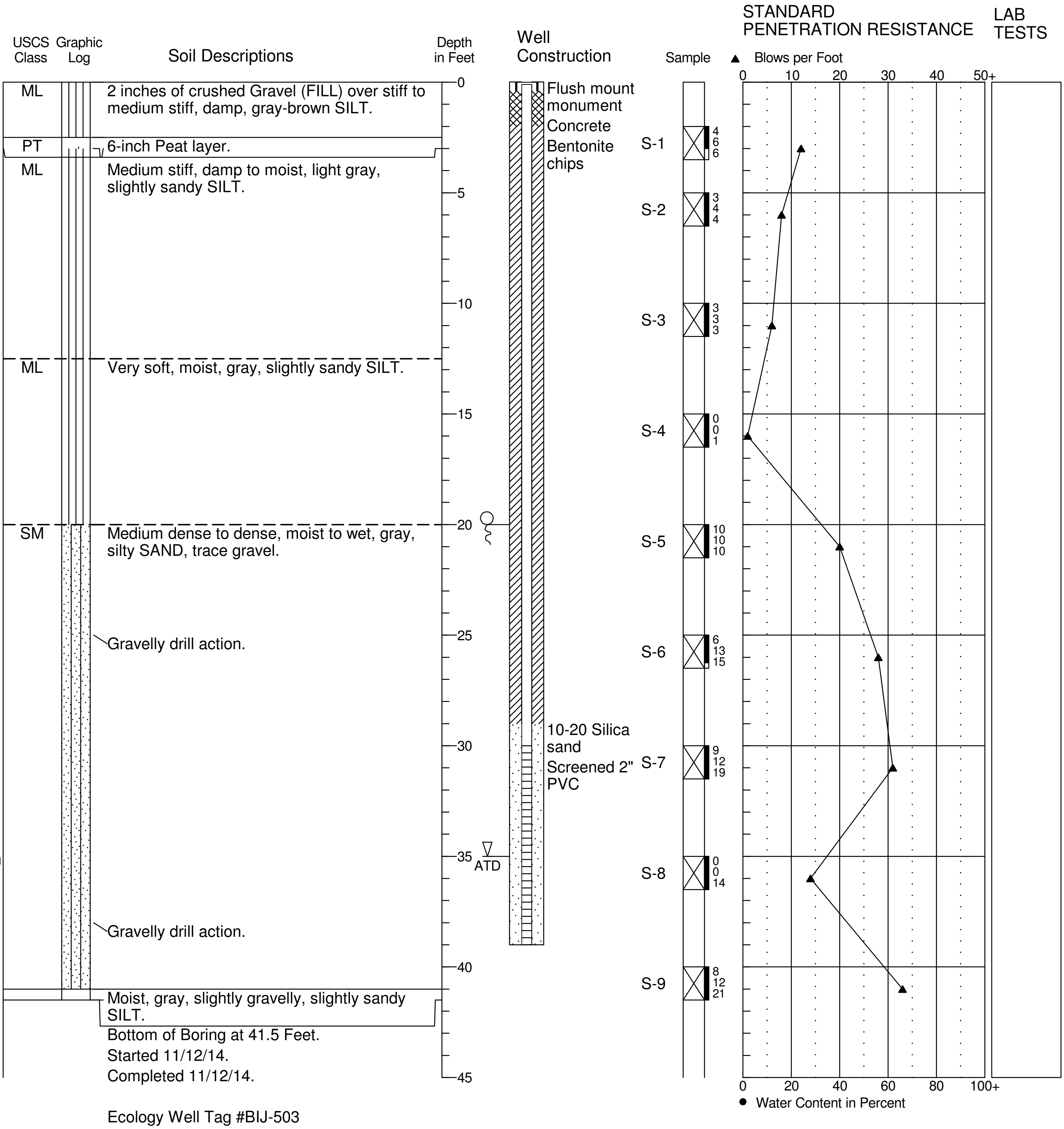
17984-01 *11/14*

Figure A-2

Boring Log HC-2

Location: 47.584729, -122.234870
Approximate Ground Surface Elevation: 83 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: HSA (Diedrich D50)
Hammer Type: Auto-Trip
Hole Diameter: 8 inches
Logged By: M. Smith   Reviewed By: M. Veenstra



Ecology Well Tag #BIJ-503

NEW BORING LOG 1798401-BL.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

17984-01   11/14

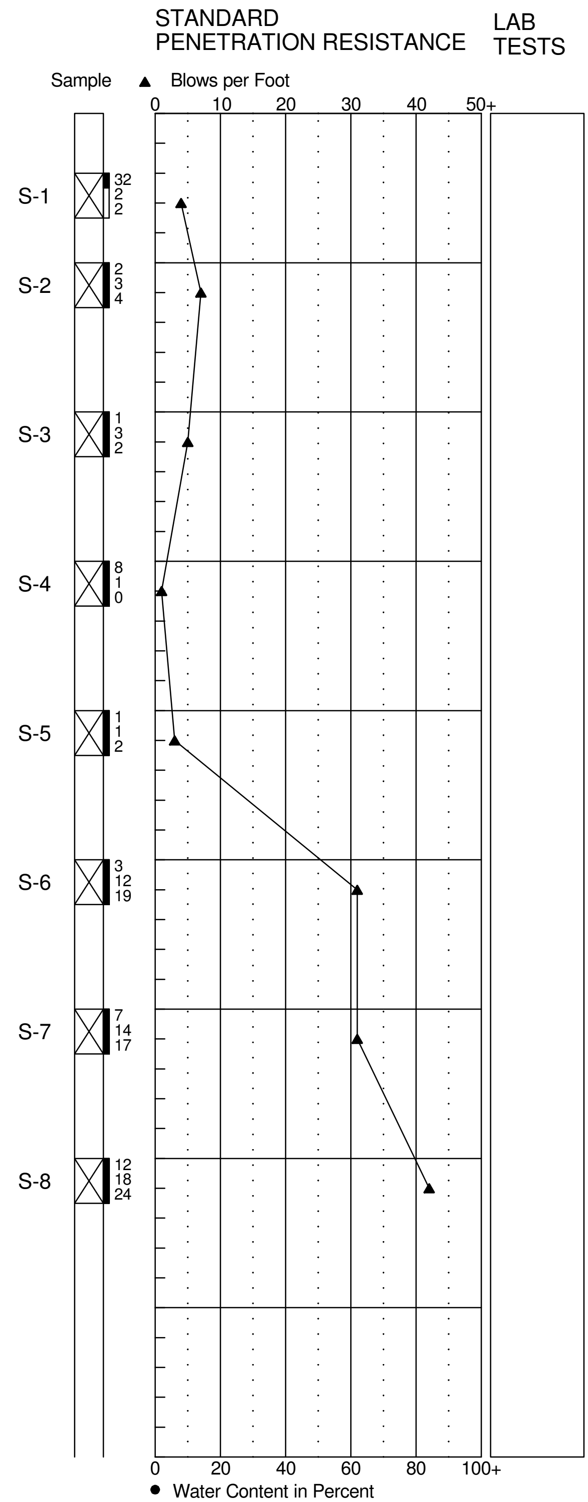
Figure A-3

Boring Log HC-3

Location: 47.585134, -122.234493
Approximate Ground Surface Elevation: 83 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: HSA (Diedrich D50)
Hammer Type: Auto-Trip
Hole Diameter: 8 inches
Logged By: M. Smith   Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet
ML	3 inches of Asphalt Concrete Pavement. Soft, damp, gray-brown, slightly gravelly, sandy SILT. (FILL)	0
ML	Medium stiff, moist, brown SILT, trace sand and gravel. (FILL)	5
ML	Medium stiff, moist to wet, gray SILT, trace sand and gravel.	10
ML	Very soft to soft, moist, gray SILT, trace sand.	15, 20
ML	Hard, moist, gray, slightly gravelly, sandy SILT.	25
SM	Dense, wet, gray, silty SAND, trace gravel.	30 ATD, 35
ML	Moist, gray SILT. Bottom of Boring at 36.5 Feet. Started 11/12/14. Completed 11/12/14.	40, 45



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

17984-01 *11/14*

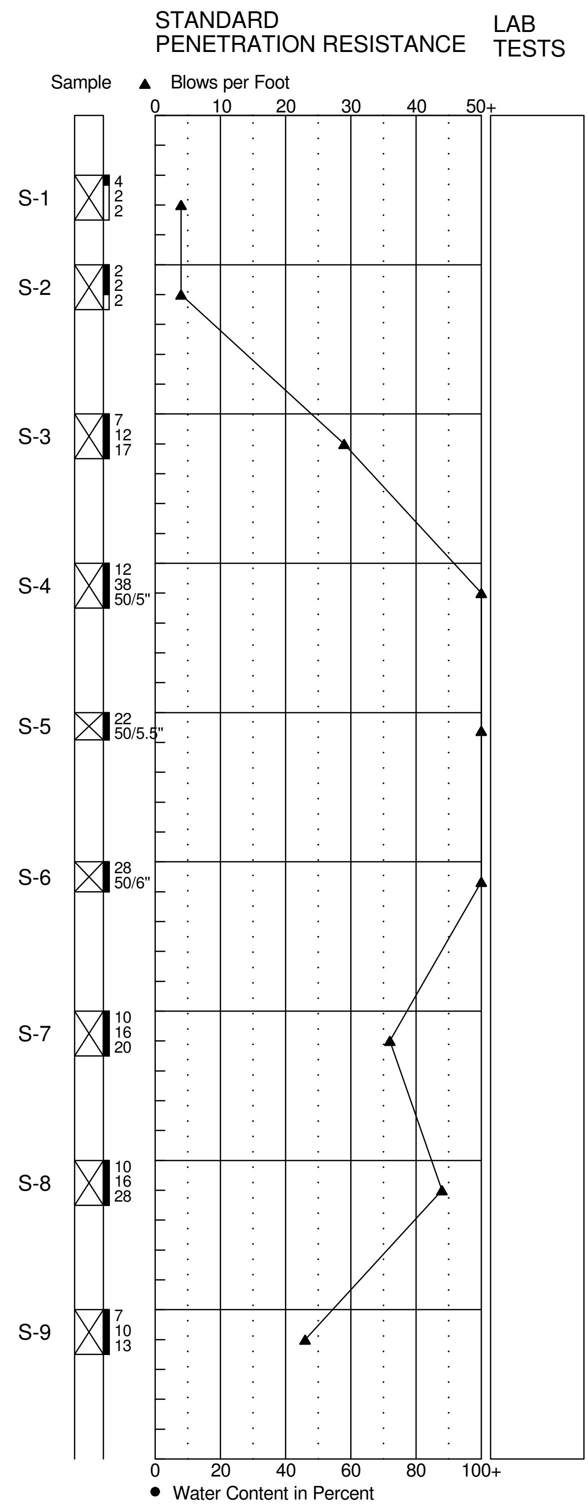
Figure A-4

Boring Log HC-4

Location: 47.585142, -122.233965
Approximate Ground Surface Elevation: 88 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: HSA (Diedrich D50)
Hammer Type: Auto-Trip
Hole Diameter: 8 inches
Logged By: M. Smith Reviewed By: M. Veenstra

USCS Class	Soil Descriptions
SM/ML	3 inches of Asphalt Concrete Pavement. Very loose, damp to moist, silty, gravelly SAND and gravelly, sandy SILT. (FILL) Gravelly drill action.
ML	Very loose, damp, gray, gravelly, sandy SILT.
ML	Very stiff to hard, damp, light brown to gray, slightly gravelly, sandy SILT. Gravelly drill action.
ML	Hard to very stiff, moist, gray, sandy SILT with interbedded sand and silt layers. Sand layers wet, silt layers moist. Gravelly drill action.
	Bottom of Boring at 41.5 Feet. Started 11/12/14. Completed 11/12/14.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

17984-01 *11/14*

Figure A-5

Push Probe Log B-1

Location: Lat: 47.58453 Long: -122.2343
Approximate Ground Surface Elevation: 82 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet	Sample	LAB TESTS & (PID)
	2 inches of Asphalt.	0		
SP	(Loose), moist, gray-brown, slightly silty, slightly gravelly SAND. (FILL)		S-1	(<0.1) No odor, NS
ML	(Medium stiff to stiff), moist, gray-brown, mottled, clayey SILT with fine sand pockets and trace organic material.	5	S-2	(<0.1) No odor, NS PP=1.0 TSF
ML	(Soft to medium stiff), moist to wet, brown, sandy SILT.	ATD	S-3	(<0.1) No odor, NS
CL-ML	(Soft), moist, gray, clayey SILT.	10		
	Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13.	15		

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-6

Push Probe Log B-2

Location: Lat: 47.58454 Long: -122.2345
Approximate Ground Surface Elevation: 82 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet	Sample	LAB TESTS & (PID)
	2 inches of Asphalt.	0		
SP	(Loose to medium dense), moist, brown, slightly silty, gravelly, fine to coarse SAND. (FILL)		S-1	(<0.1) Slight odor, NS
ML	(Medium stiff to stiff), moist, gray, slightly sandy SILT with trace organic material to (soft to medium stiff), moist, gray to red-brown, mottled, clayey SILT with fine sand pockets.	5	S-2	(<0.1) No odor, NS PP=2.0 TSF
	Wet.	∇ ATD		
			S-3	(<0.1) No odor, NS PP=0.5 TSF
CL-ML	(Soft), moist, gray, clayey SILT.	10		
	Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13.			
		15		

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-7

Push Probe Log B-3

Location: Lat: 47.58477 Long: -122.2349
Approximate Ground Surface Elevation: 84 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet	Sample	LAB TESTS
GP	4 inches of Gravel over (medium dense), moist, brown-gray, slightly silty, sandy GRAVEL.	0	S-1	No odor, NS
ML	(Soft), moist, red-brown, sandy SILT to black organic SILT.			
ML	(Very stiff), moist, red-brown to gray, slightly sandy, mottled SILT with scattered organic material.	5	S-2	No odor, NS PP=3.0 TSF
ML	(Stiff), moist, gray, laminated, slightly sandy to sandy SILT.	10	S-3	No odor, NS PP=1.5 TSF
	Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13.	15		

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-8

Push Probe Log B-4

Location: Lat: 47.58468 Long: -122.2348
Approximate Ground Surface Elevation: 84 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet
	4 inches of sandy GRAVEL.	0
ML	(Soft), moist, brown, gravelly, sandy SILT. (FILL)	
GP SP	(Loose), moist, gray to red-brown, sandy GRAVEL to fine to medium SAND.	
ML	(Stiff to very stiff), moist, red-brown to gray, mottled SILT with scattered organic material.	5
CL-ML	(Medium stiff to stiff), moist, blue-gray to brown, clayey SILT to silty CLAY with occasional laminated, slightly sandy silt seams.	10
	Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13. Approximately 4 feet of water observed in hole after completion.	15

Sample	LAB TESTS
S-1	No odor PP=0.5 TSF
S-2	No odor PP=2.5 TSF
S-3	No odor PP=1.5 TSF

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-9

Push Probe Log B-5

Location: Lat: 47.5846 Long: -122.2346
Approximate Ground Surface Elevation: 81 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	LAB TESTS
		6 inches of SOD.	0		
		No recovery.		S-1	
CL-ML		(Stiff to very stiff), moist, blue-gray to red-brown, mottled, clayey SILT to silty CLAY with occasional fine sand seams.	5	S-2	No odor PP=2.0 TSF
				S-3	No odor PP=1.5 TSF
			10		
		Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13.			
			15		

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-10

Push Probe Log B-6

Location: Lat: 47.58482 Long: -122.2345
Approximate Ground Surface Elevation: 81 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald   Reviewed By: M. Veenstra

USCS Class	Soil Descriptions	Depth in Feet	Sample	LAB TESTS
	4 inches of Asphalt over 8 inches of Base Course.	0		
ML CL	(Very stiff), moist, blue-gray to red-brown, mottled, clayey SILT to silty CLAY with scattered organic material.		S-1	No odor PP=2.0 TSF
		5	S-2	No odor PP=2.0 TSF
CL-ML	(Soft to medium stiff), moist to wet, red-brown to gray, mottled, silty CLAY to clayey SILT with fine silty sand seams.		S-3	No odor PP=0.5 TSF
		10		
	Bottom of Probe at 12.0 Feet. Started 09/27/13. Completed 09/27/13.			
		15		

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

Push Probe Log B-7

Location: Lat: 47.58514 Long: -122.2342
Approximate Ground Surface Elevation: 86 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald   Reviewed By: M. Veenstra

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	LAB TESTS
		4 inches of Asphalt over 8 inches of Base Course.	0		
CL-ML		(Very stiff), moist, blue-gray, gravelly, sandy, silty CLAY to clayey SILT with black silty organic clay zones. (FILL)		S-1	No odor PP=2.75 TSF
CL-ML		(Very stiff), moist, blue-gray to red-brown, mottled, silty CLAY to clayey SILT with scattered organic material.	5	S-2	No odor PP=3.5 TSF
		Grades to (stiff), moist, blue-gray to red-brown, laminated, slightly sandy, clayey SILT to silty CLAY.		S-3	No odor PP=1.5 TSF
			10		
		Grades to (very soft to medium stiff), moist to wet, blue-gray to red-brown, mottled, silty CLAY.		S-4	No odor PP=<0.25 TSF
			15		

Bottom of Probe at 16.0 Feet.
Started 09/27/13.
Completed 09/27/13.

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 9/13
Figure A-12

Push Probe Log B-8

Location: Lat: 47.58477 Long: -122.2338
Approximate Ground Surface Elevation: 92 Feet
Horizontal Datum: WGS 84
Vertical Datum: NAVD88

Drill Equipment: Push Probe
Sample Type: Acetate Liner
Hole Diameter: 2 inches
Logged By: W. McDonald Reviewed By: M. Veenstra

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	LAB TESTS
		4 inches of Asphalt over Base Course and Brick.	0		
ML		(Medium stiff), moist, brown to gray, gravelly, sandy SILT. (FILL)		S-1	No odor PP=1.0 TSF
ML		(Very stiff), moist, gray, slightly mottled, fine to medium sandy SILT.	5	S-2	No odor PP=3.0 TSF
ML		(Very stiff to hard), damp, brown, fine to medium sandy SILT.		S-3	No odor
		Grades to moist brown, slightly sandy SILT with occasional organic material.	10	S-4	No odor PP=>4.0 TSF
		Bottom of Probe at 13.0 Feet. Started 09/27/13. Completed 09/27/13.	15		

PUSH PROBE LOG-ENV 1798400-PP.GPJ HC_CORP.GDT 5/1/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. NS = No Sheen; SS = Slight Sheen; MS = Moderate Sheen; HS = Heavy Sheen

HARTCROWSER

17984-00 *9/13*

Figure A-13

WILDCAT DYNAMIC CONE LOG

Hart Crowser
1700 Westlake Ave N.
Seattle, WA 98109

PROJECT NUMBER: 1798401
DATE STARTED: 11-20-2014
DATE COMPLETED: 11-20-2014

HOLE #: HC-5
CREW: Jesse Overton
PROJECT: Mercer Island Multi-Family
ADDRESS:
LOCATION: Mercer Island, Washington

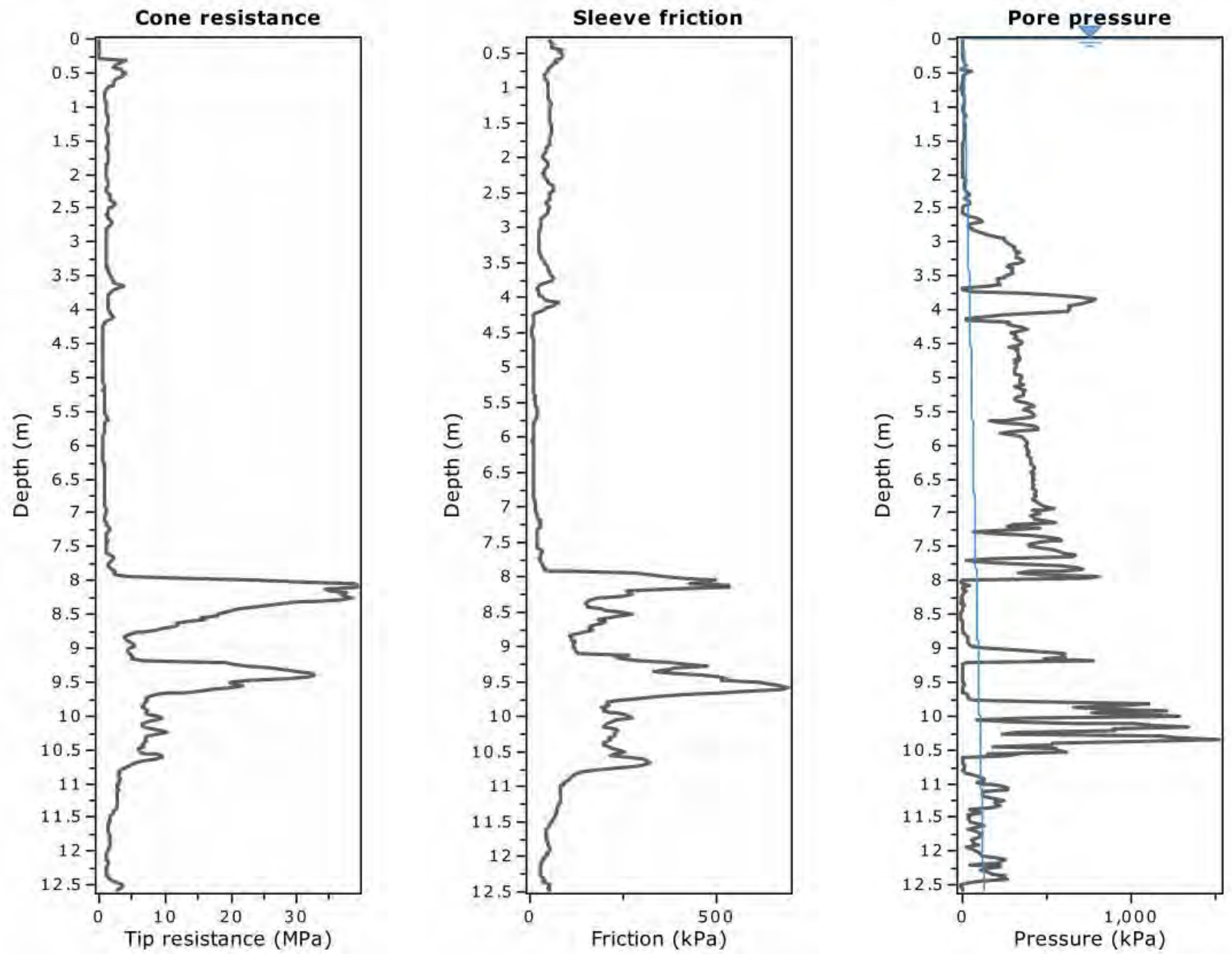
SURFACE ELEVATION:
WATER ON COMPLETION:
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	COHESIVE
-	18	79.9	••••••••••••••••••••••	22	MEDIUM DENSE	VERY STIFF
-	23	102.1	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 1 ft	14	62.2	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	12	53.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
-	10	44.4	••••••••••••	12	MEDIUM DENSE	STIFF
- 2 ft	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	10	44.4	••••••••••••	12	MEDIUM DENSE	STIFF
- 3 ft	14	62.2	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
- 1 m	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	11	42.5	••••••••••••	12	MEDIUM DENSE	STIFF
- 4 ft	11	42.5	••••••••••••	12	MEDIUM DENSE	STIFF
-	11	42.5	••••••••••••	12	MEDIUM DENSE	STIFF
-	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
- 5 ft	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 6 ft	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 2 m	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 7 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	5	17.1	••••	4	VERY LOOSE	SOFT
- 8 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	5	17.1	••••	4	VERY LOOSE	SOFT
-	5	17.1	••••	4	VERY LOOSE	SOFT
- 9 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
- 3 m 10 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	12	36.7	••••••••••	10	LOOSE	STIFF
- 11 ft	9	27.5	•••••••	7	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
- 12 ft	4	12.2	•••	3	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 4 m 13 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF

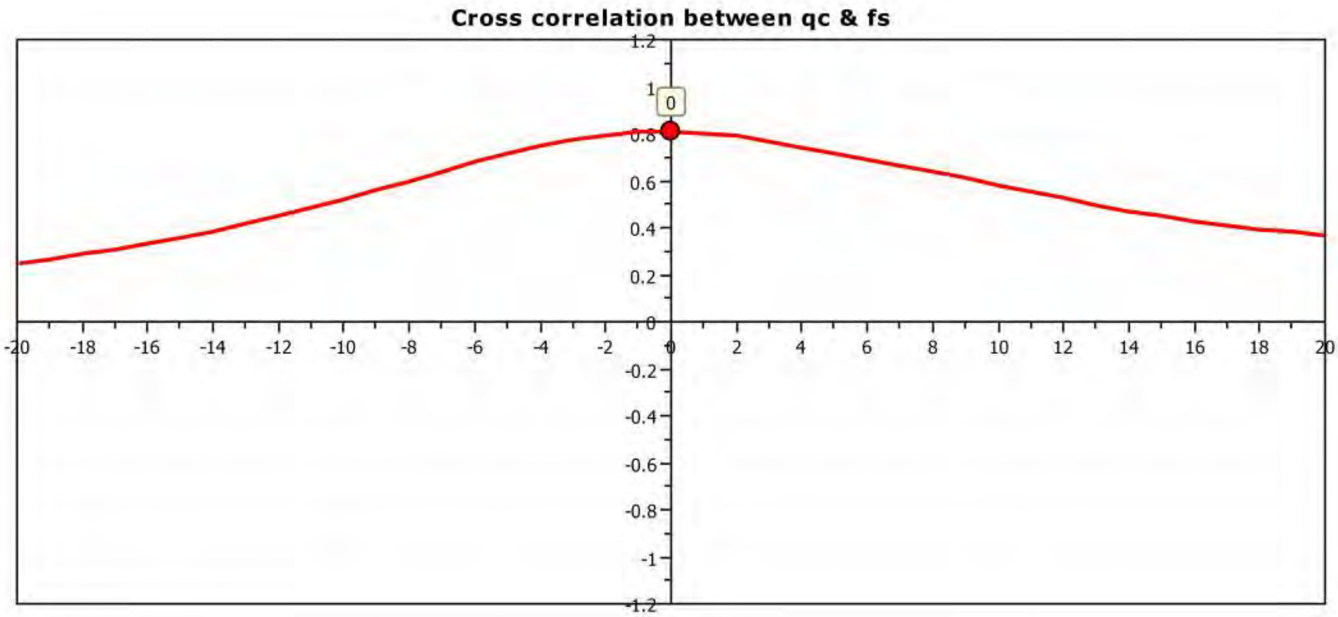
HOLE #: HC-5 **WILDCAT DYNAMIC CONE LOG** Page 2 of 2

PROJECT: Mercer Island Multi-Family PROJECT NUMBER: 1798401

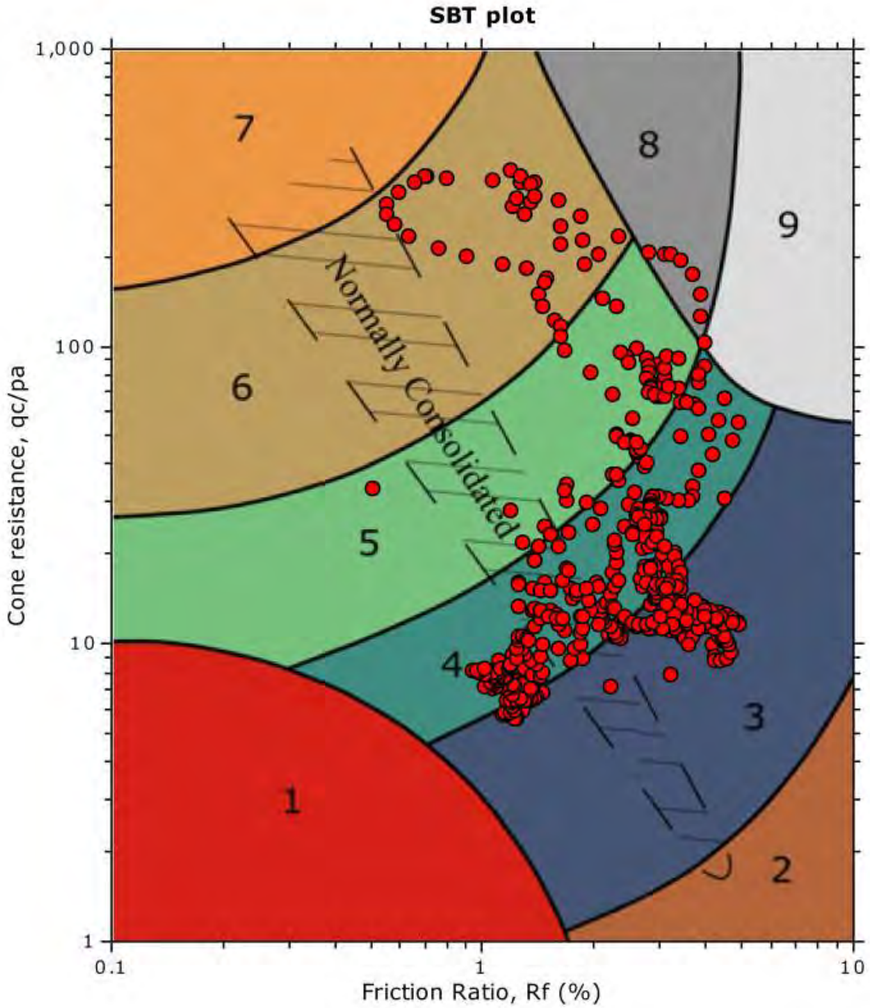
DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	COHESIVE
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
- 14 ft	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
-	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
-	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 15 ft	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
-	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
-	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
- 16 ft	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 5 m	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	9	22.9	••••••	6	LOOSE	MEDIUM STIFF
- 17 ft	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
-	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
-	12	30.5	••••••••	8	LOOSE	MEDIUM STIFF
- 18 ft	11	27.9	••••••••	7	LOOSE	MEDIUM STIFF
-	12	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	24	61.0	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
- 19 ft	33	83.8	••••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
-	21	53.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 6 m	21	53.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 20 ft	20	46.6	•••••••••••••	13	MEDIUM DENSE	STIFF
-	28	65.2	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
-	50	116.5	•••••••••••••••••••••••••••••••••	25+	DENSE	HARD
- 21 ft						
-						
-						
- 22 ft						
-						
-						
- 7 m 23 ft						
-						
-						
- 24 ft						
-						
-						
- 25 ft						
-						
-						
- 26 ft						
- 8 m						
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						

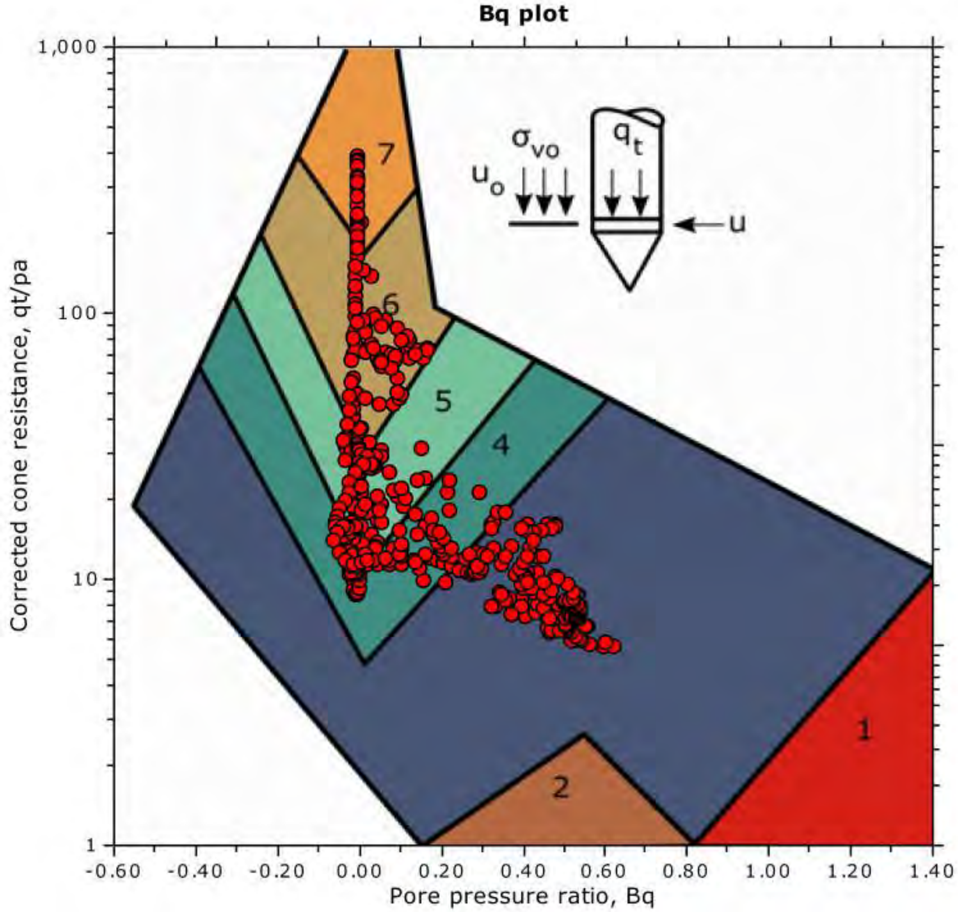


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots

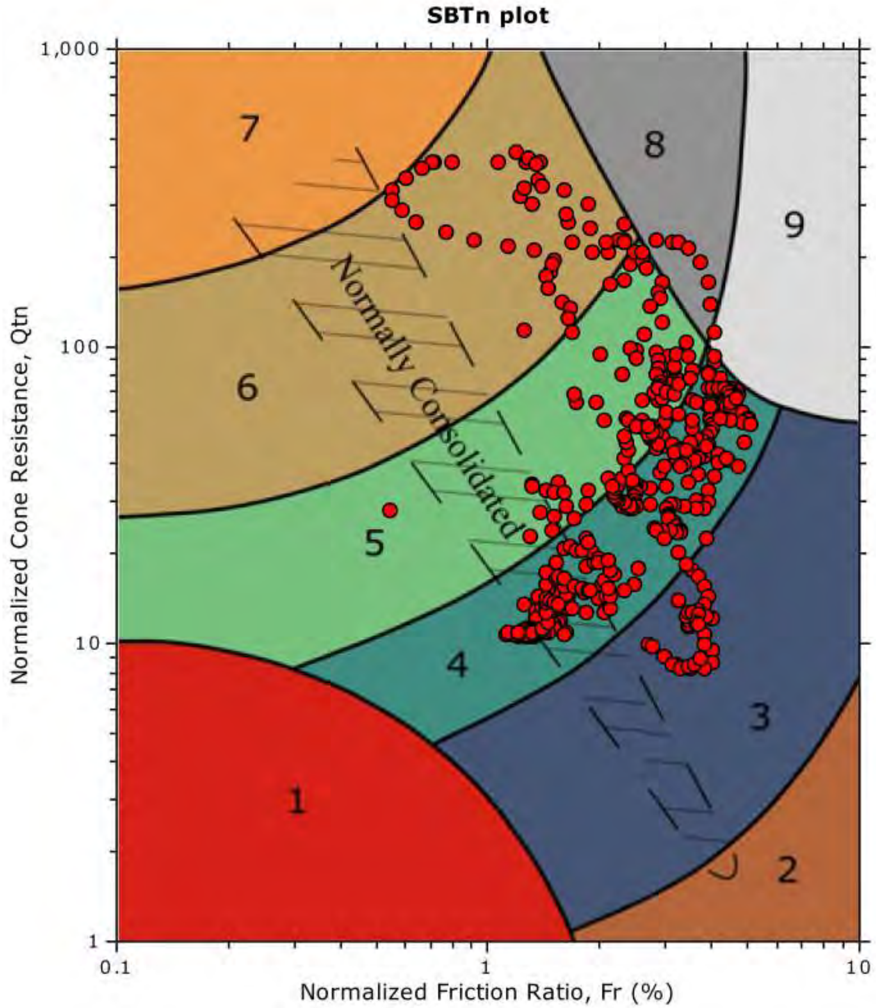


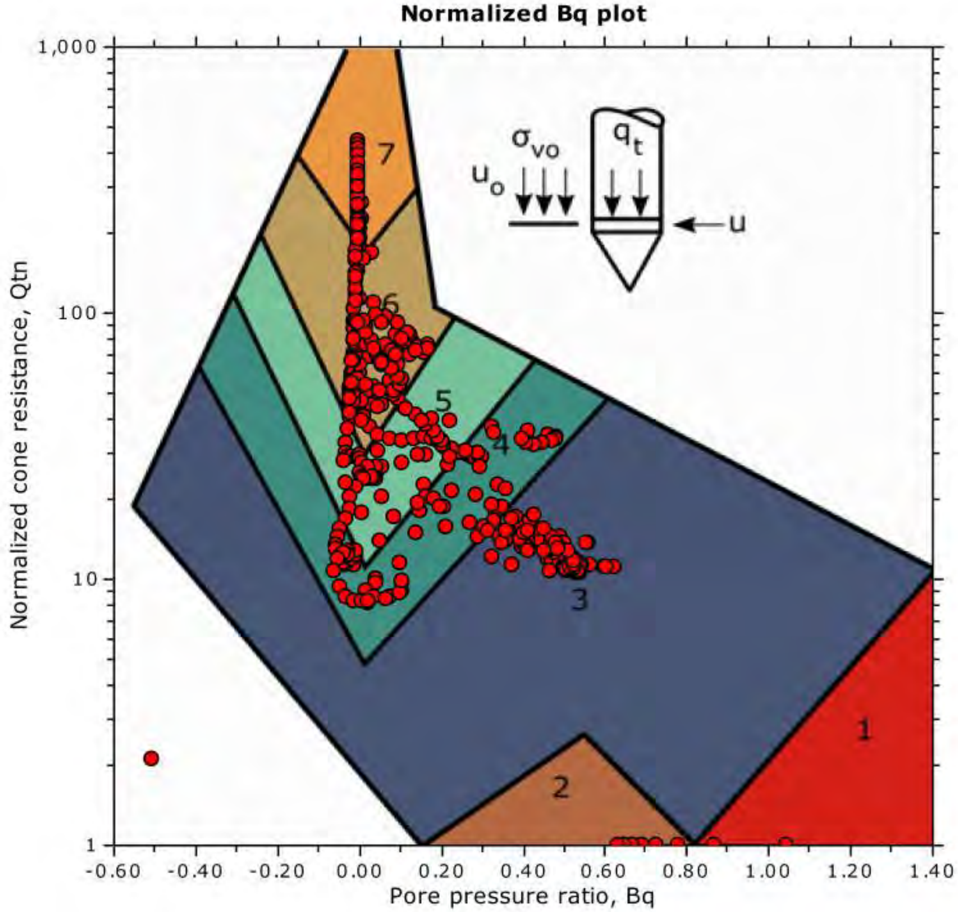


SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

SBT - Bq plots (normalized)

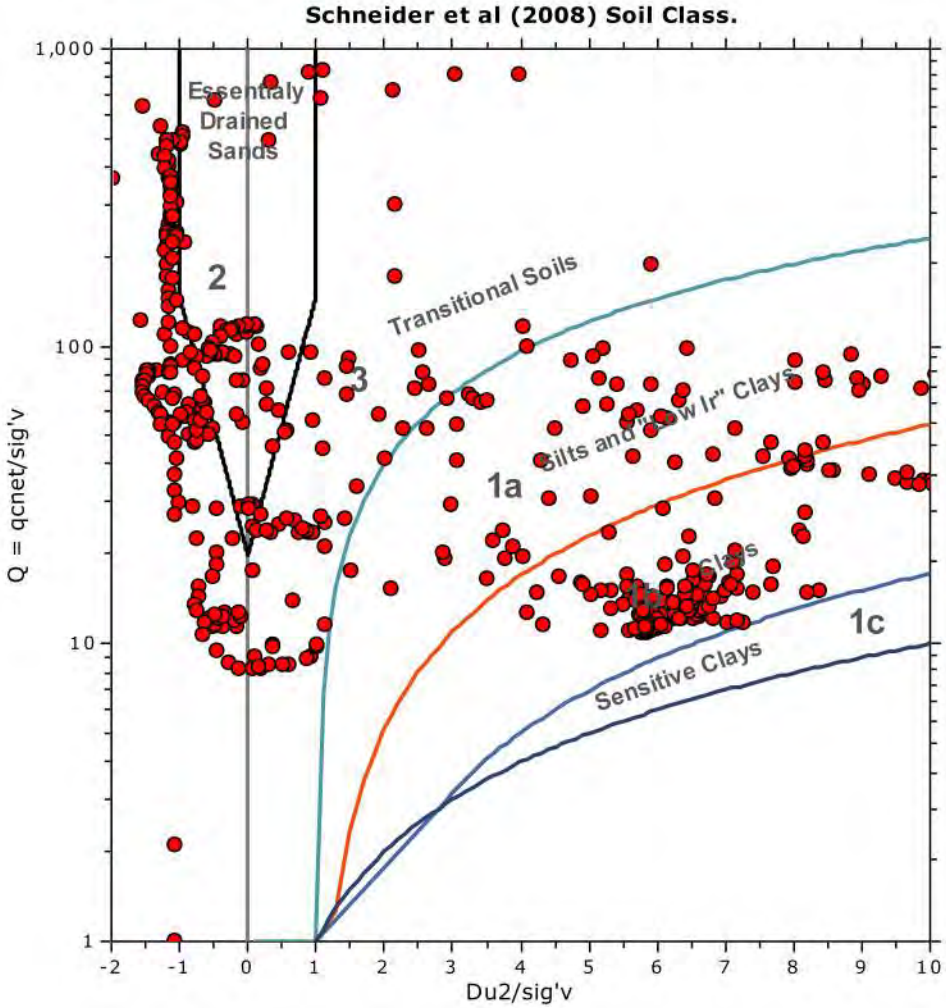


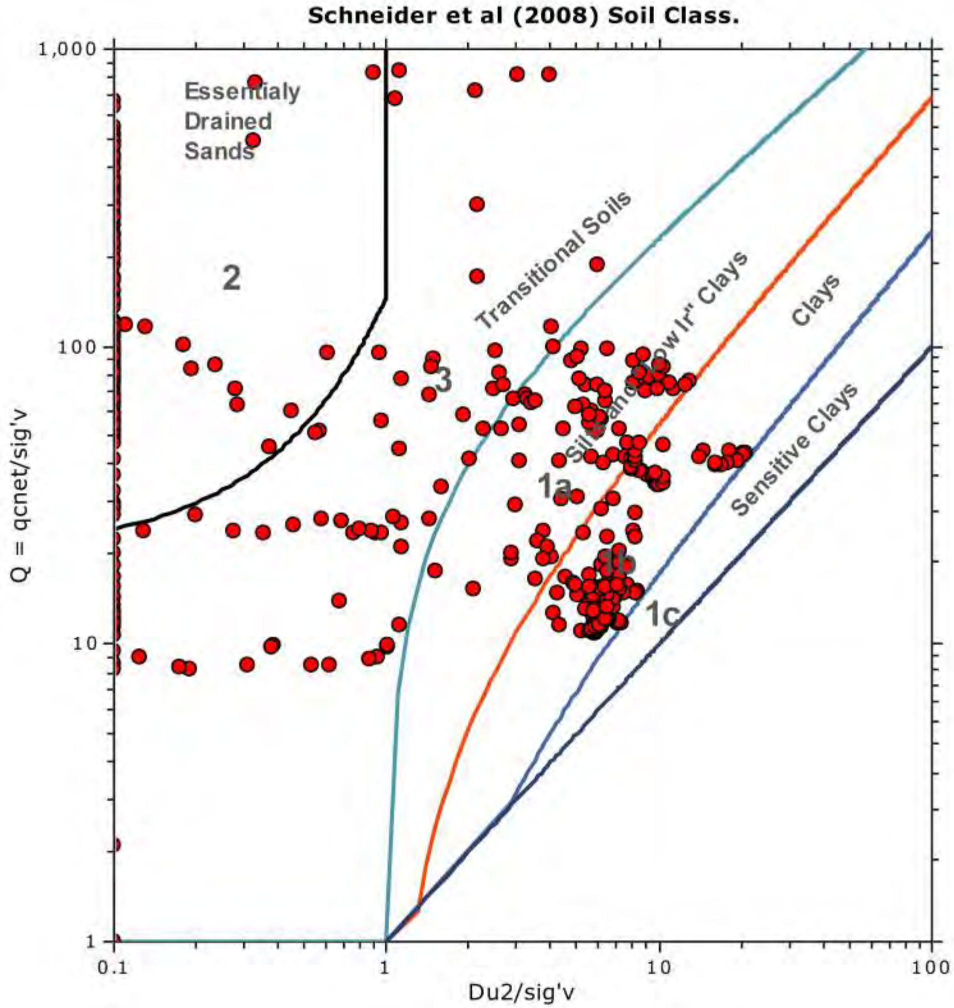


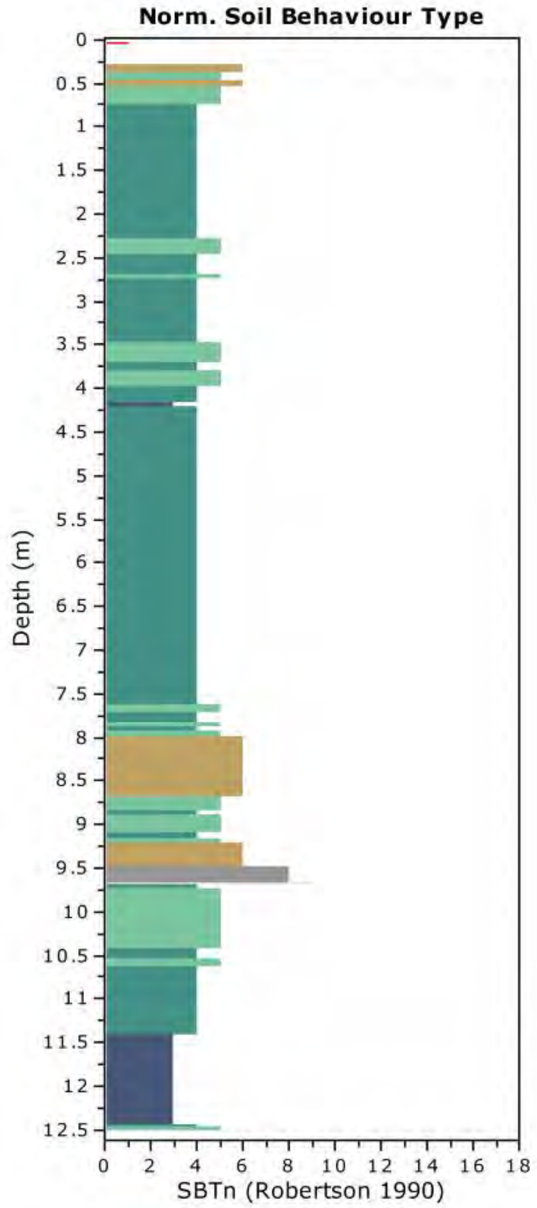
SBTn legend

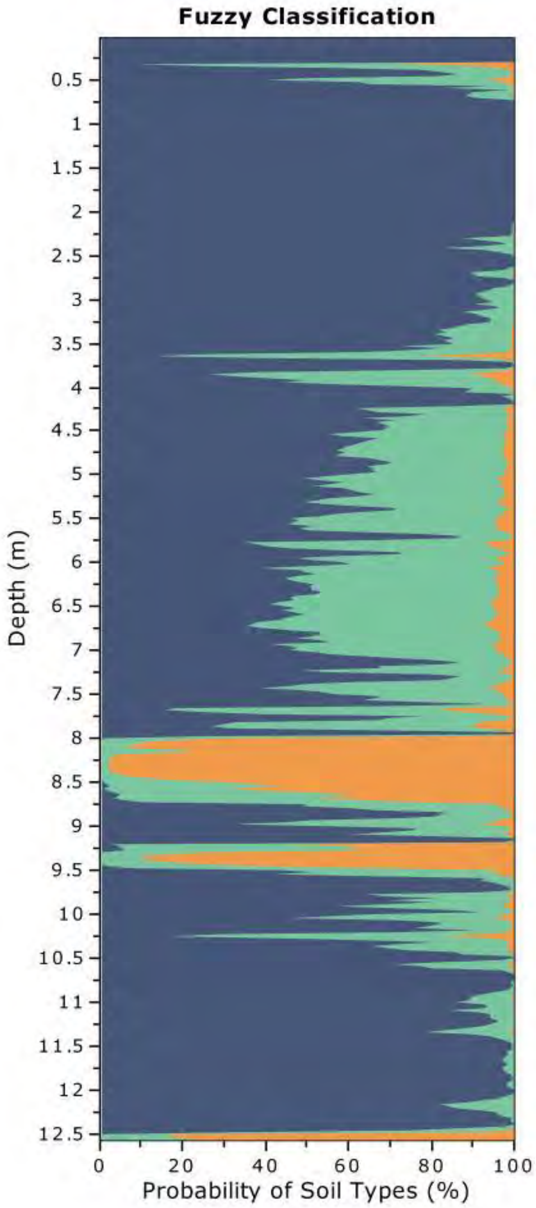
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Bq plots (Schneider)





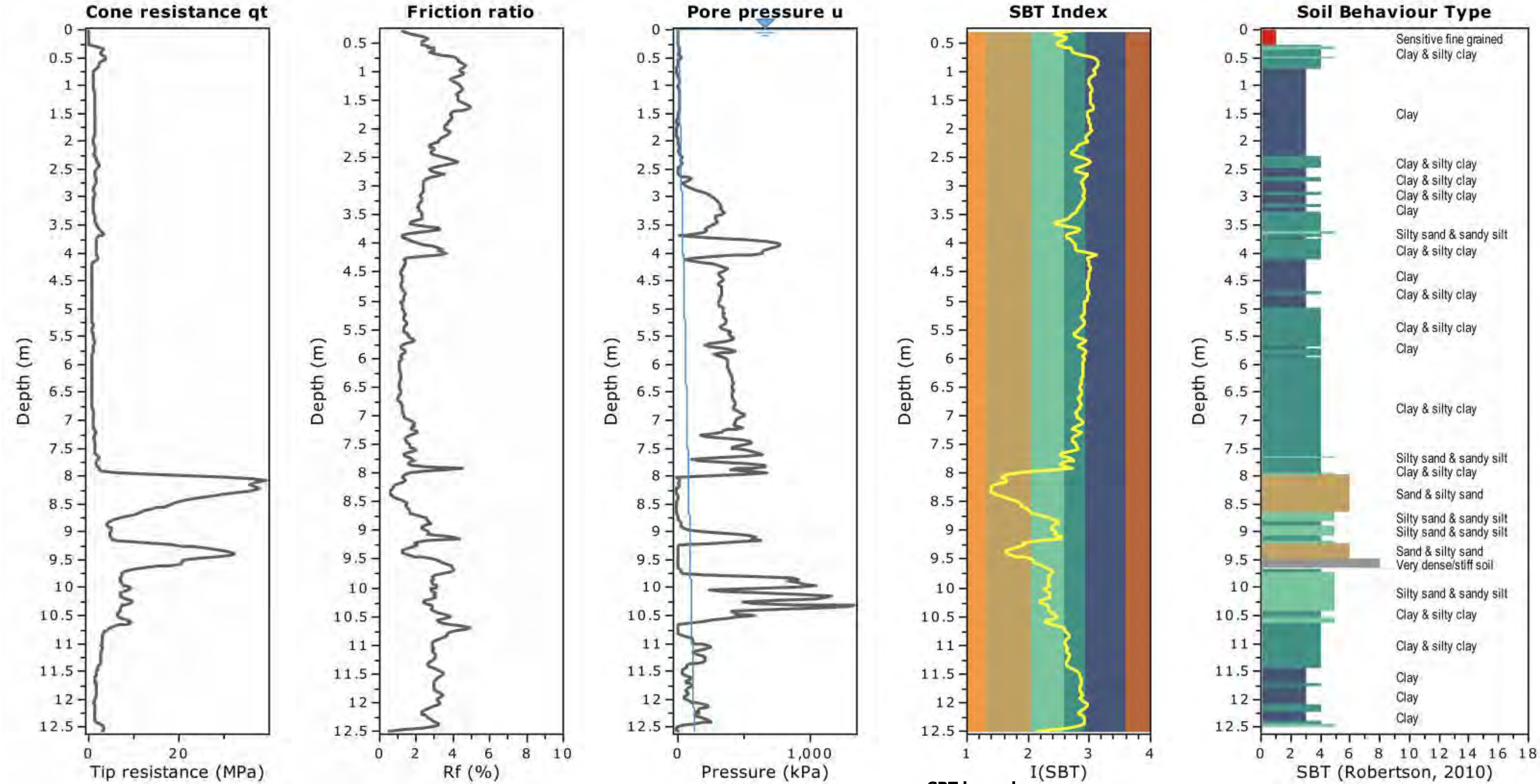

Norm. Soil Behaviour Type
Depth (m)
SBTn (Robertson 1990)


Fuzzy Classification
Depth (m)
Probability of Soil Types (%)

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

CPT: Sheet1
Total depth: 12.57 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



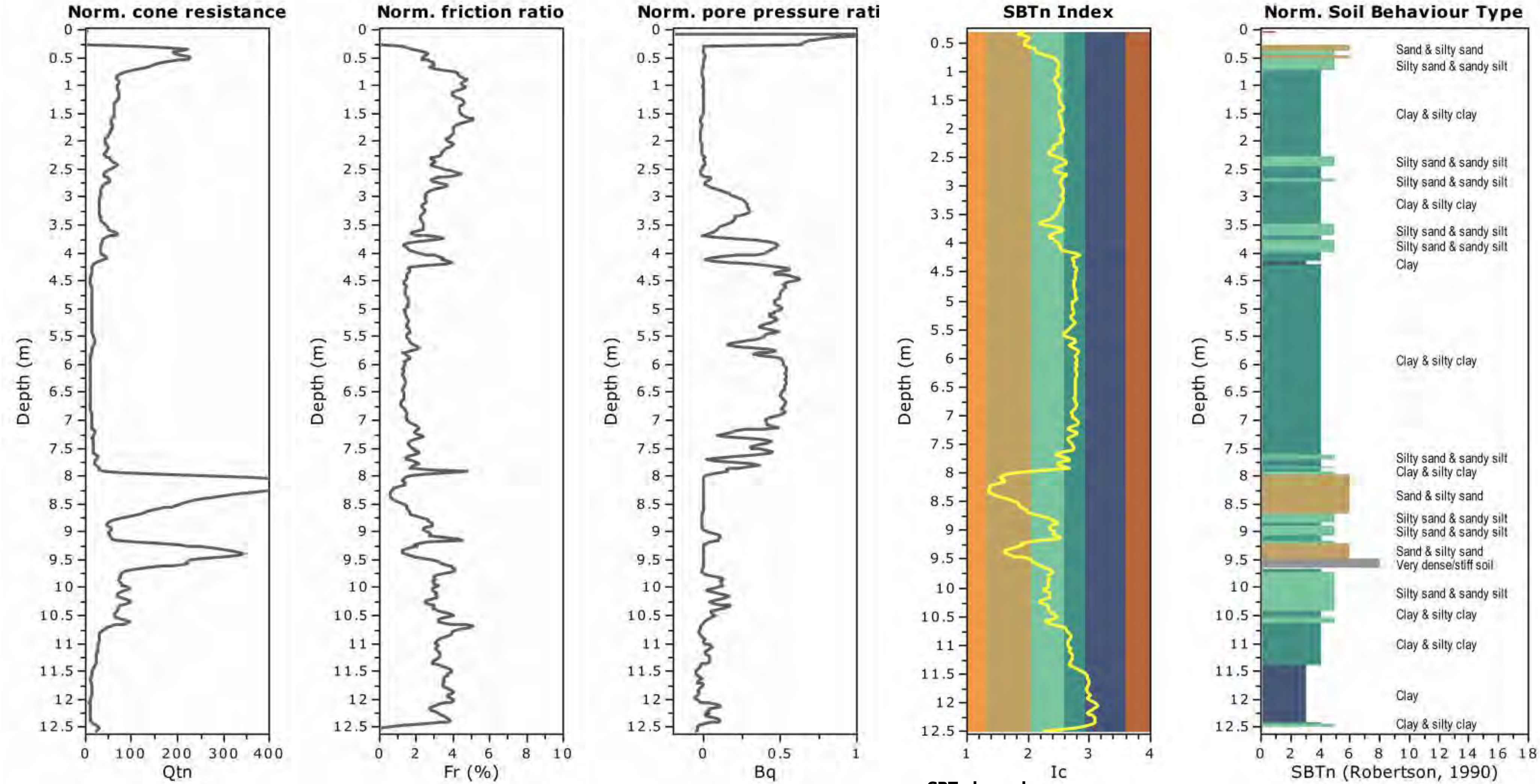
SBT legend

1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT: Sheet1

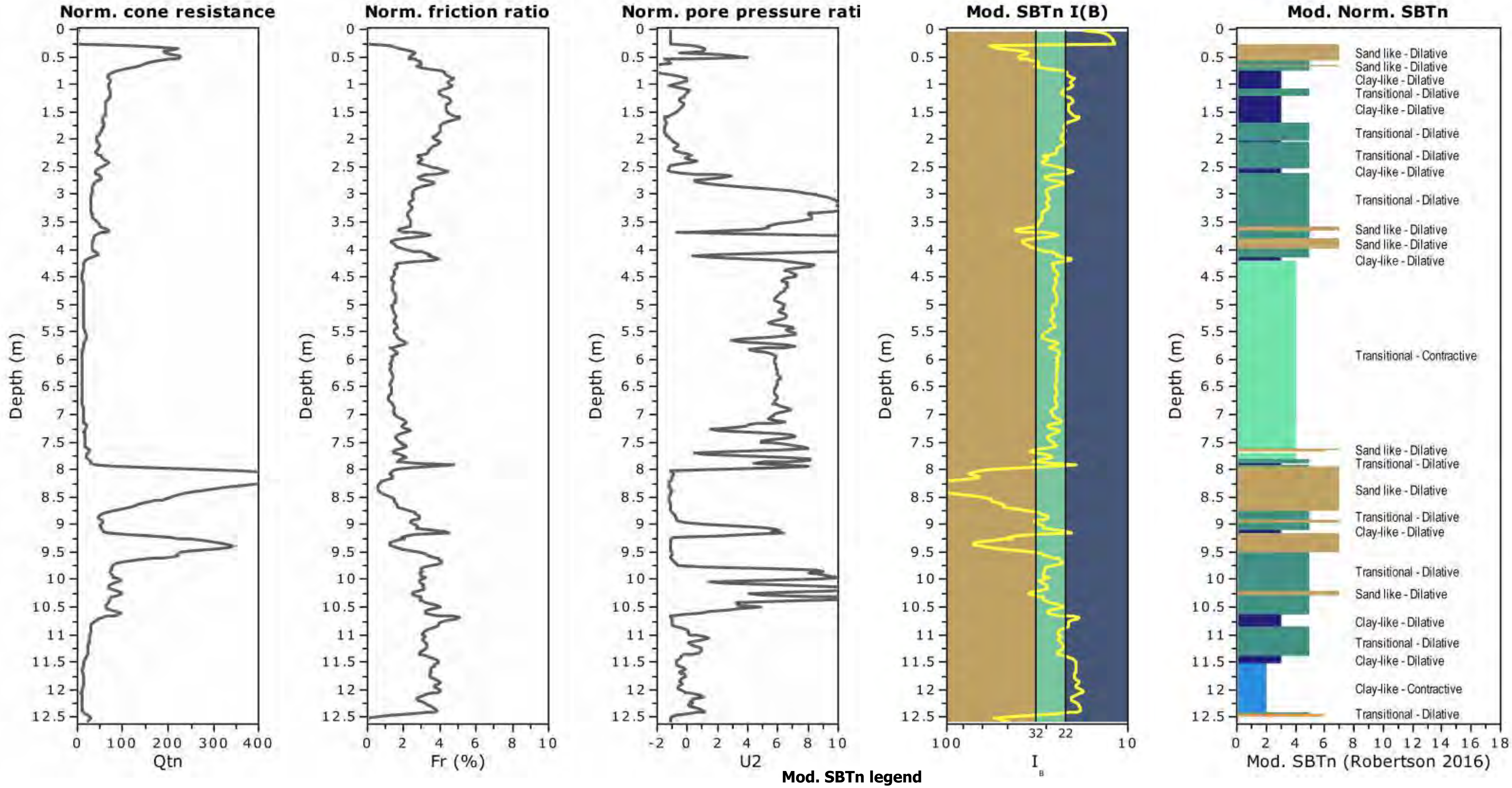
Total depth: 12.57 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



SBTn legend

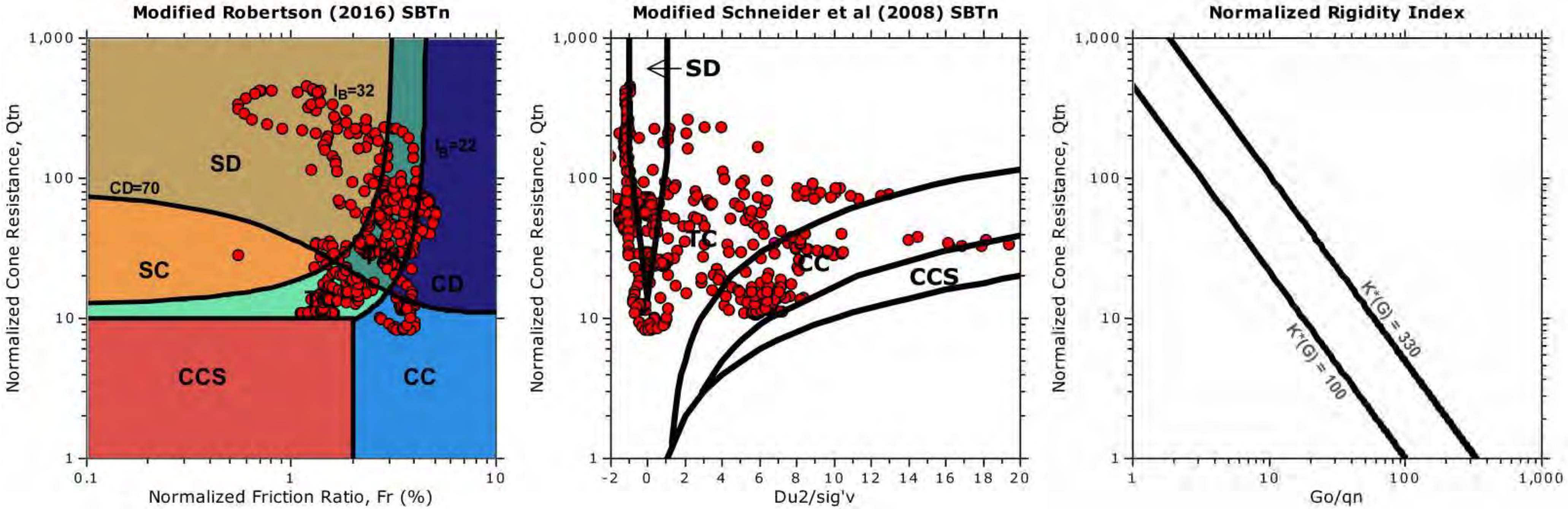
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained



Mod. SBTn legend

- 1. CCS: ClayLike - Contractive, Sensitive
- 2. CC: Clay-like - Contractive
- 3. CD: Clay-Like: Dilative
- 4. TC: Transitional - Contractive
- 5. TD: Transitional - Dilative
- 6. SC: Sand-like - Contractive
- 7. SD: Sand-like - Dilative

Updated SBTn plots



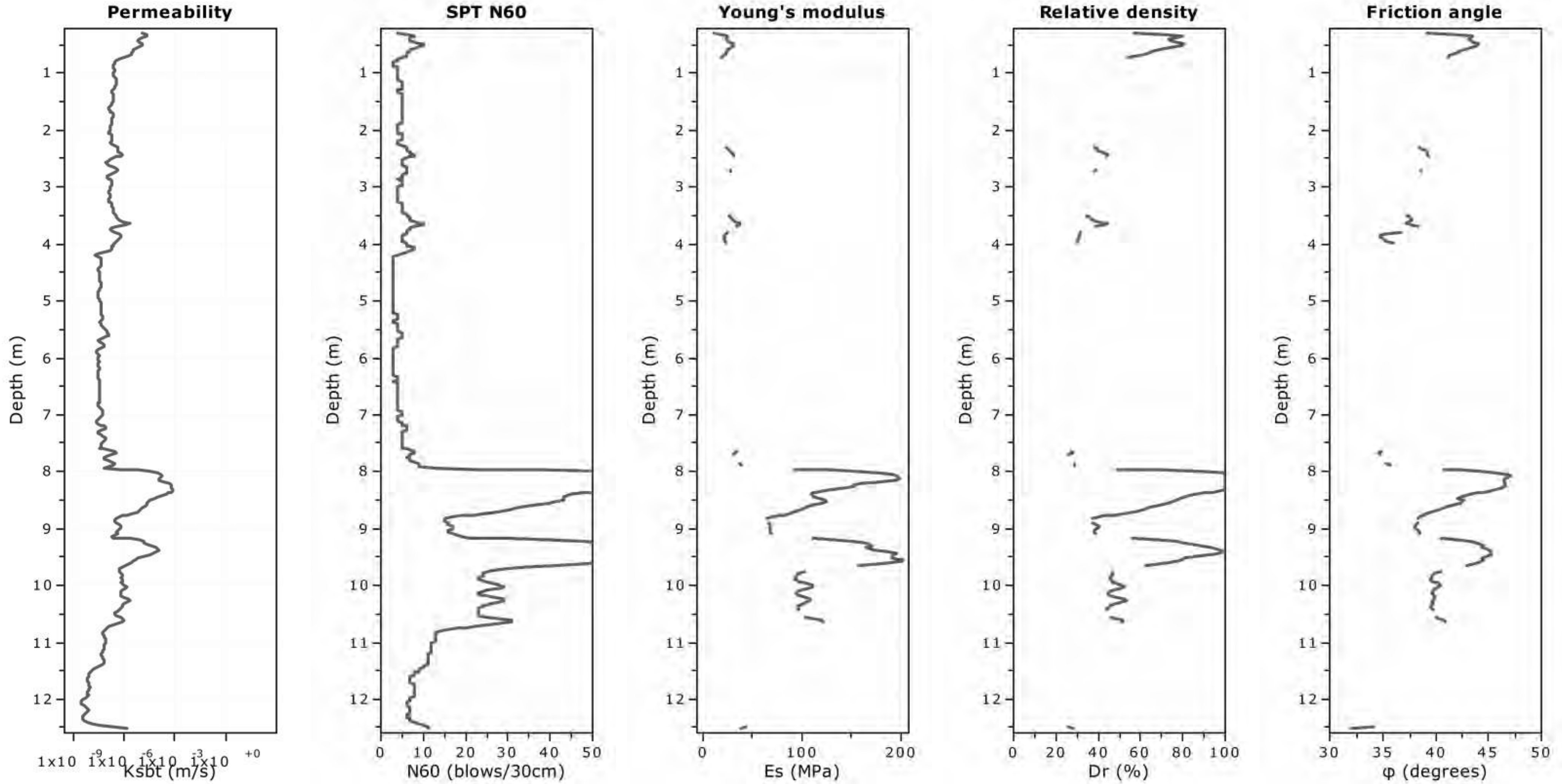
CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

CPT: Sheet1
Total depth: 12.57 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

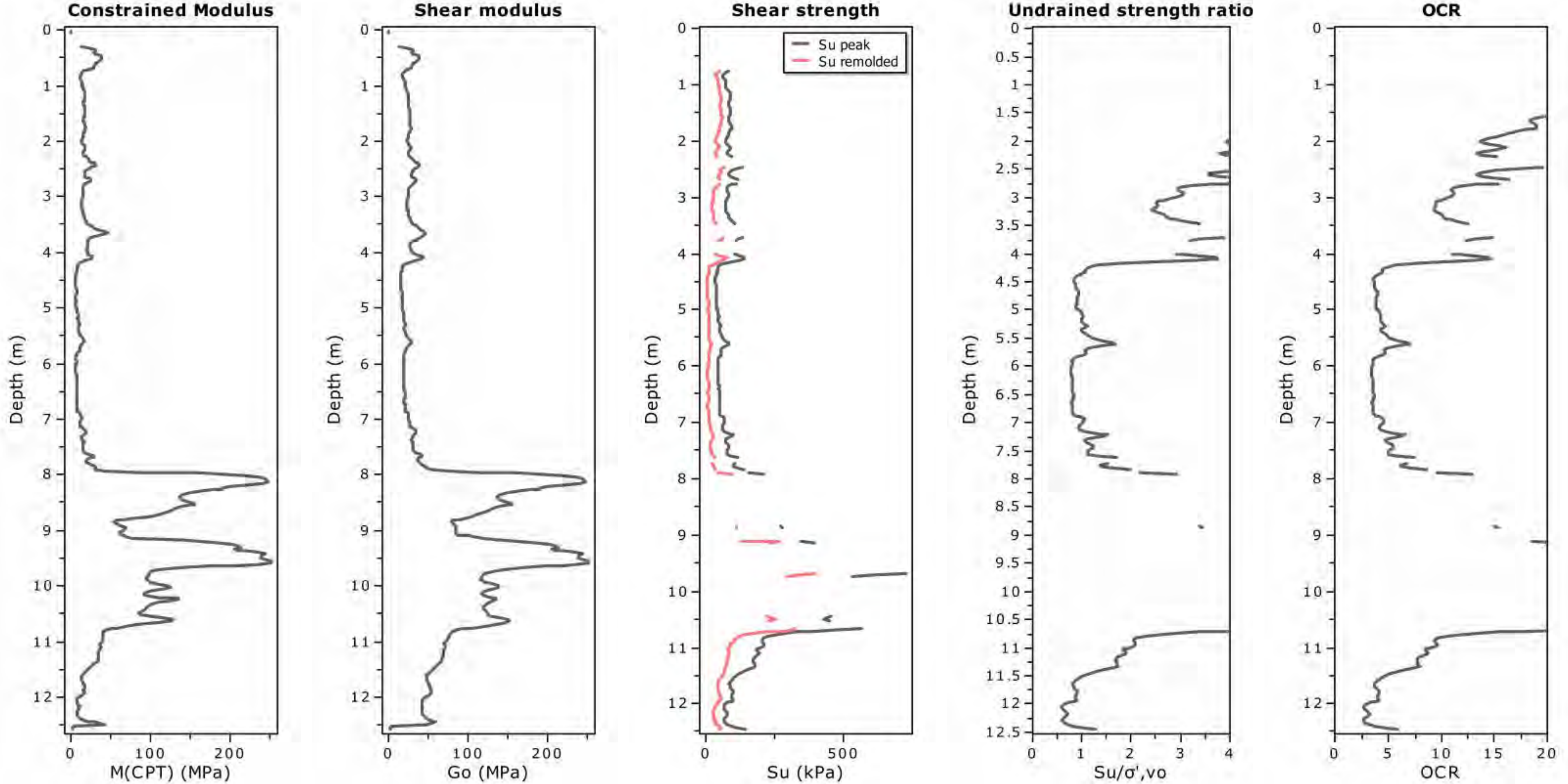
Project:
Location:



Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

CPT: Sheet1
Total depth: 12.57 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

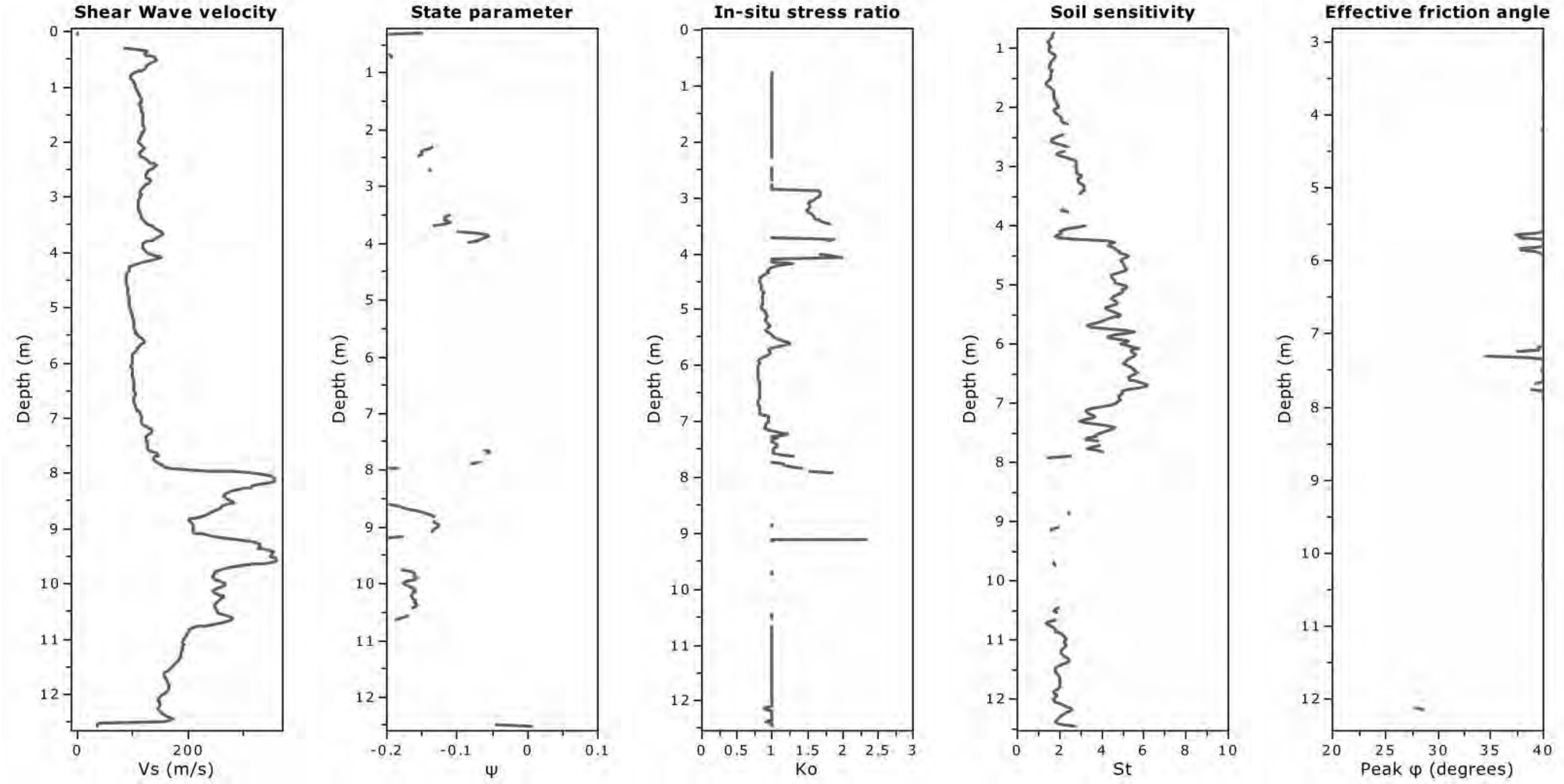
Project:
Location:



Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

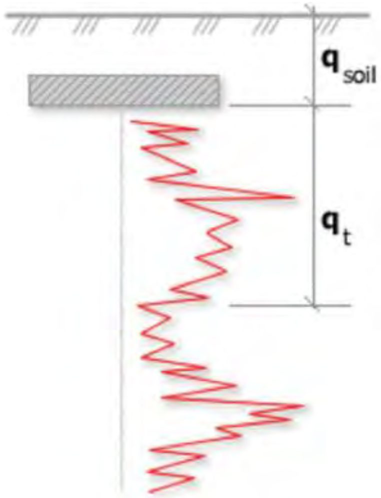
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

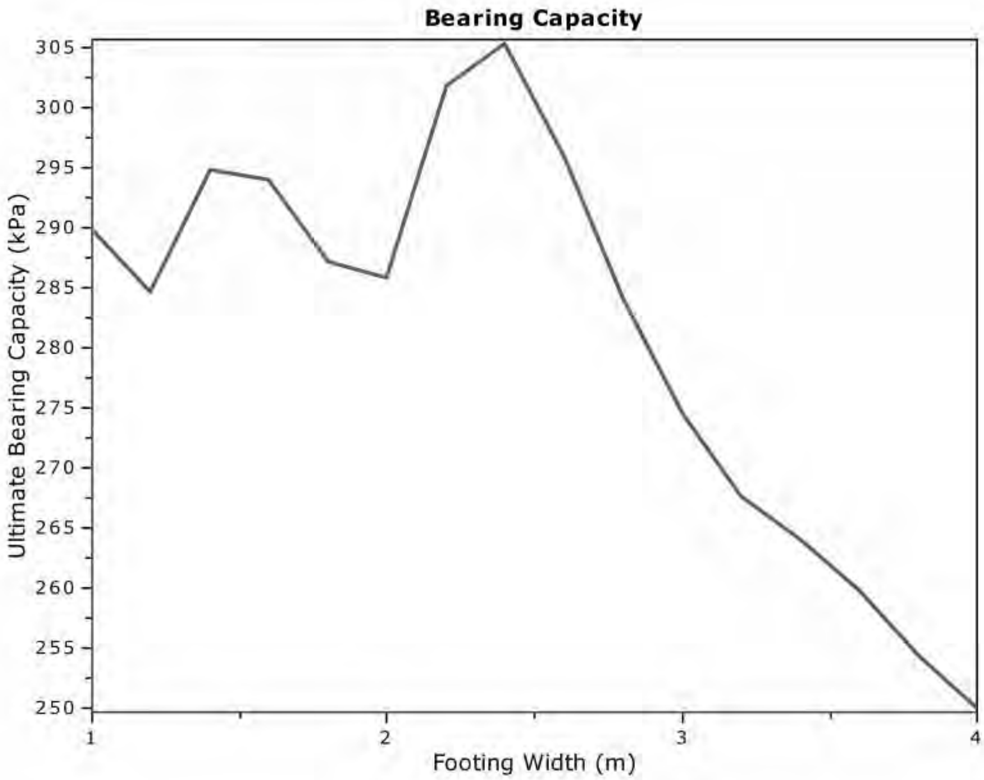
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	1.40	0.20	9.50	289.89
2	1.20	0.50	2.30	1.38	0.20	9.50	284.82
3	1.40	0.50	2.60	1.43	0.20	9.50	294.99
4	1.60	0.50	2.90	1.42	0.20	9.50	294.00
5	1.80	0.50	3.20	1.39	0.20	9.50	287.30
6	2.00	0.50	3.50	1.38	0.20	9.50	285.92
7	2.20	0.50	3.80	1.46	0.20	9.50	301.91
8	2.40	0.50	4.10	1.48	0.20	9.50	305.41
9	2.60	0.50	4.40	1.43	0.20	9.50	295.95
10	2.80	0.50	4.70	1.37	0.20	9.50	284.16
11	3.00	0.50	5.00	1.33	0.20	9.50	274.64
12	3.20	0.50	5.30	1.29	0.20	9.50	267.79
13	3.40	0.50	5.60	1.27	0.20	9.50	264.04
14	3.60	0.50	5.90	1.25	0.20	9.50	259.91
15	3.80	0.50	6.20	1.22	0.20	9.50	254.45
16	4.00	0.50	6.50	1.20	0.20	9.50	250.05

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

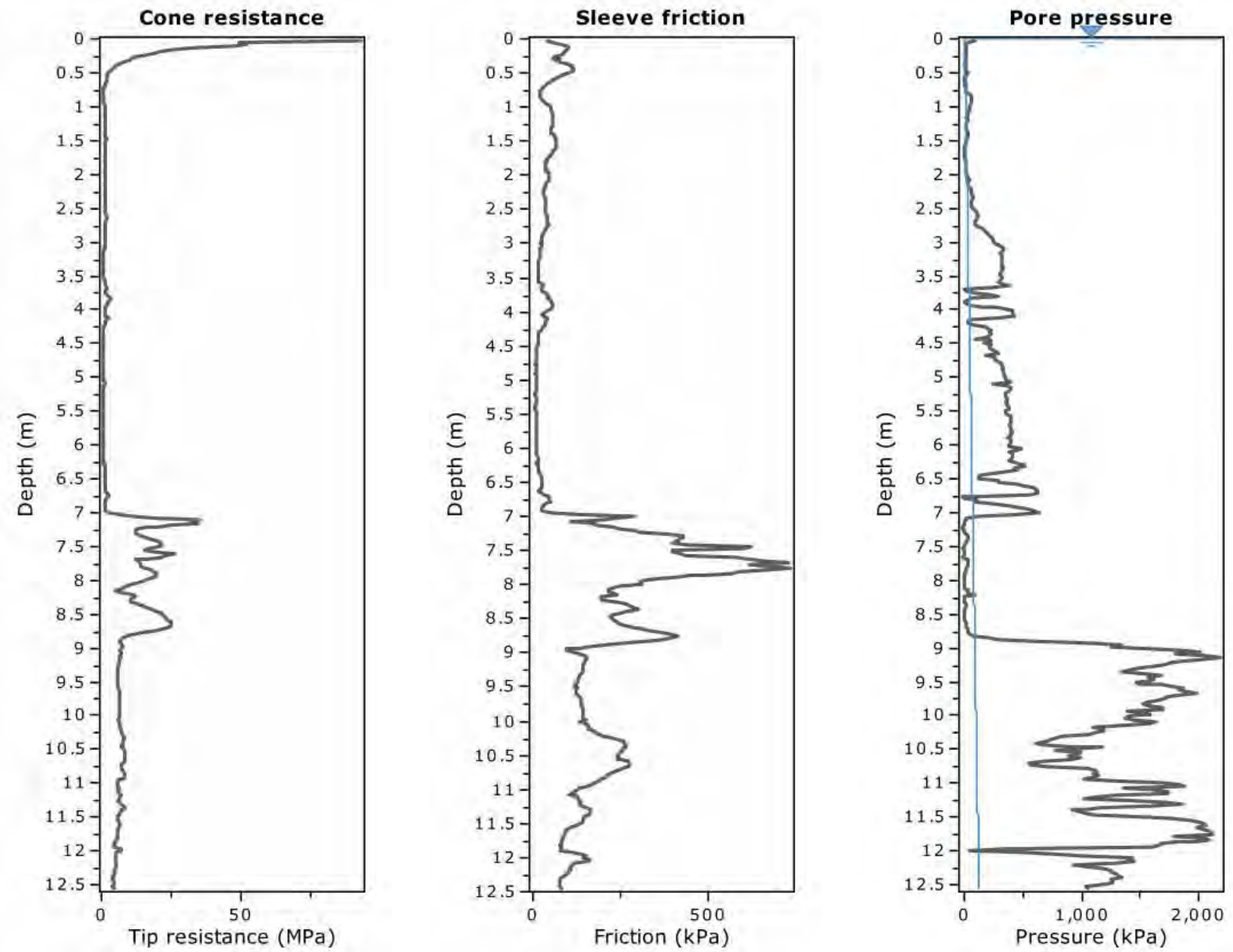
References

• Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012

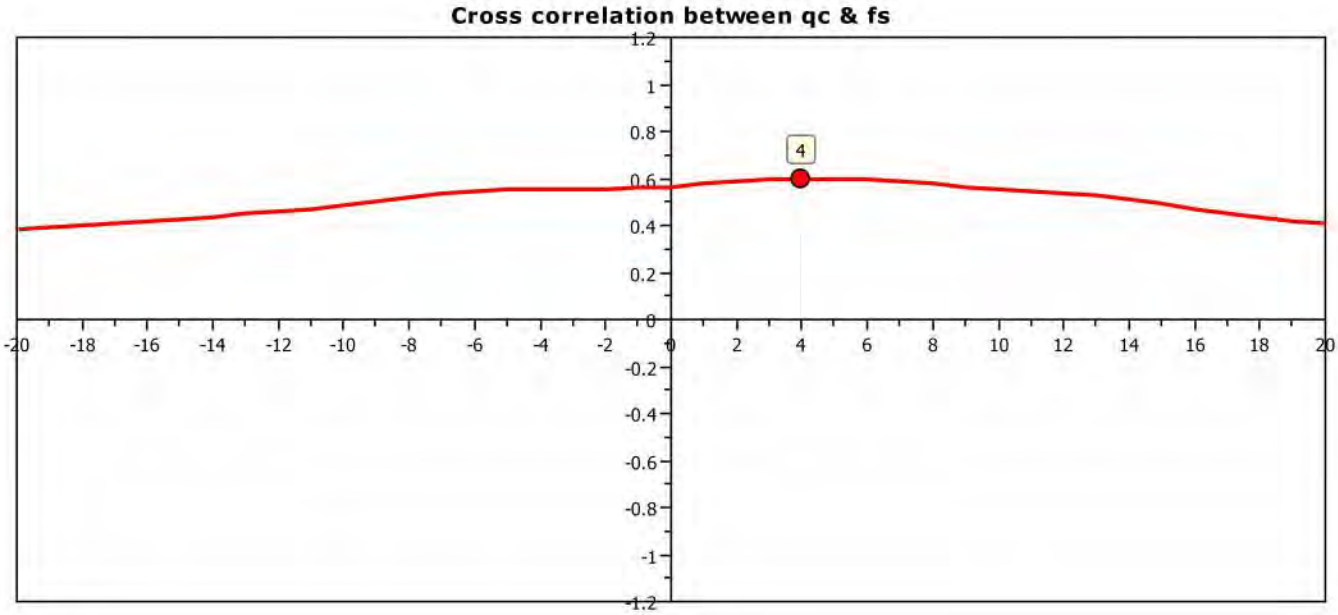
• Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

• N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

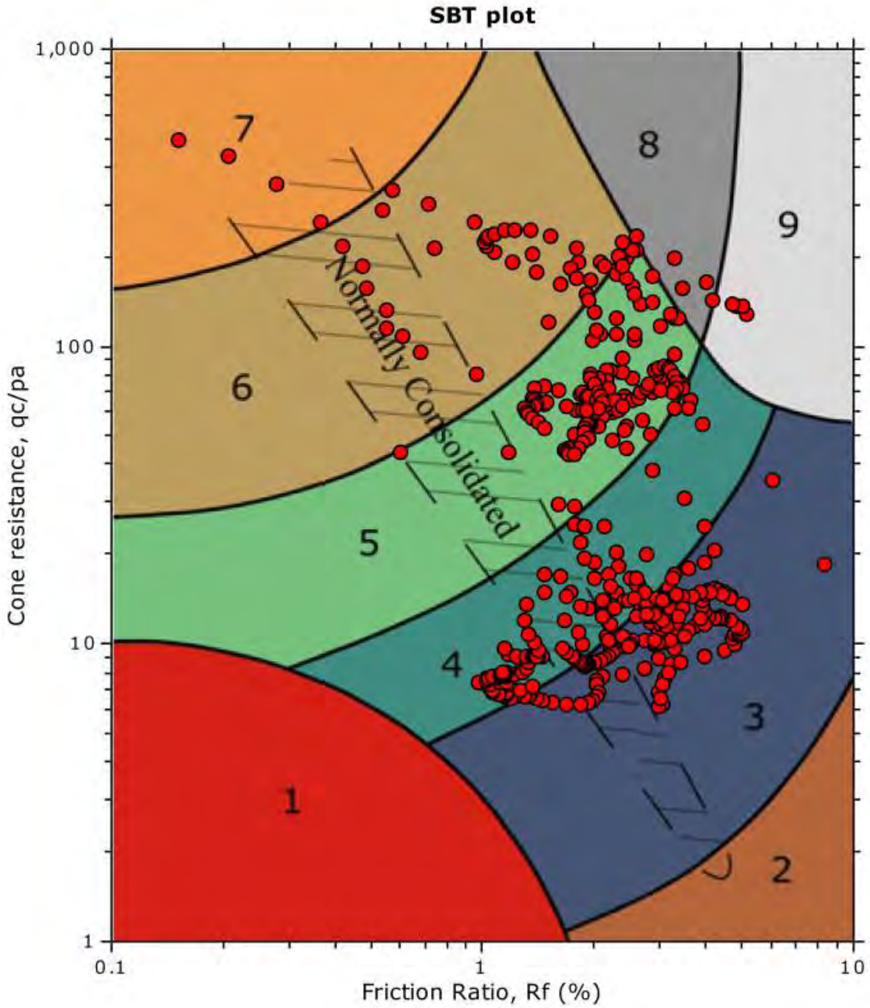
Project:
Location:

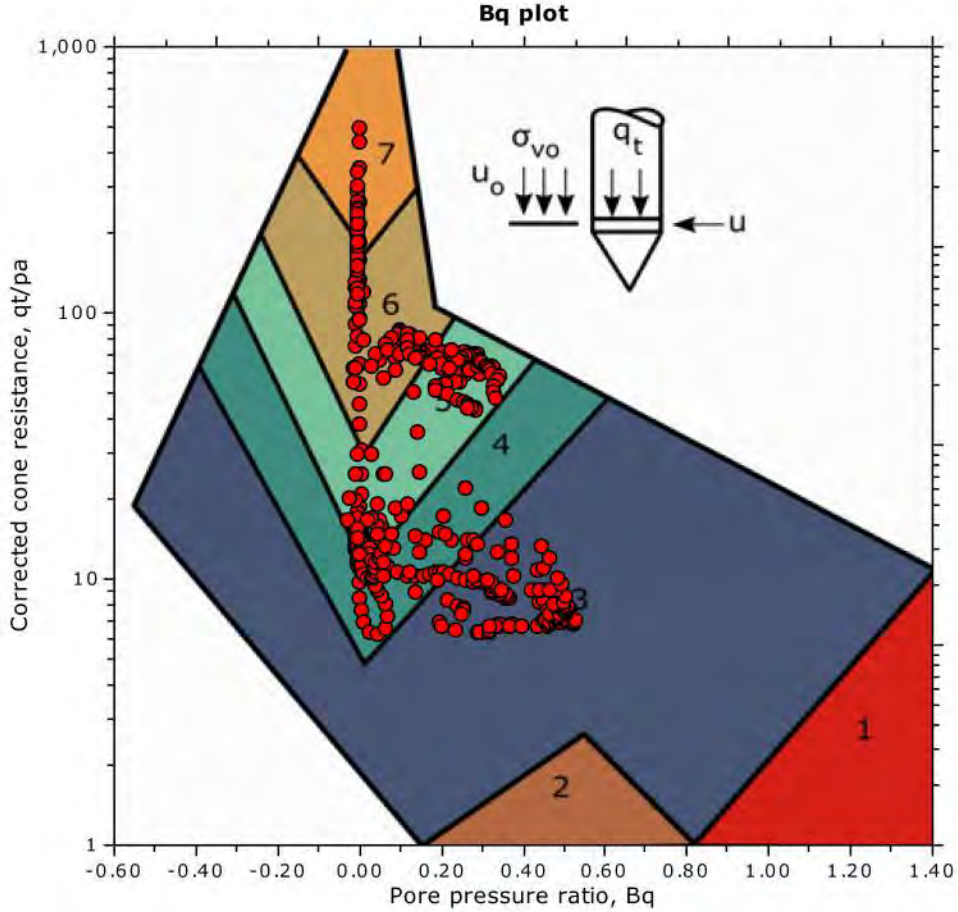


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots

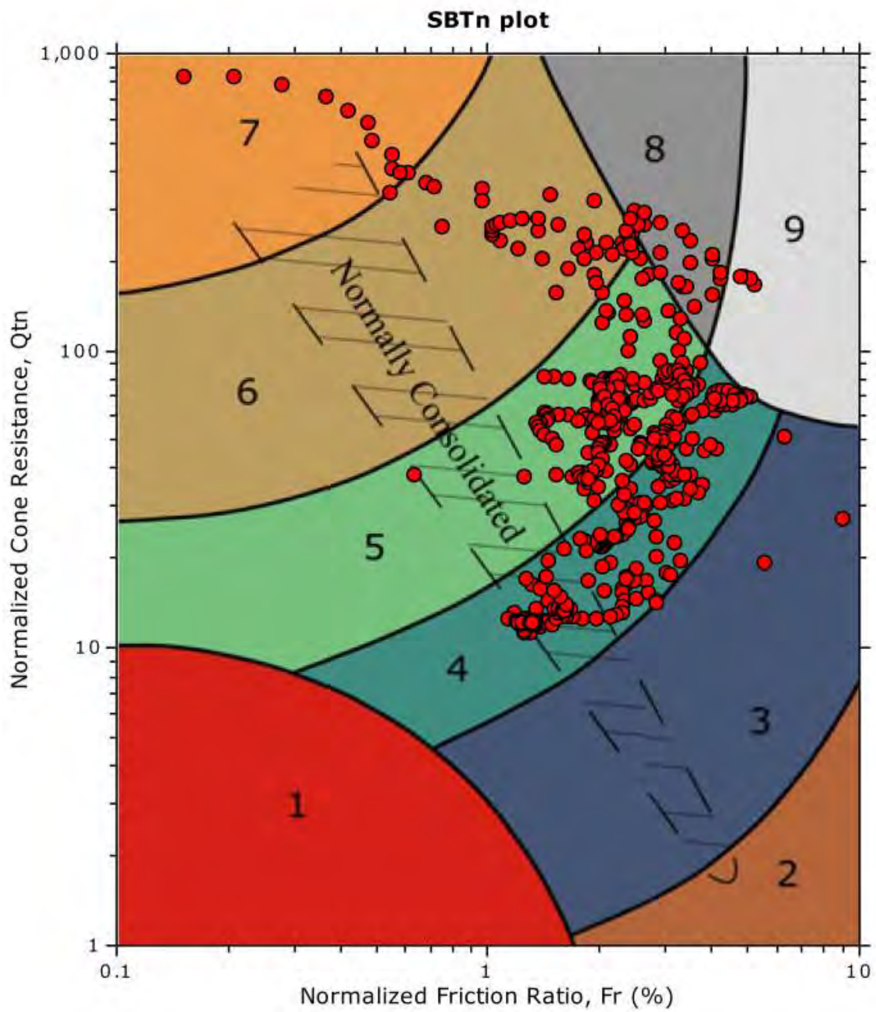


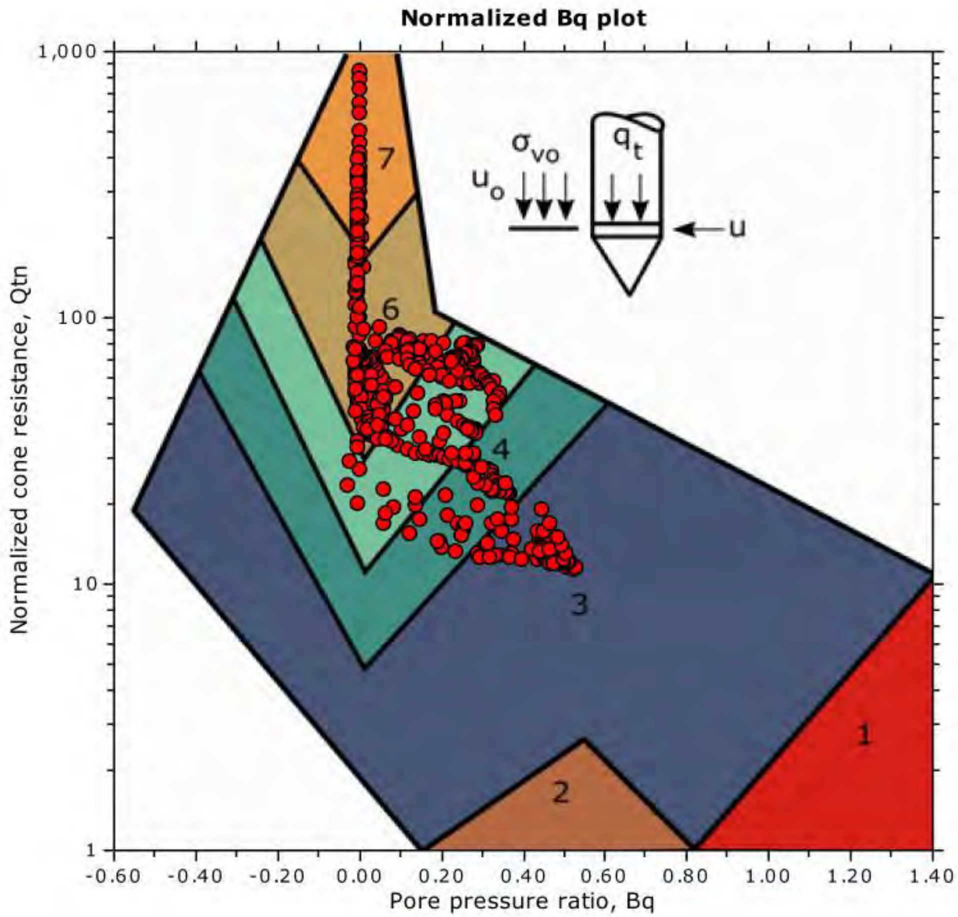


SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

SBT - Bq plots (normalized)

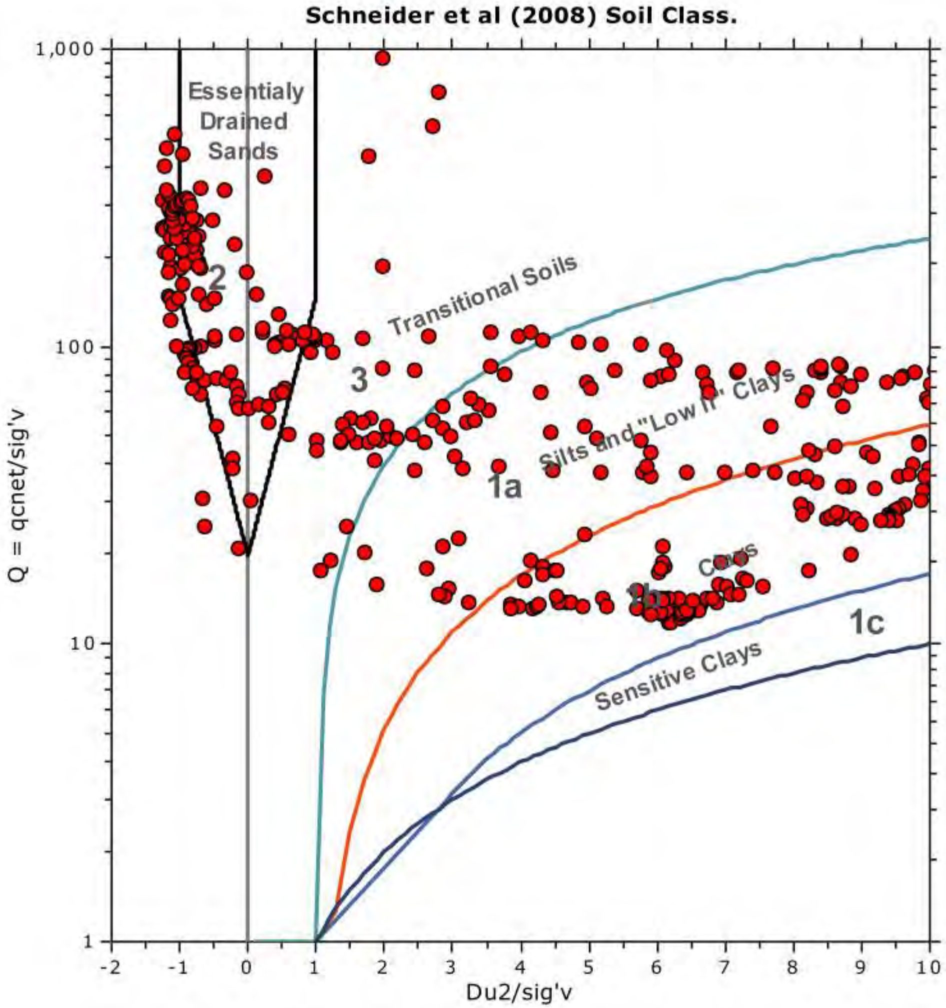


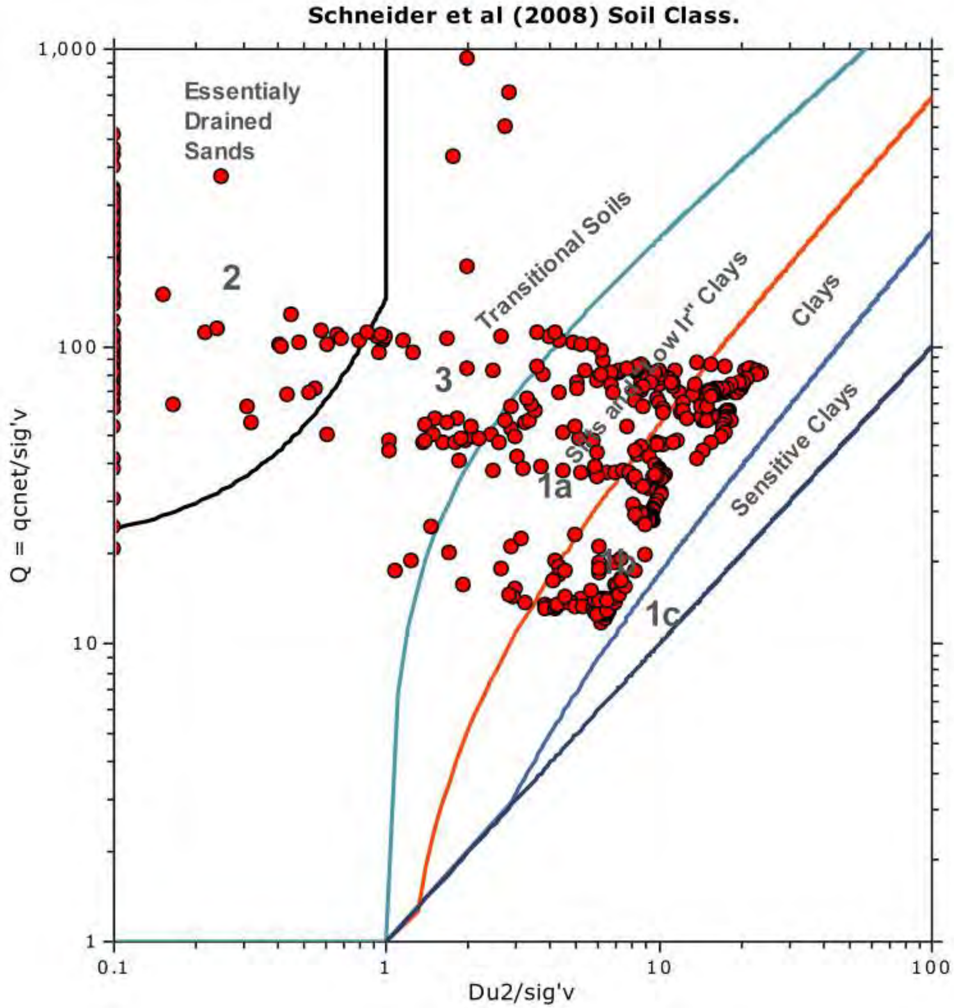


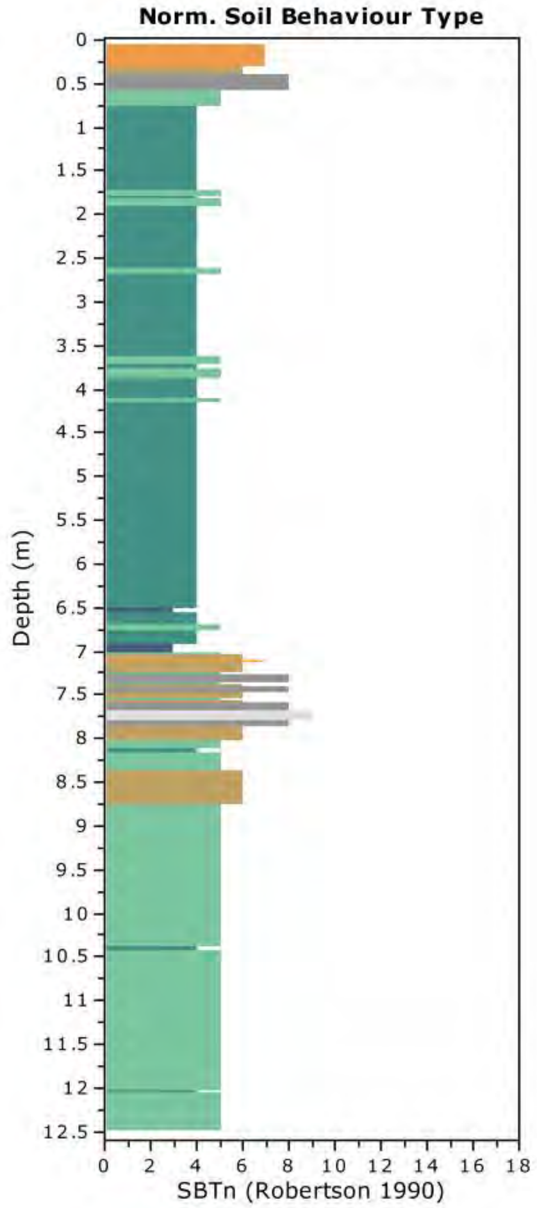
SBTn legend

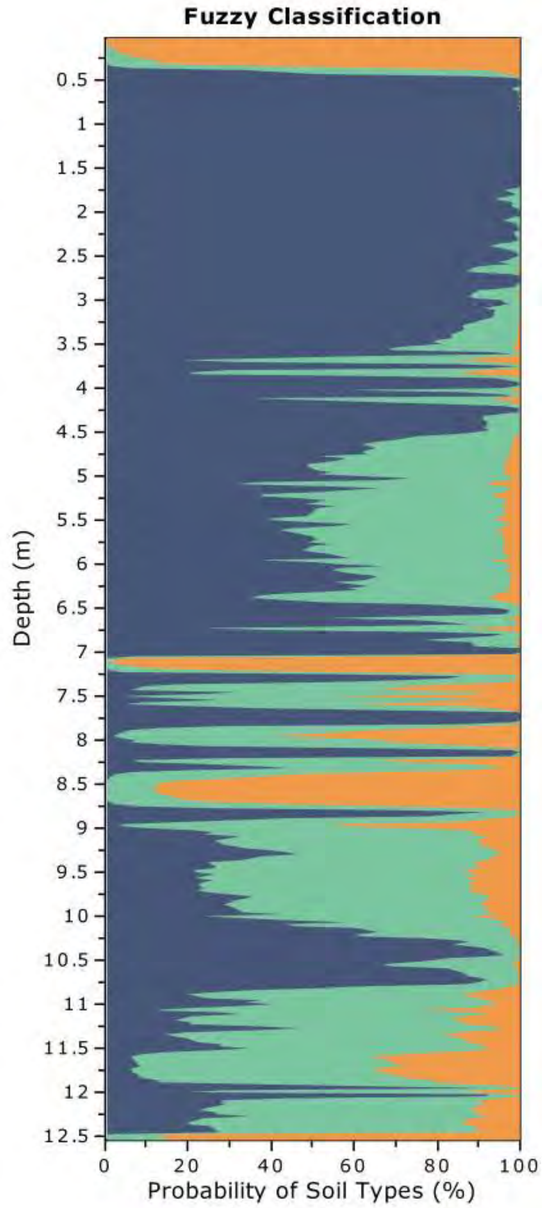
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Bq plots (Schneider)

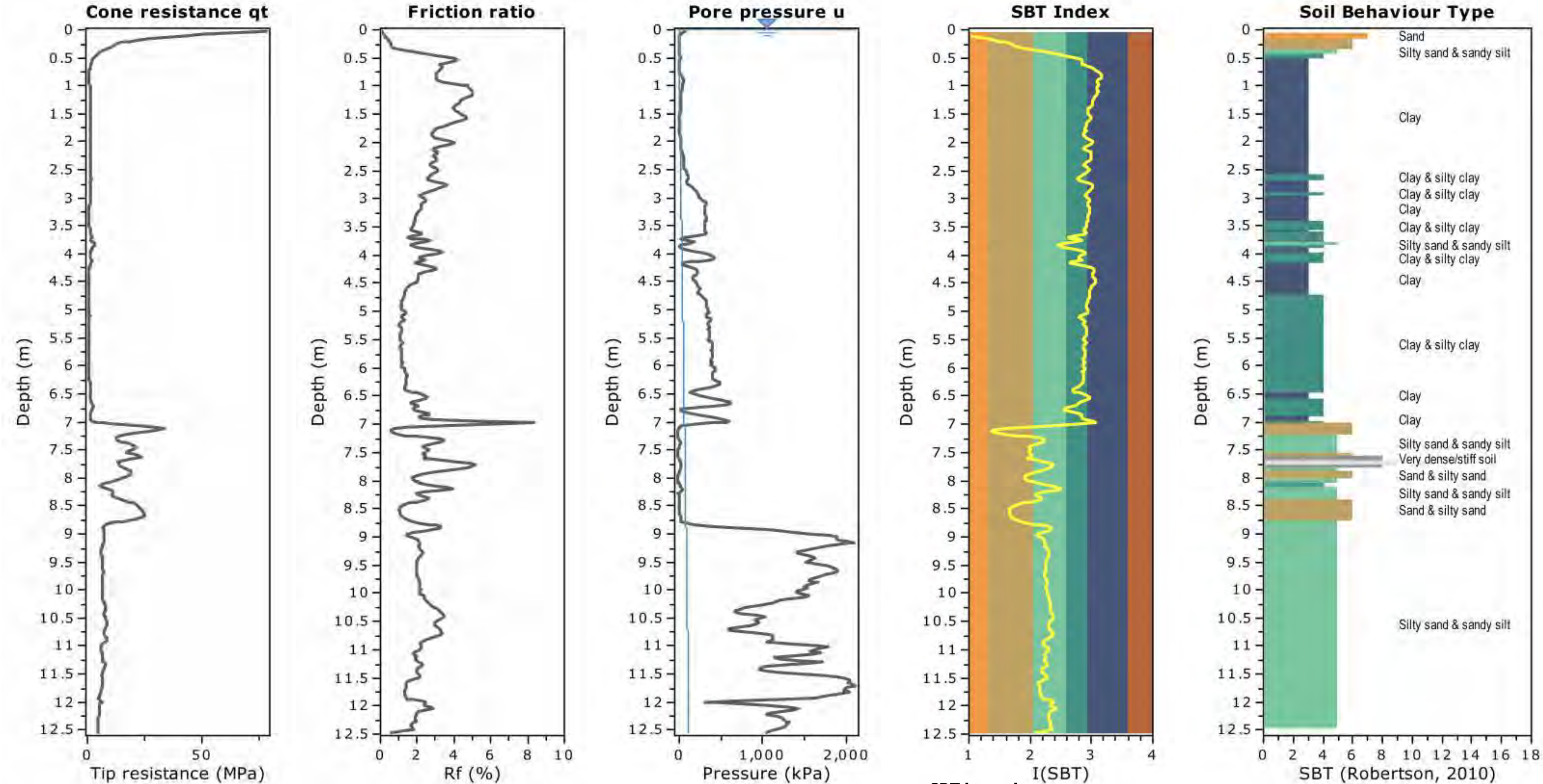





Norm. Soil Behaviour Type
Depth (m)
SBTn (Robertson 1990)


Fuzzy Classification
Depth (m)
Probability of Soil Types (%)

Project:
Location:



SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

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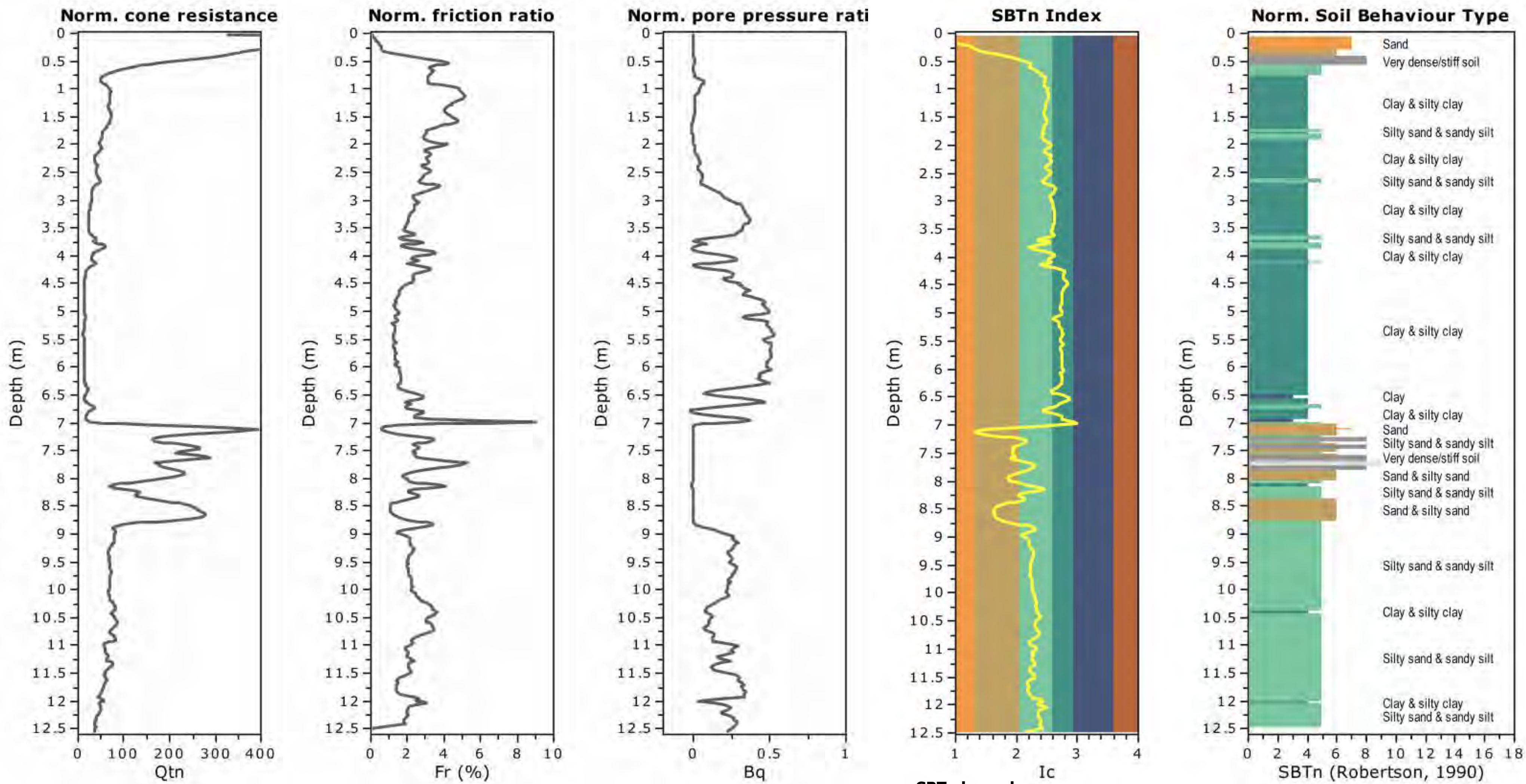
CPT: Sheet1
Total depth: 12.55 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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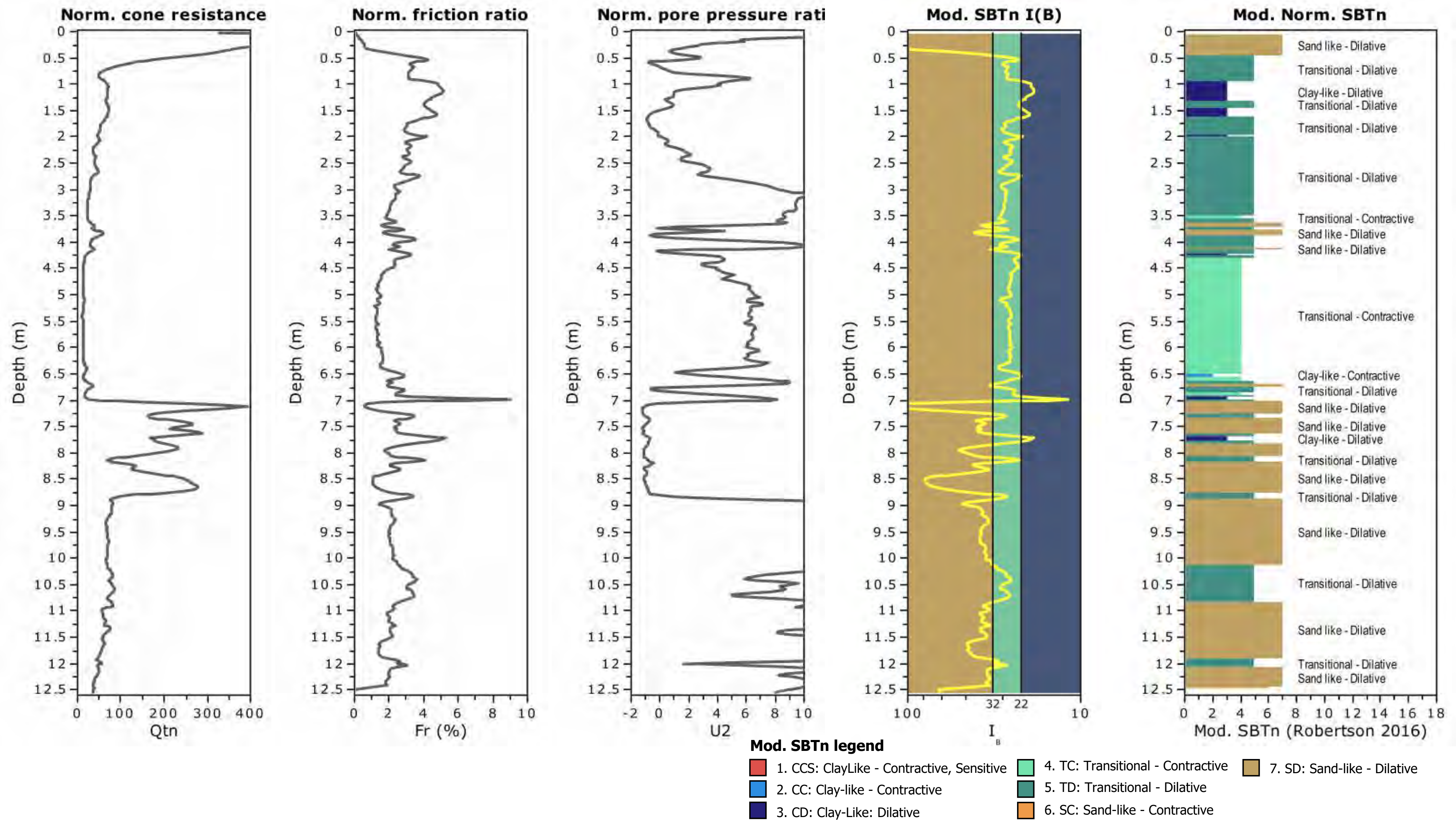
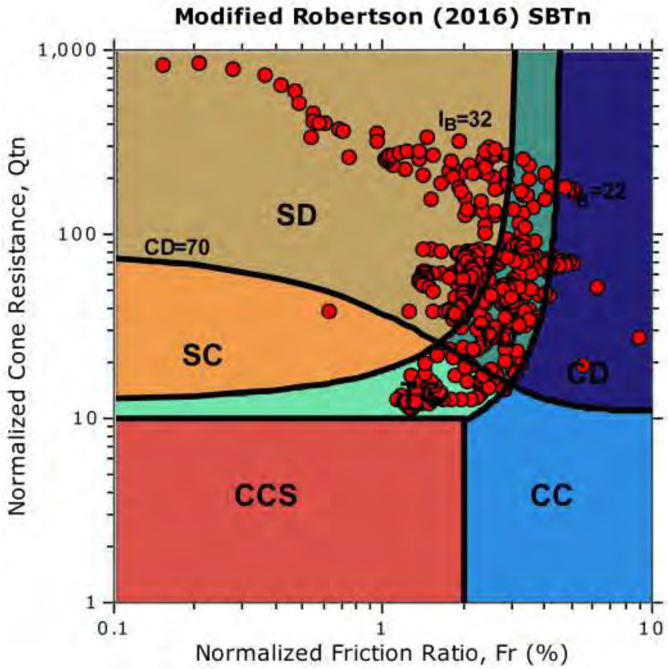
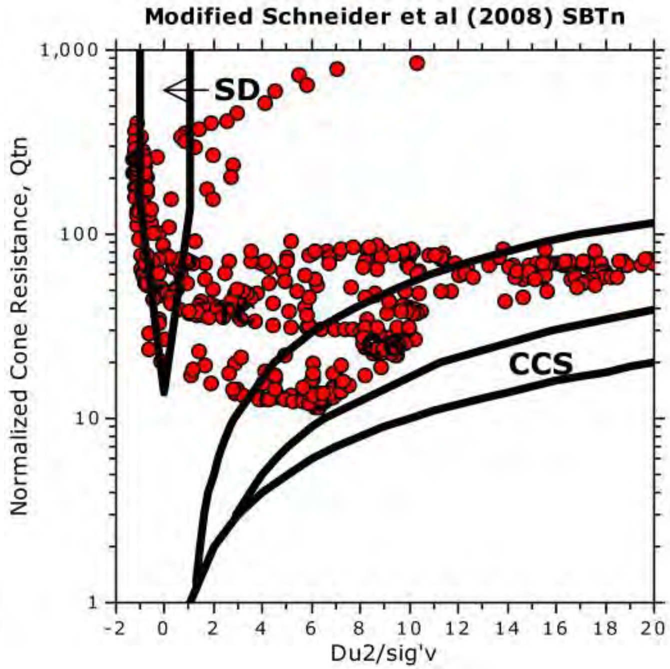
CPT: Sheet1
Total depth: 12.55 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

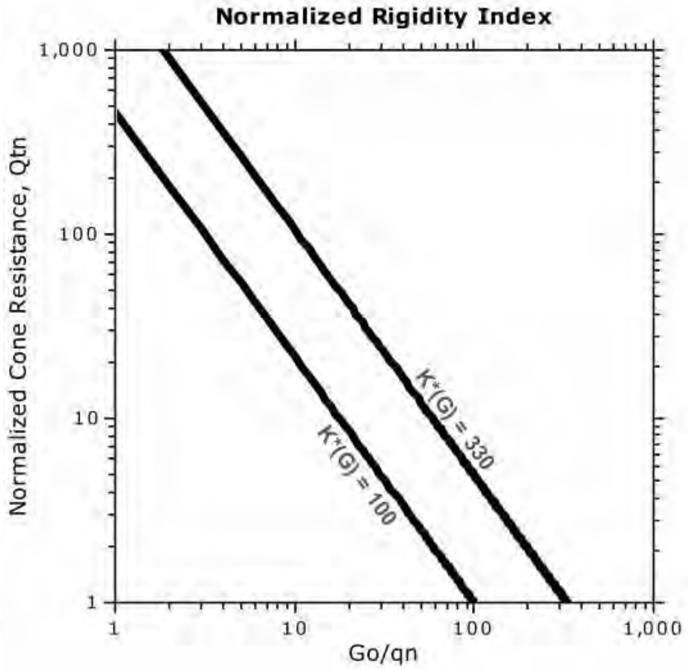
Project:
Location:



Updated SBTn plots

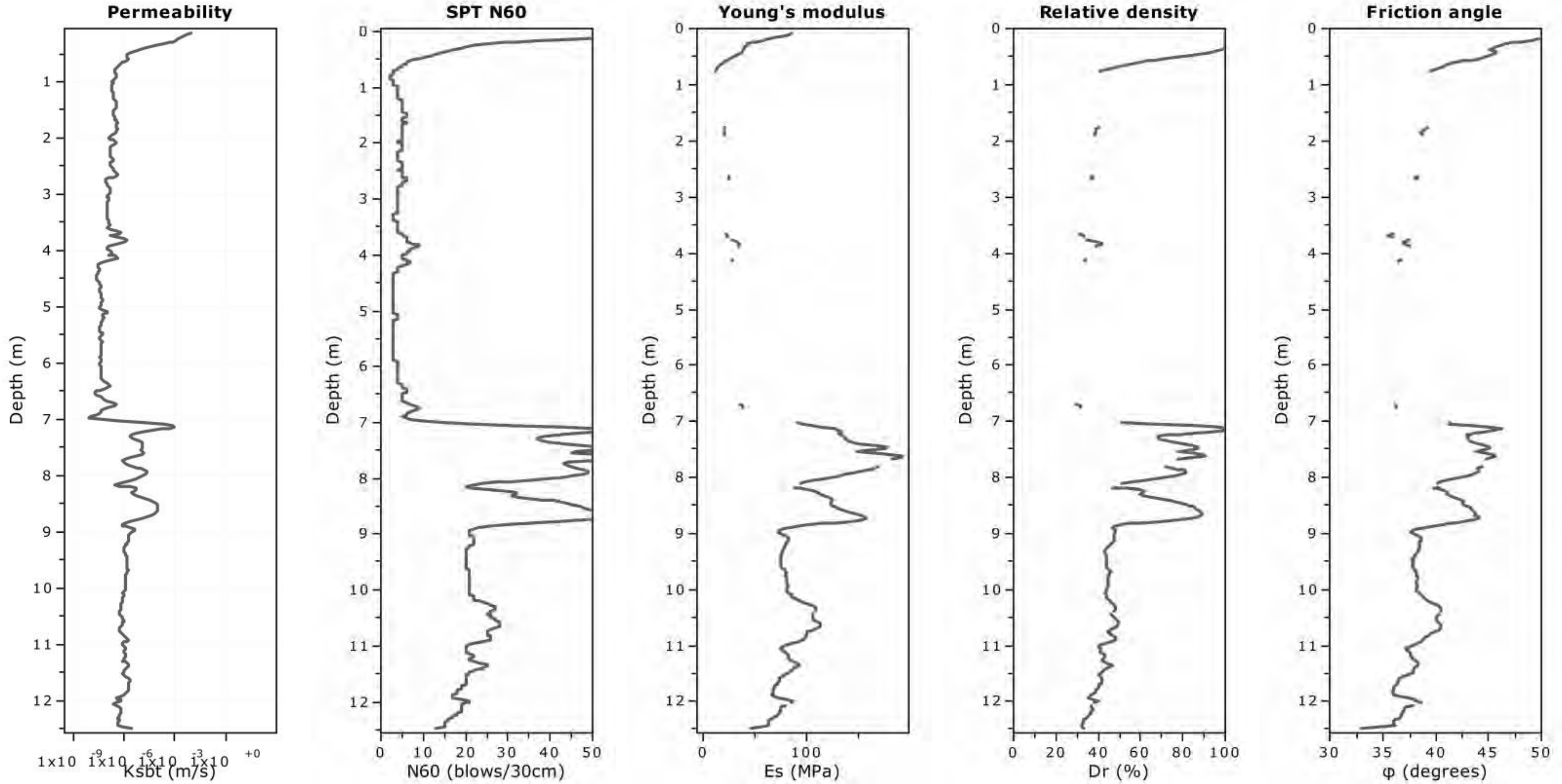






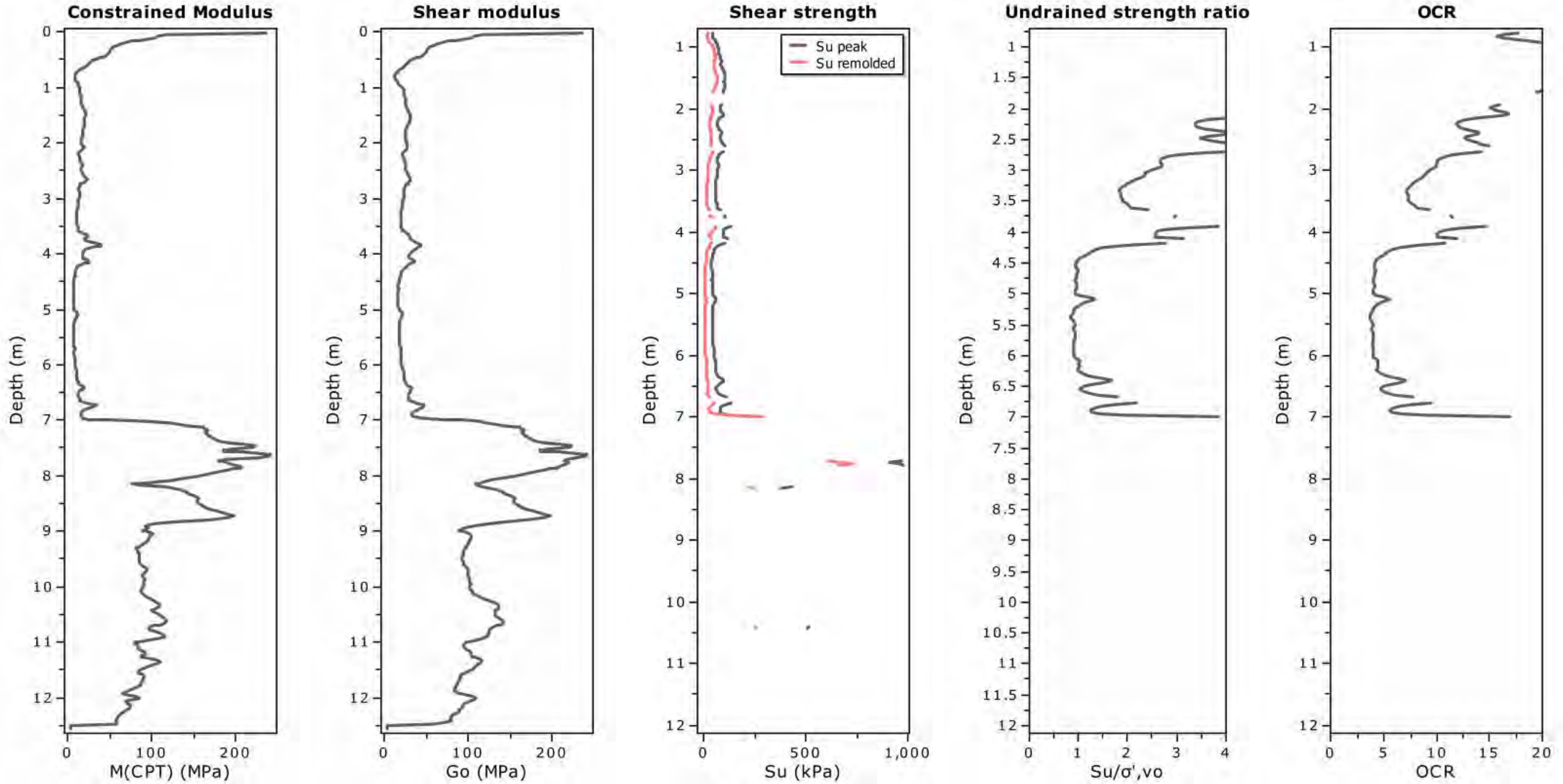
CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)



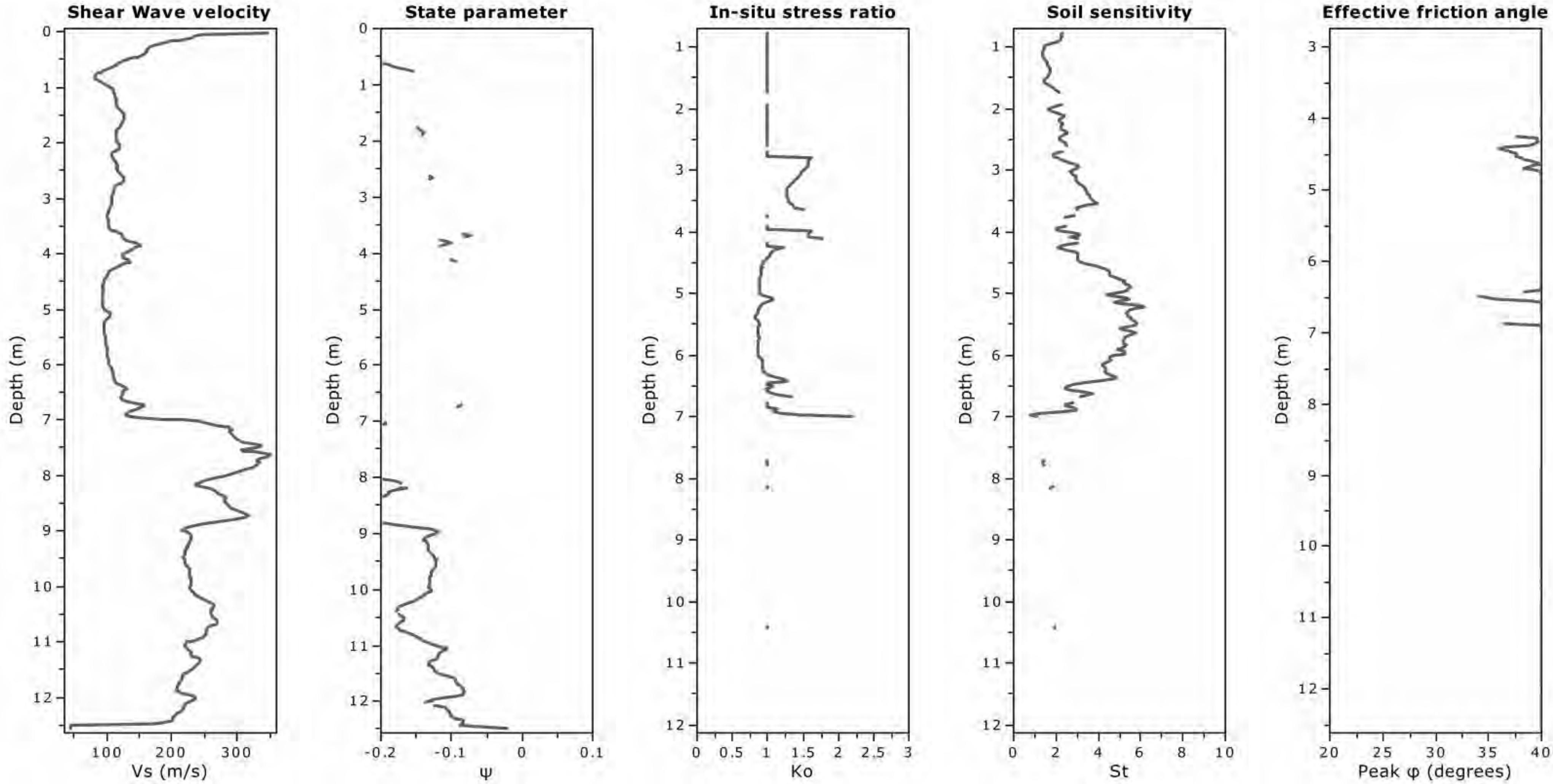
Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data



Calculation parameters

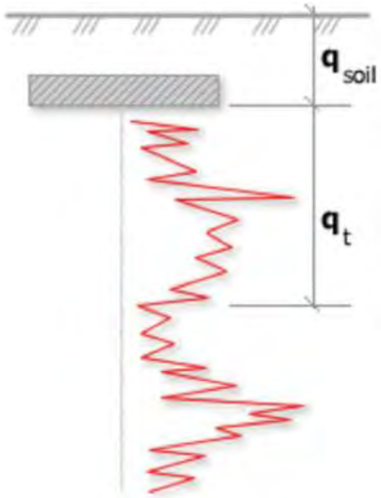
Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

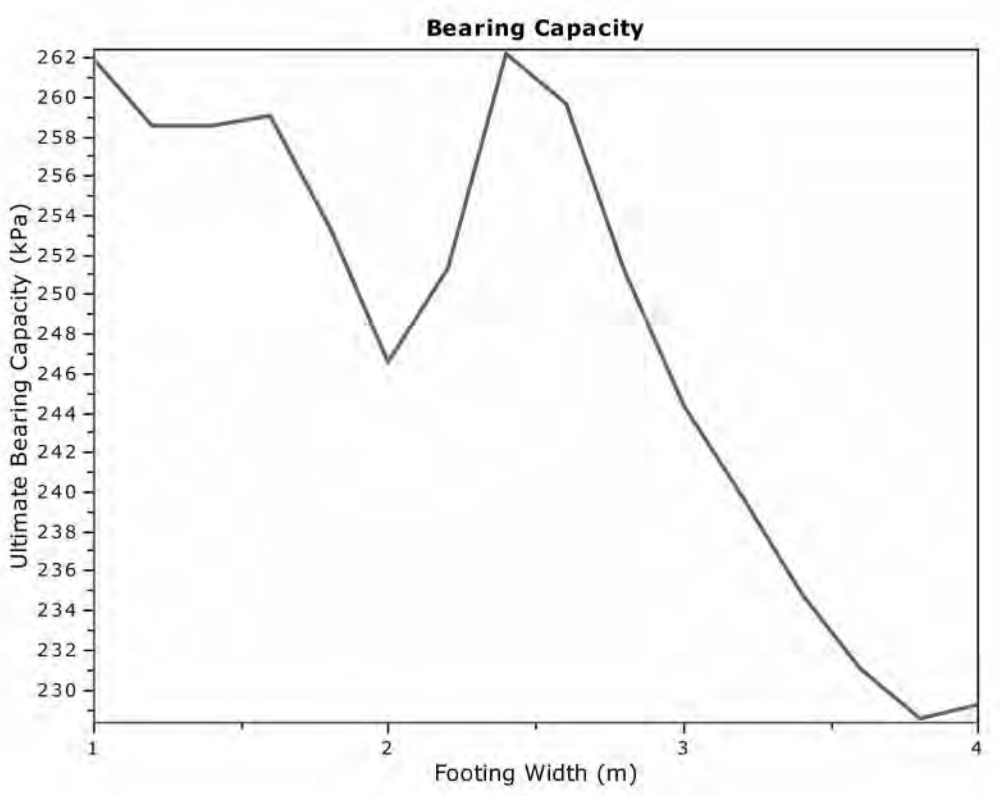
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	1.26	0.20	9.50	261.92
2	1.20	0.50	2.30	1.25	0.20	9.50	258.60
3	1.40	0.50	2.60	1.25	0.20	9.50	258.58
4	1.60	0.50	2.90	1.25	0.20	9.50	259.03
5	1.80	0.50	3.20	1.22	0.20	9.50	253.35
6	2.00	0.50	3.50	1.19	0.20	9.50	246.59
7	2.20	0.50	3.80	1.21	0.20	9.50	251.38
8	2.40	0.50	4.10	1.26	0.20	9.50	262.21
9	2.60	0.50	4.40	1.25	0.20	9.50	259.70
10	2.80	0.50	4.70	1.21	0.20	9.50	251.25
11	3.00	0.50	5.00	1.17	0.20	9.50	244.33
12	3.20	0.50	5.30	1.15	0.20	9.50	239.76
13	3.40	0.50	5.60	1.13	0.20	9.50	234.83
14	3.60	0.50	5.90	1.11	0.20	9.50	231.05
15	3.80	0.50	6.20	1.10	0.20	9.50	228.54
16	4.00	0.50	6.50	1.10	0.20	9.50	229.21

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

[illegible]

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

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If $I_c > 2.20$
$a = 14$ for $Q_{tn} > 14$
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$M_{CPT} = a \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$
[illegible]

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

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$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$OCR = k_{OCR} \cdot Q_{tn}$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

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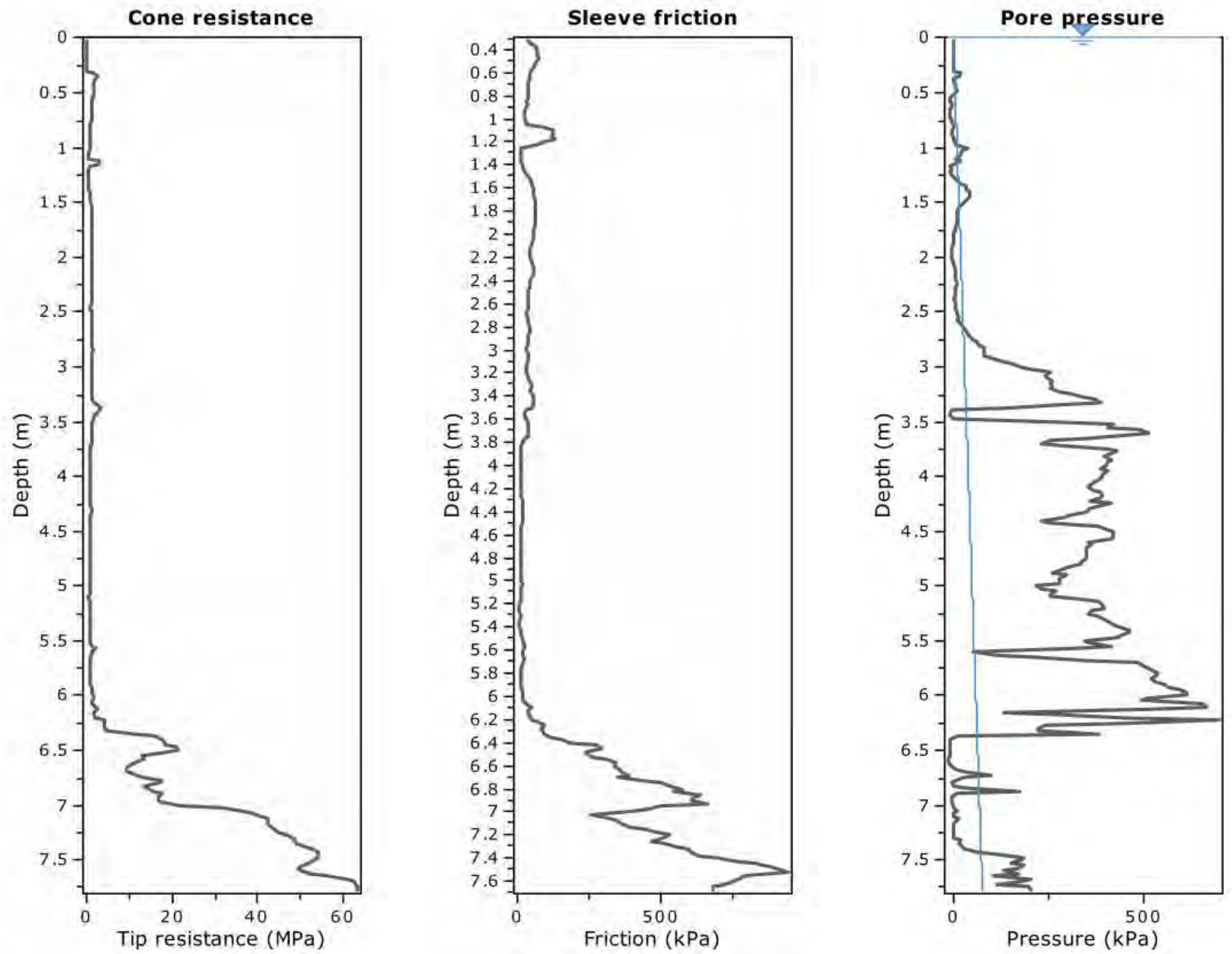
Project:
Location:



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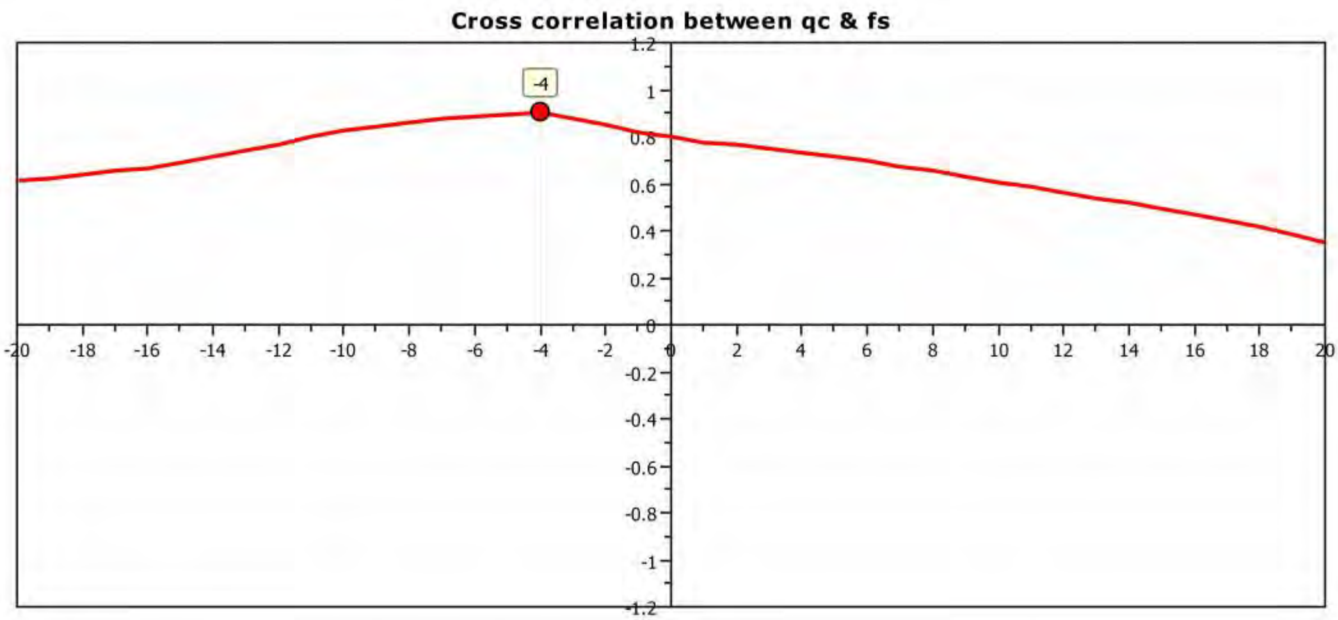

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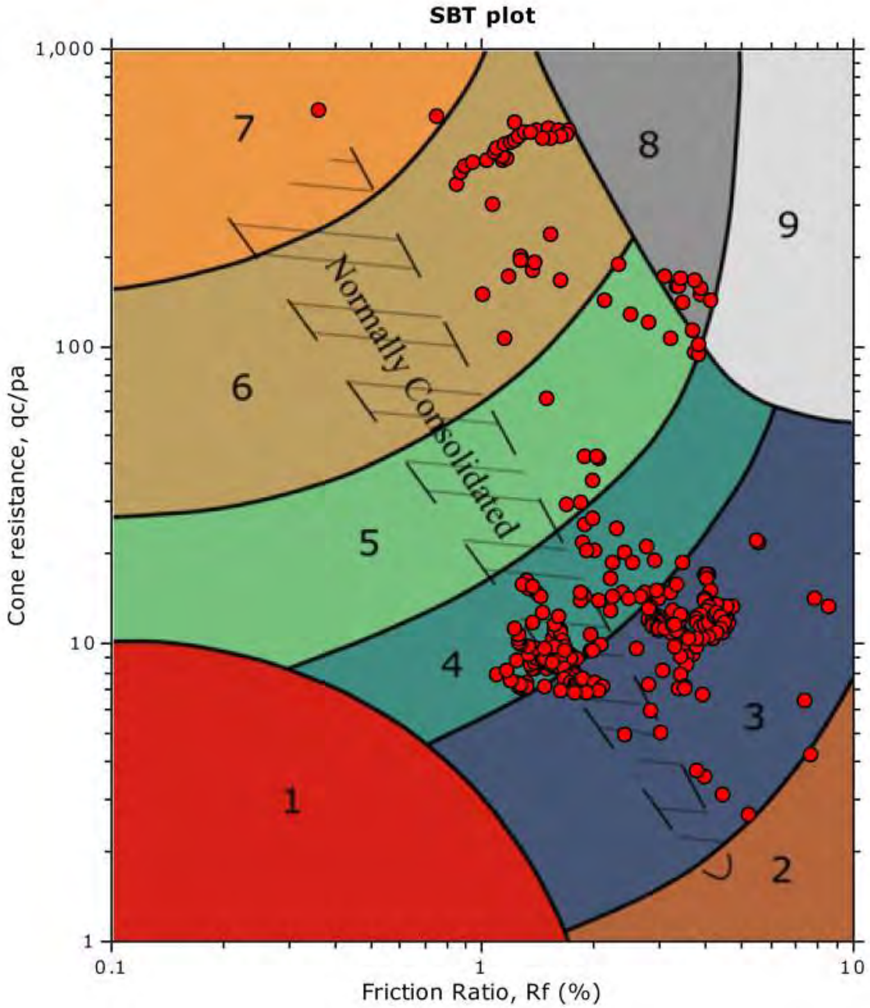
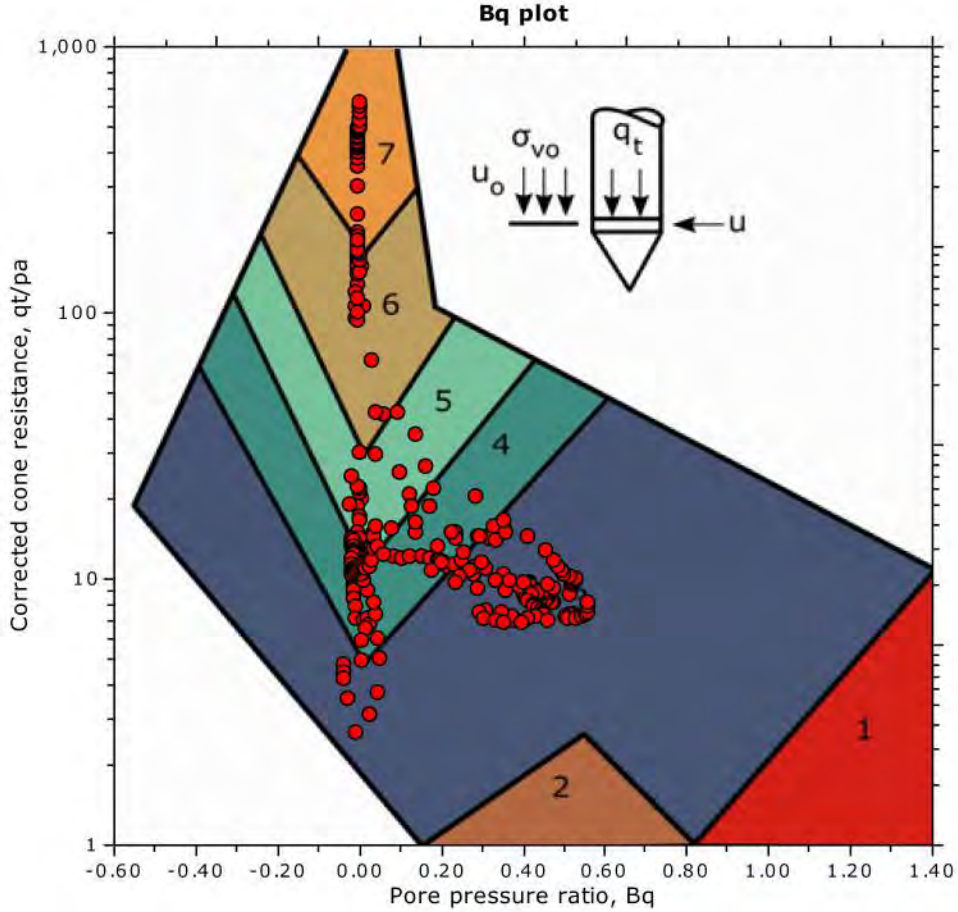
CPT: Sheet1
Total depth: 7.78 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:

SBT - Bq plots

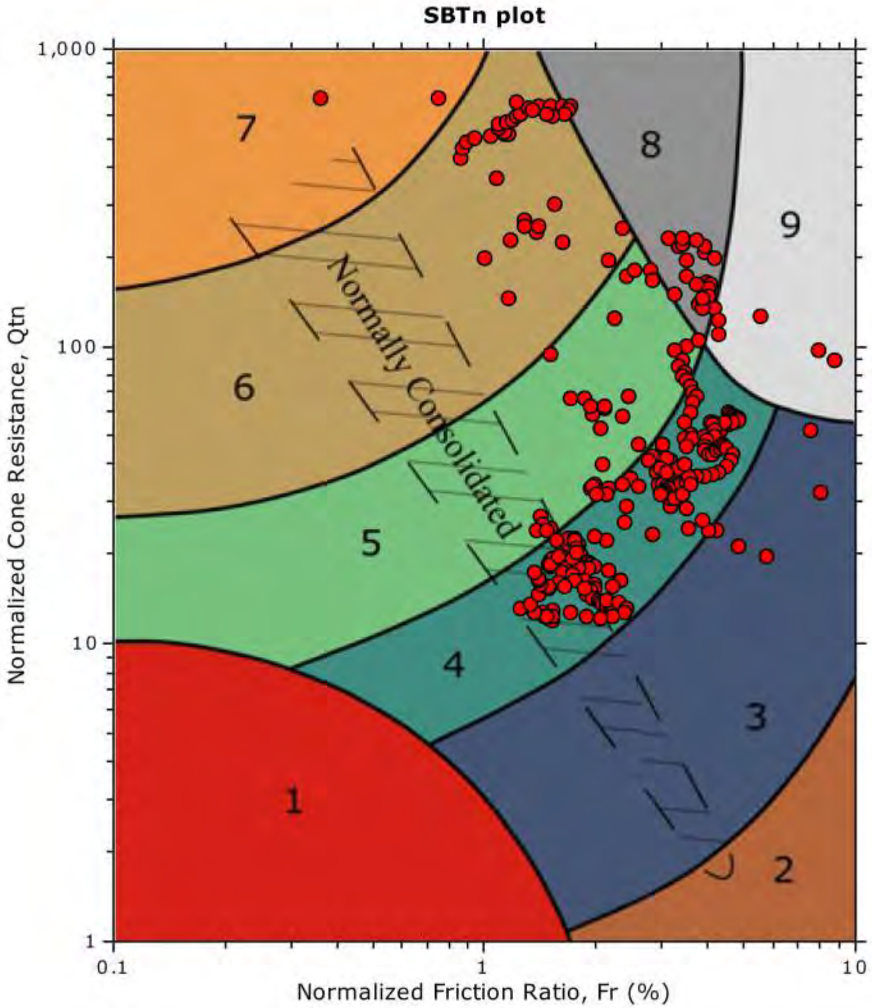


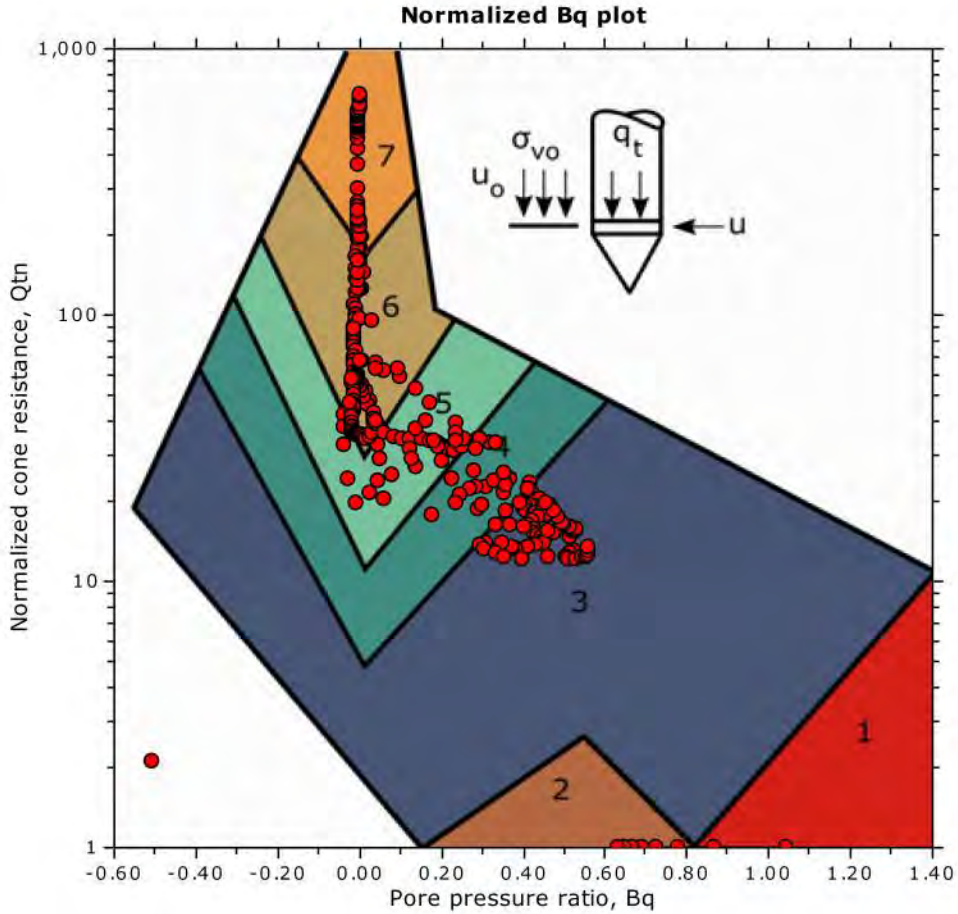


SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

SBT - Bq plots (normalized)

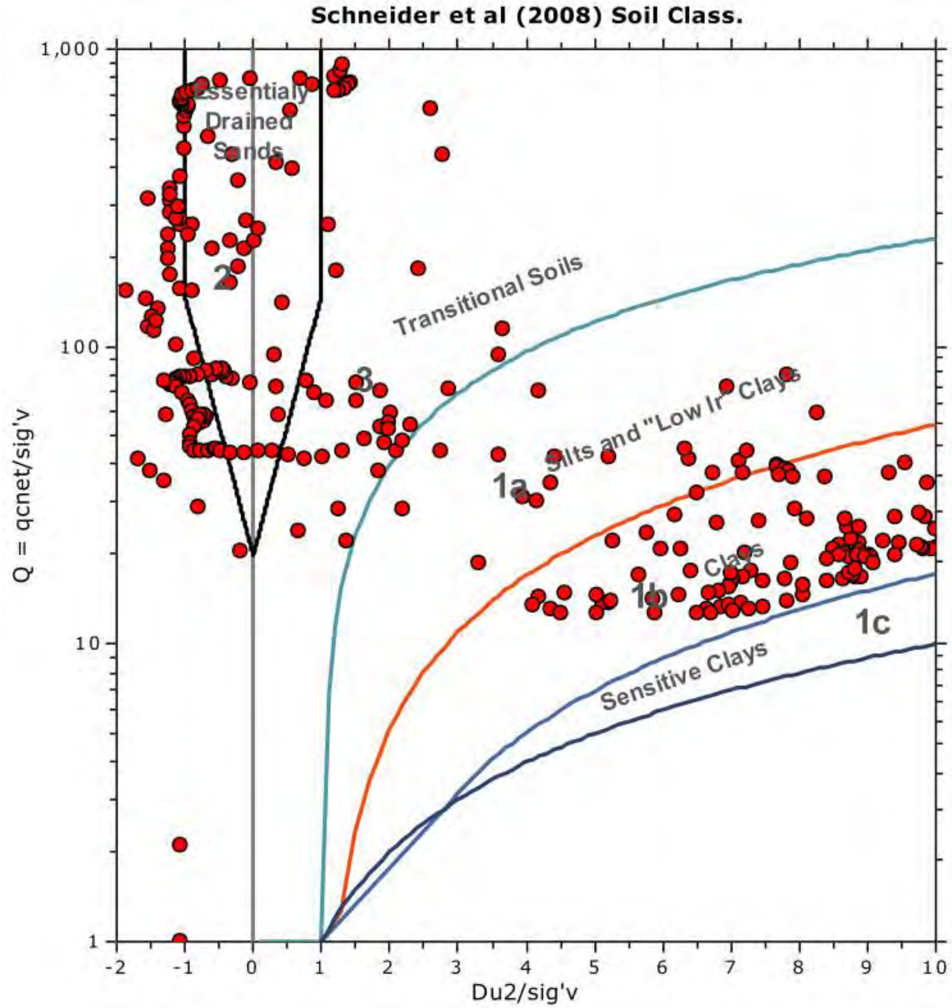


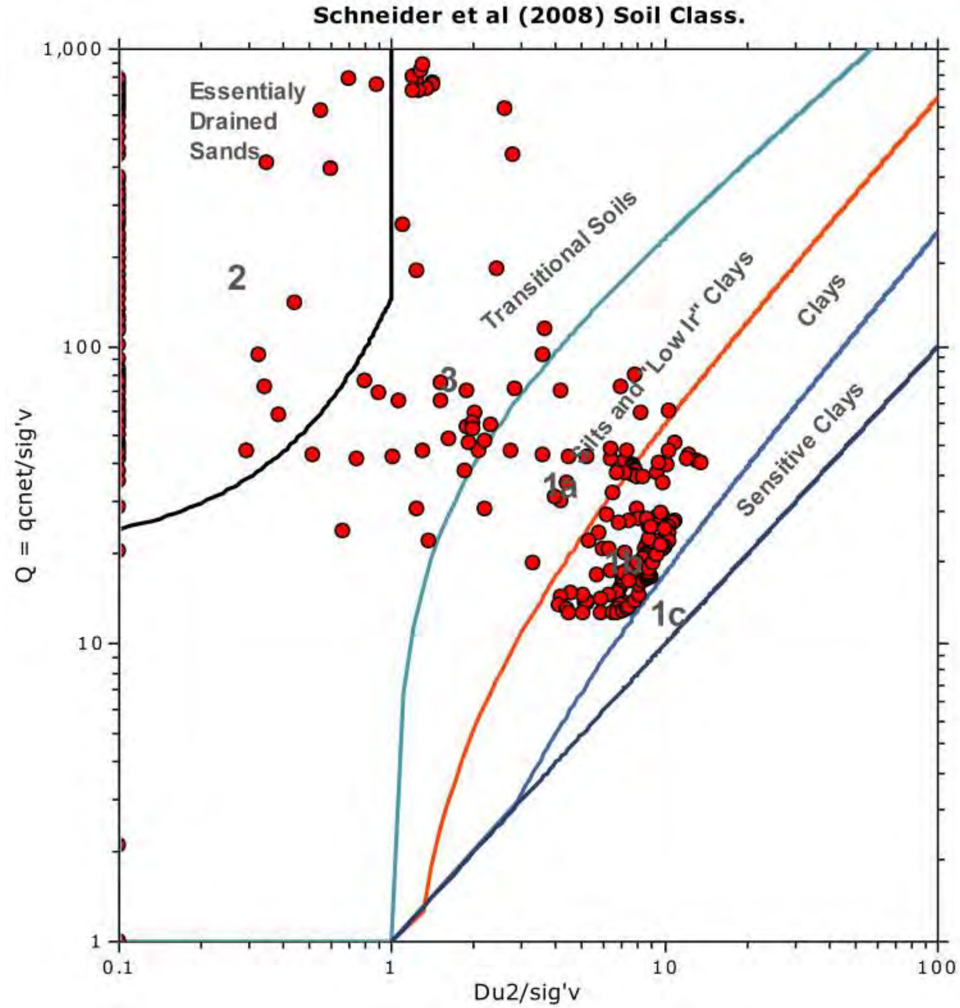


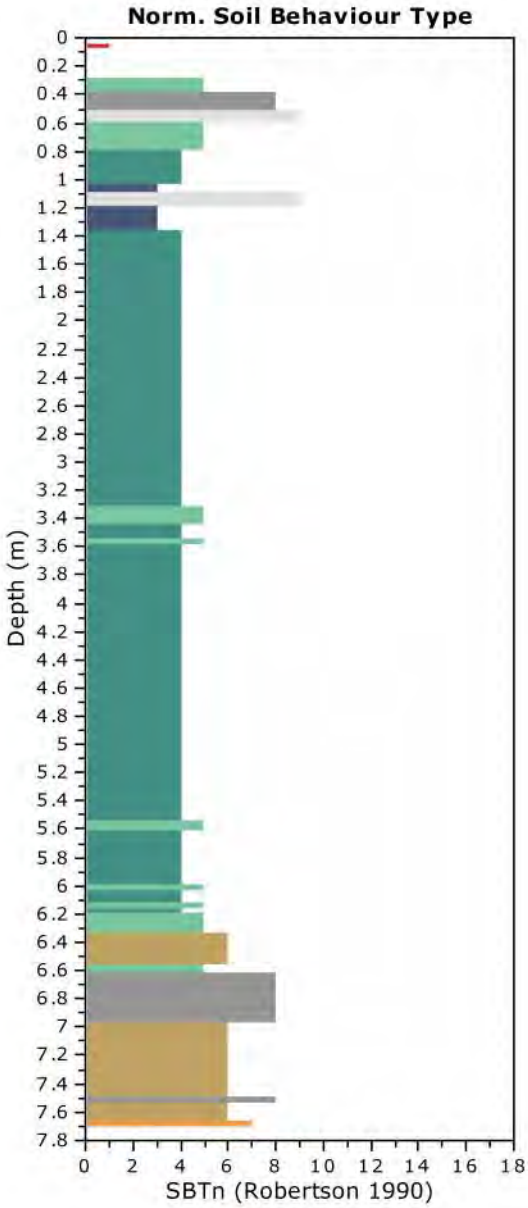
SBTn legend

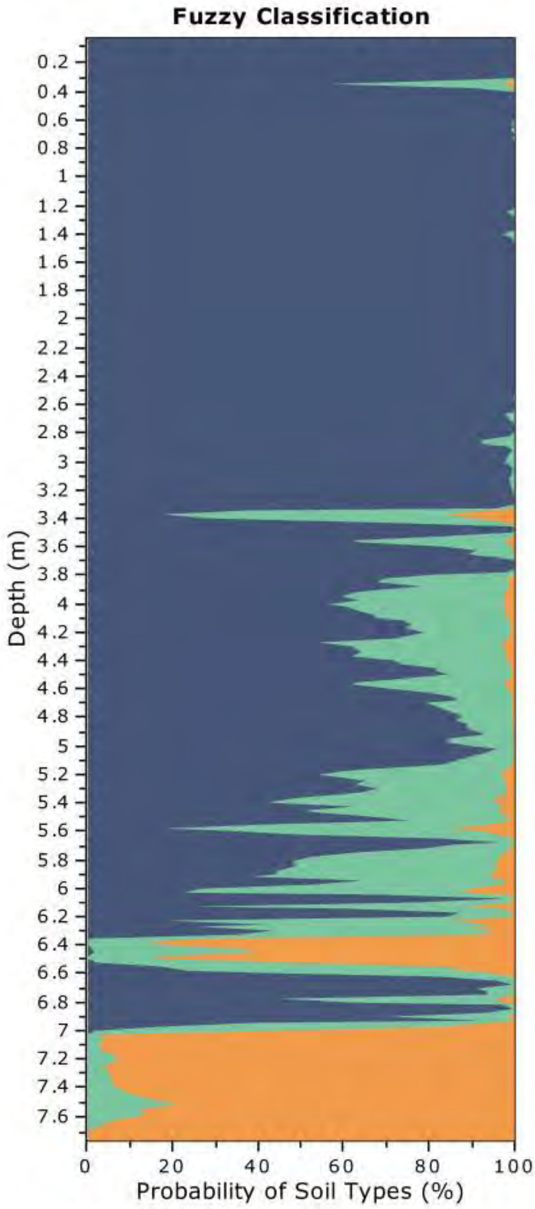
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Bq plots (Schneider)






Norm. Soil Behaviour Type
Depth (m)
SBTn (Robertson 1990)


Fuzzy Classification
Depth (m)
Probability of Soil Types (%)

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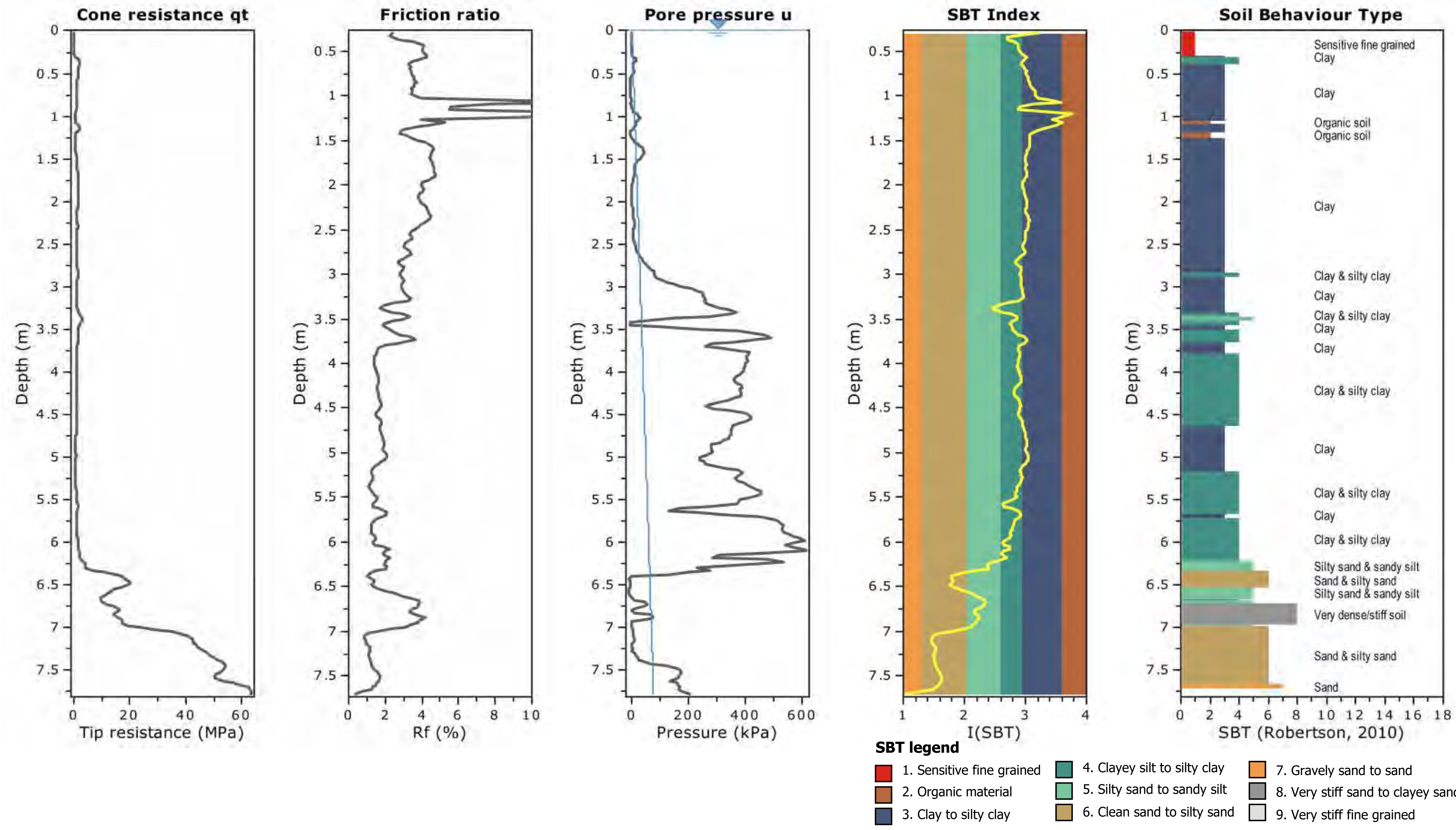
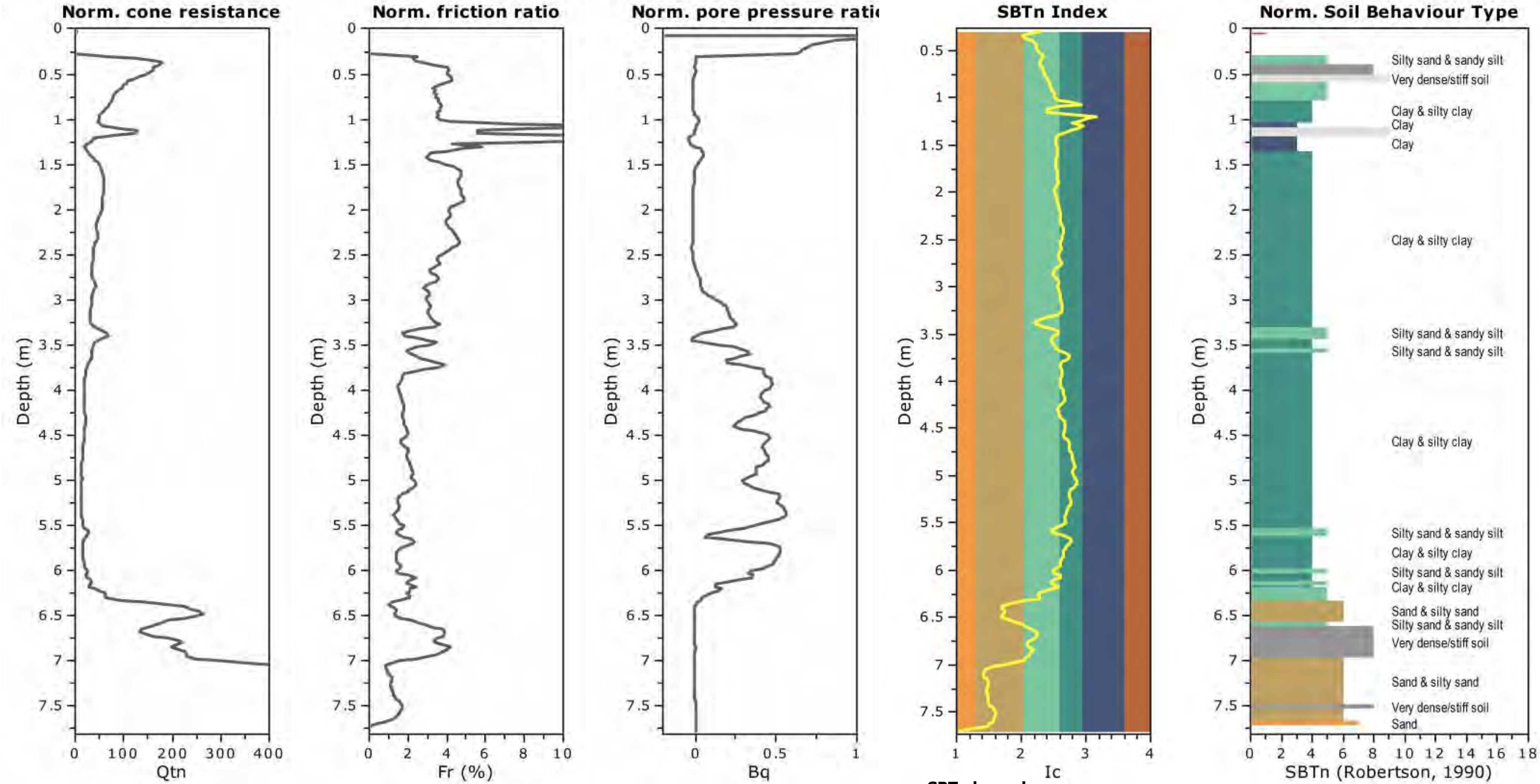
CPT: Sheet1
Total depth: 7.78 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:





SBTn legend

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7. Gravely sand to sand
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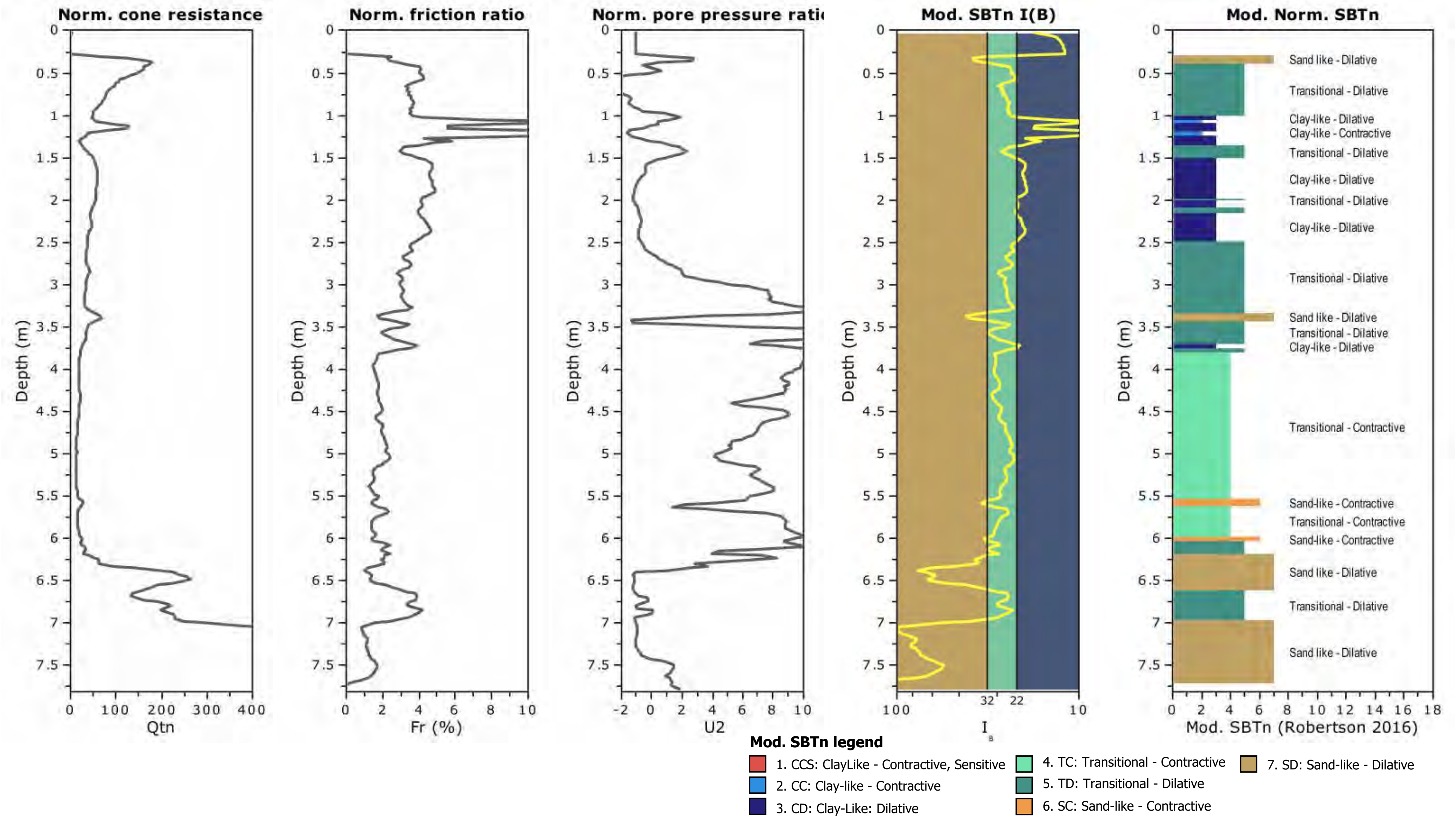
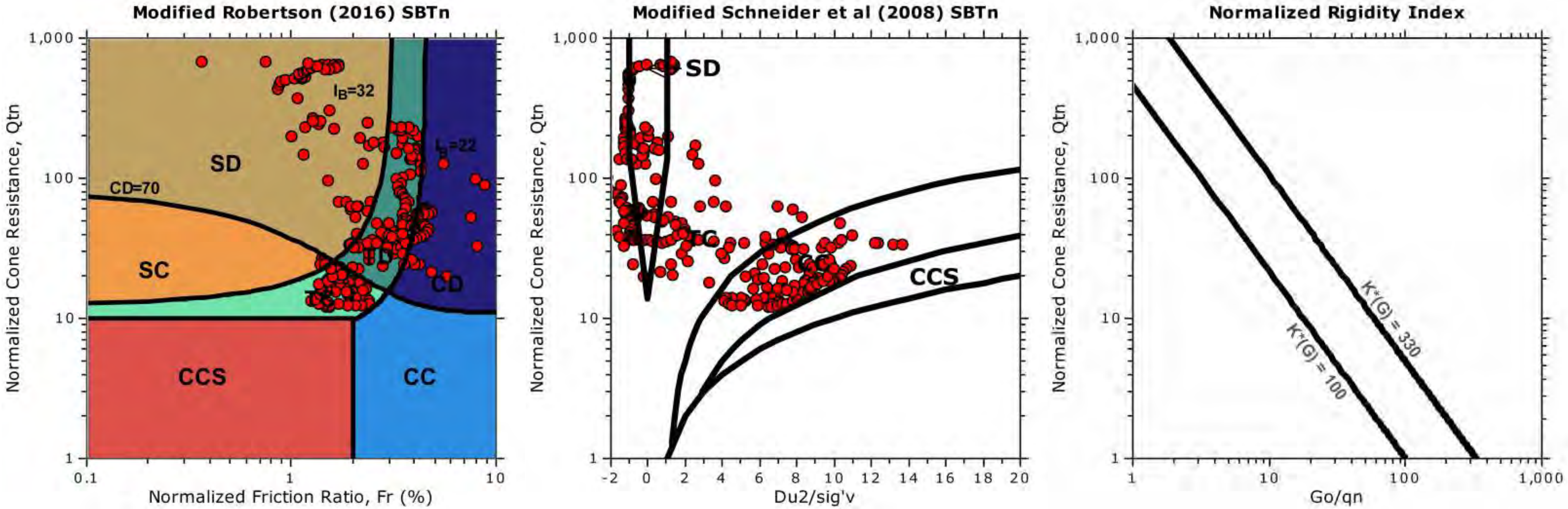
9. Very stiff fine grained


Norm. cone resistance
Qtn
Depth (m)
Norm. friction ratio
Fr (%)
Norm. pore pressure ratio
U2
Mod. SBTn I(B)
32
22
I_B
Mod. Norm. SBTn
Sand like - Dilative
Transitional - Dilative
Clay-like - Dilative
Clay-like - Contractive
Sand-like - Contractive
Transitional - Contractive
Mod. SBTn (Robertson 2016)
Mod. SBTn legend
1. CCS: ClayLike - Contractive, Sensitive
2. CC: Clay-like - Contractive
3. CD: Clay-Like: Dilative
4. TC: Transitional - Contractive
5. TD: Transitional - Dilative
6. SC: Sand-like - Contractive
7. SD: Sand-like - Dilative

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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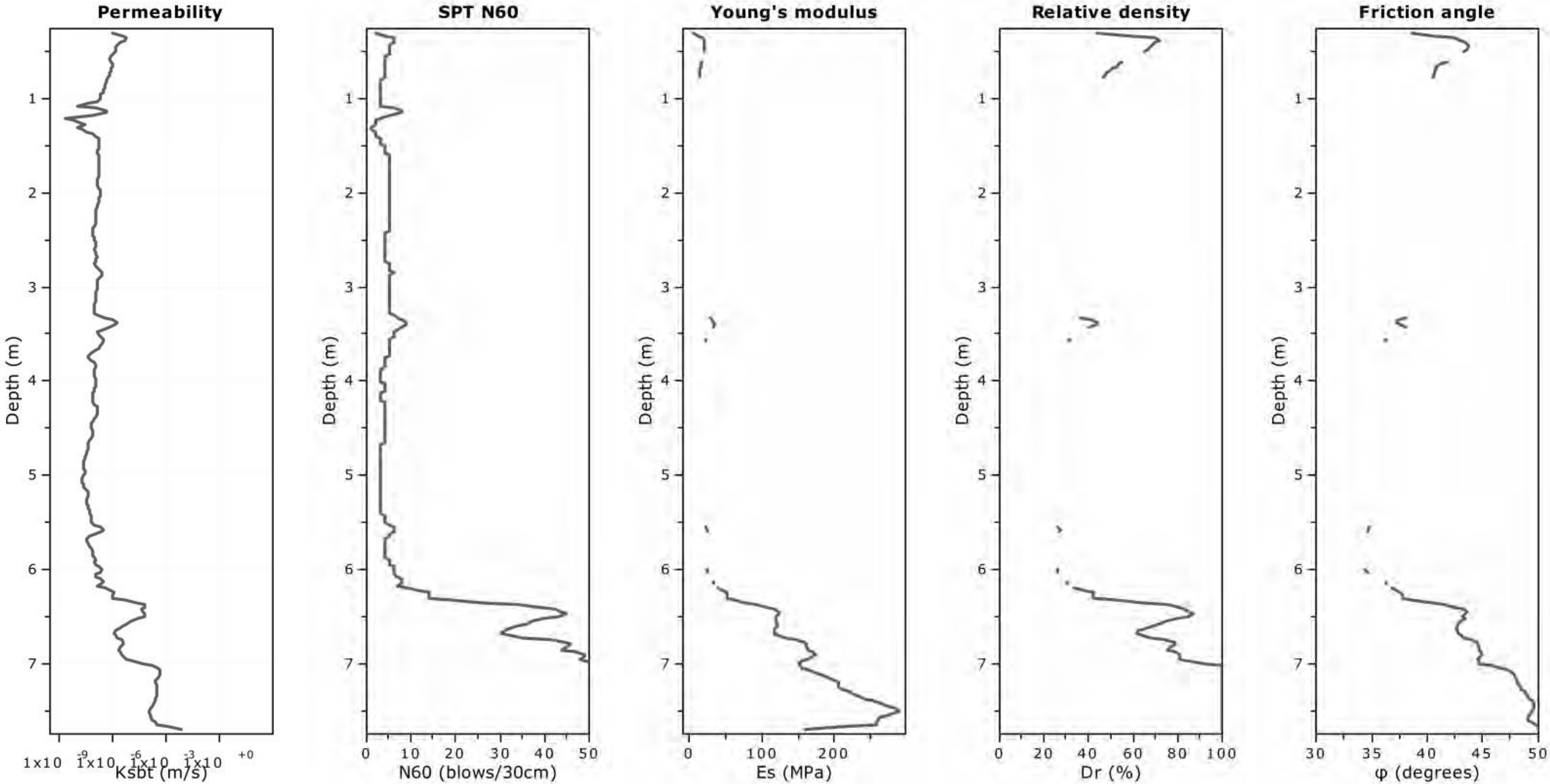
CPT: Sheet1
Total depth: 7.78 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



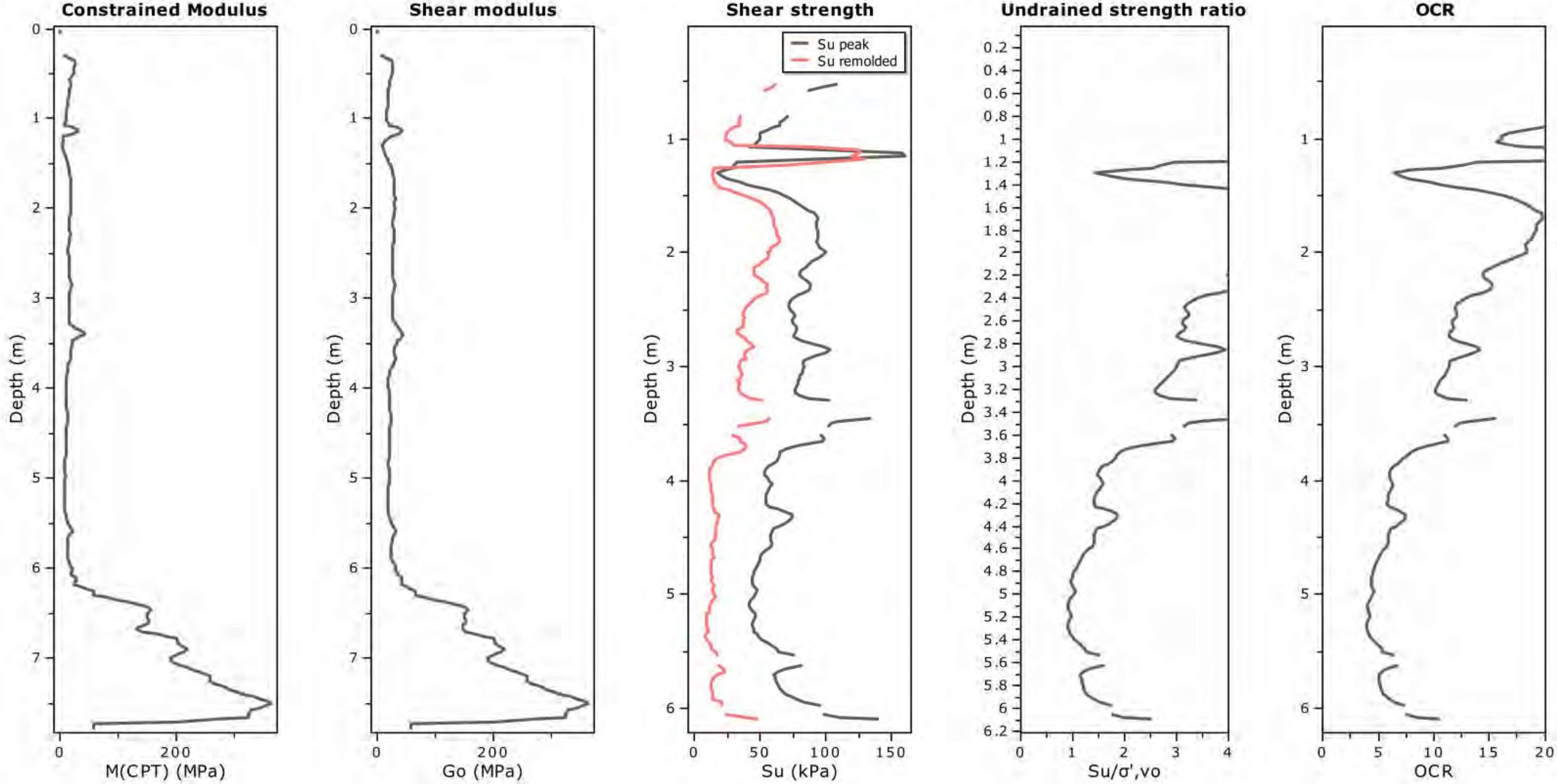
Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

CPT: Sheet1

Total depth: 7.78 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

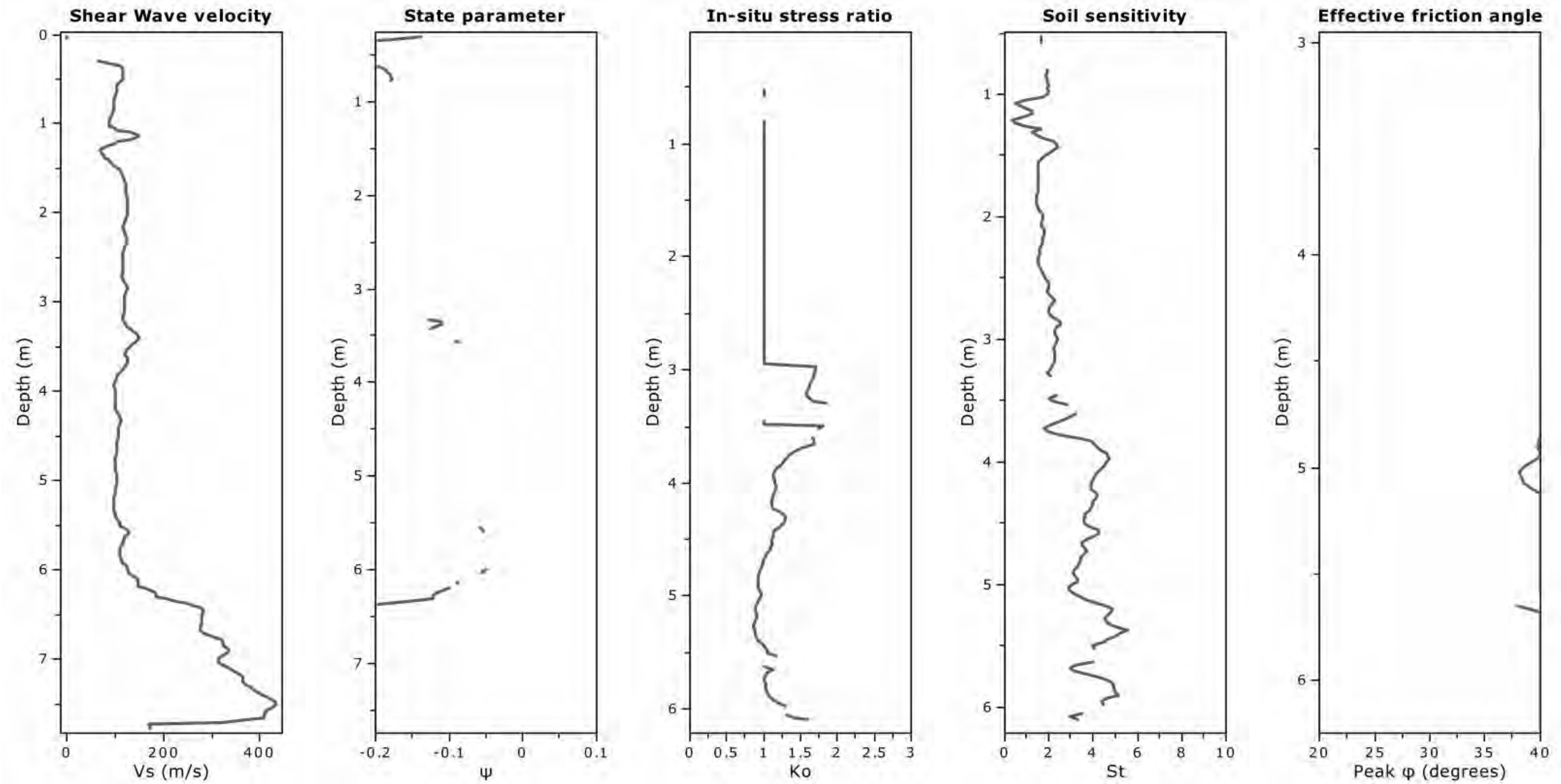
Project:
Location:



Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

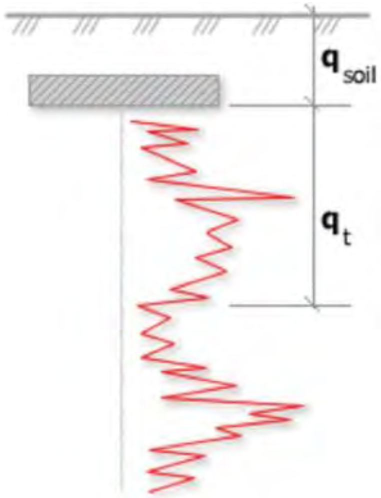
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

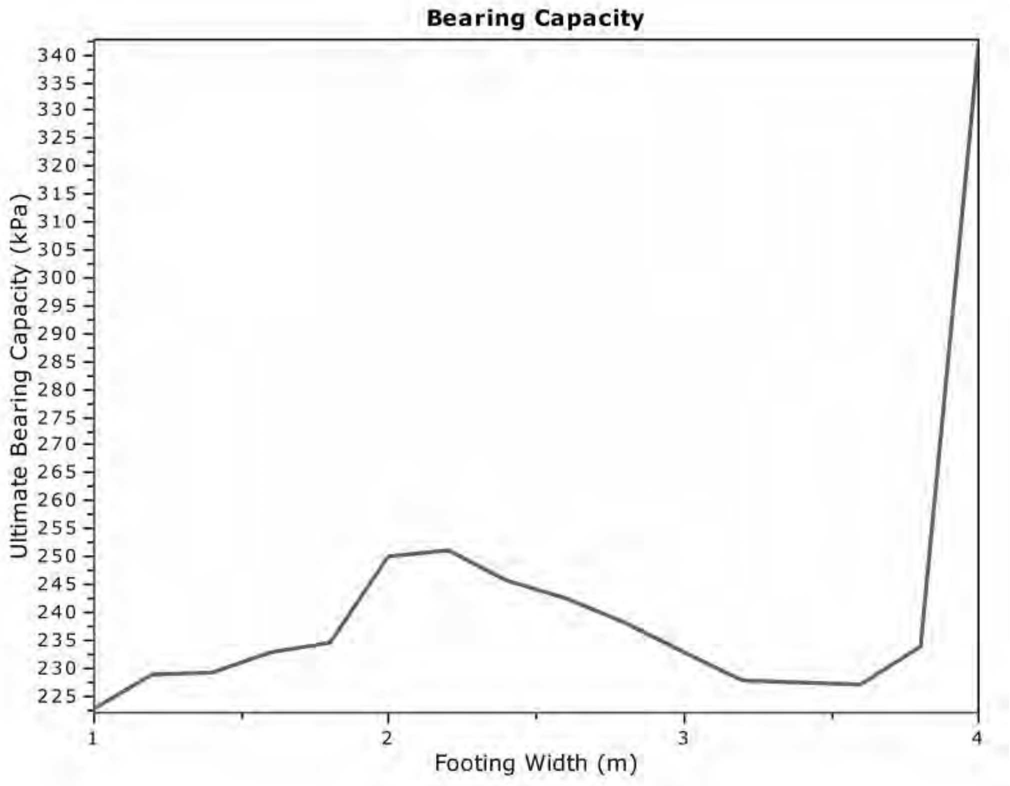
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	1.07	0.20	9.50	222.72
2	1.20	0.50	2.30	1.10	0.20	9.50	228.80
3	1.40	0.50	2.60	1.10	0.20	9.50	229.22
4	1.60	0.50	2.90	1.12	0.20	9.50	232.78
5	1.80	0.50	3.20	1.12	0.20	9.50	234.41
6	2.00	0.50	3.50	1.20	0.20	9.50	250.13
7	2.20	0.50	3.80	1.21	0.20	9.50	251.21
8	2.40	0.50	4.10	1.18	0.20	9.50	245.64
9	2.60	0.50	4.40	1.16	0.20	9.50	242.34
10	2.80	0.50	4.70	1.14	0.20	9.50	238.18
11	3.00	0.50	5.00	1.12	0.20	9.50	232.78
12	3.20	0.50	5.30	1.09	0.20	9.50	227.81
13	3.40	0.50	5.60	1.09	0.20	9.50	227.48
14	3.60	0.50	5.90	1.09	0.20	9.50	226.94
15	3.80	0.50	6.20	1.12	0.20	9.50	234.03
16	4.00	0.50	6.50	1.66	0.20	9.50	342.02

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

[illegible]

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$
[illegible]

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$OCR = k_{OCR} \cdot Q_{tn}$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

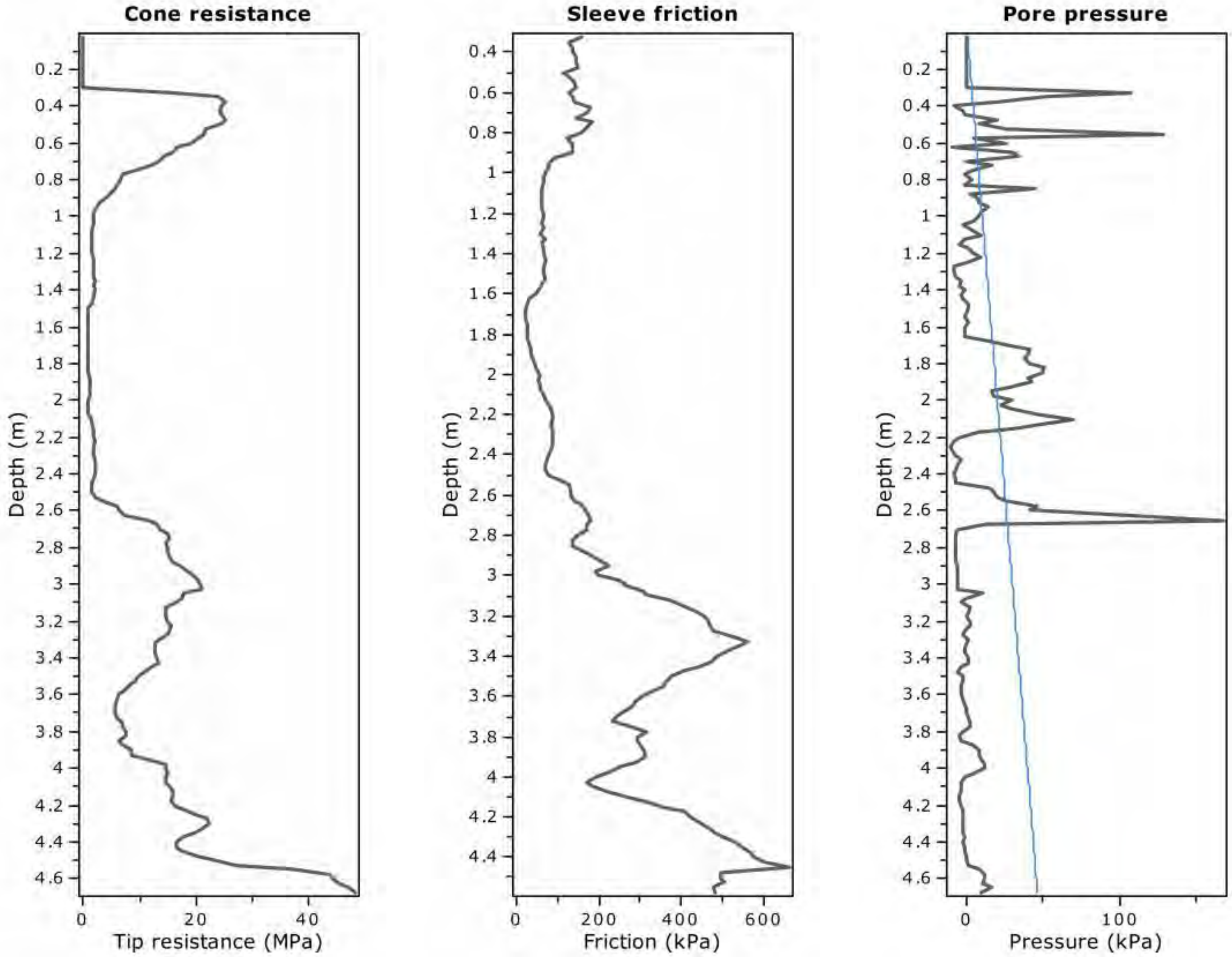
(applicable for $0.10 < B_q < 1.00$)

References

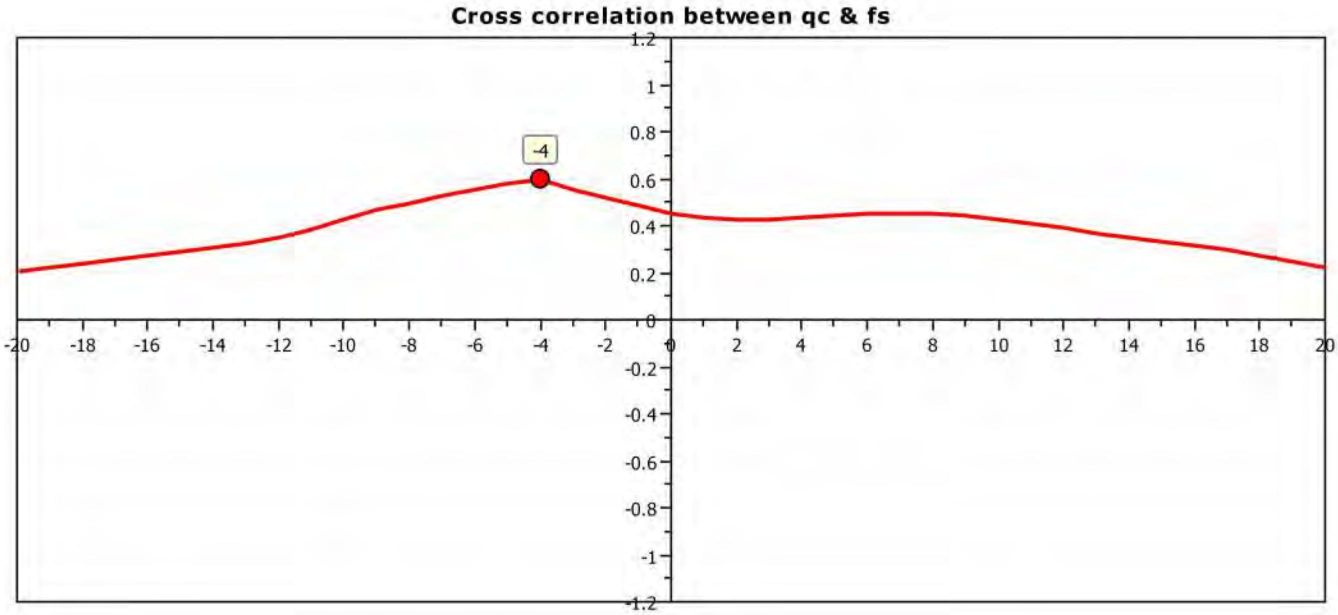
• Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012

• Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

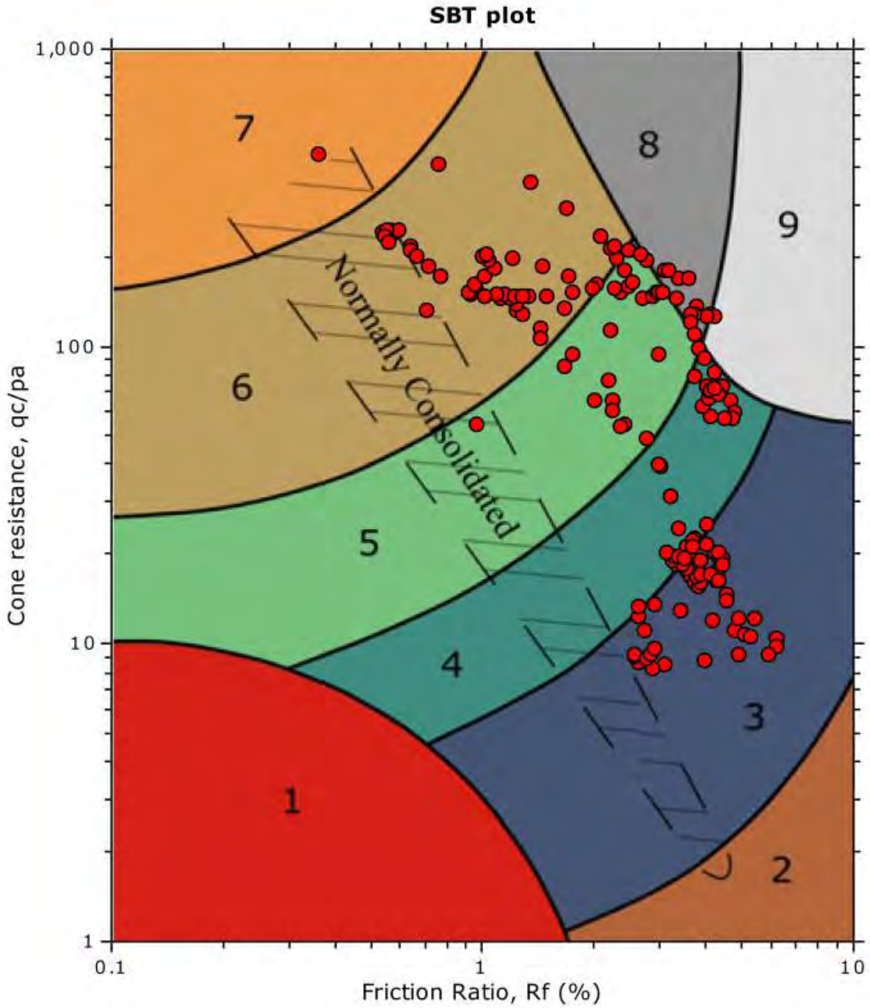
• N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

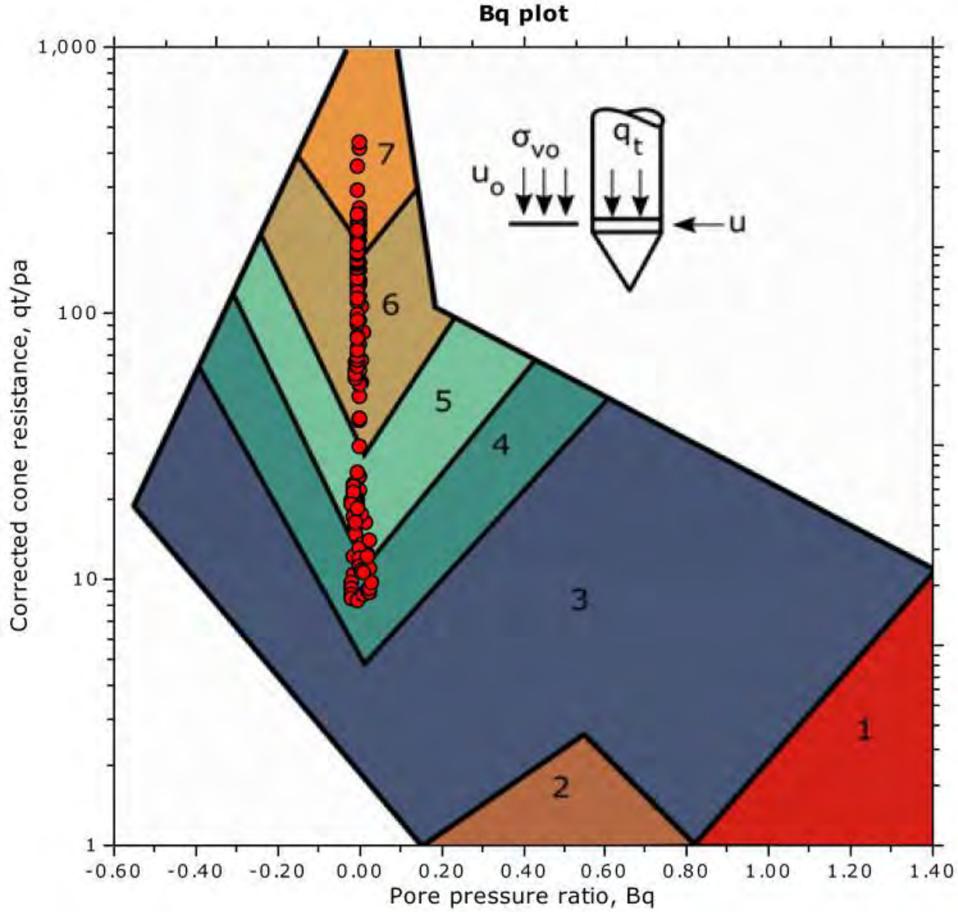


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots

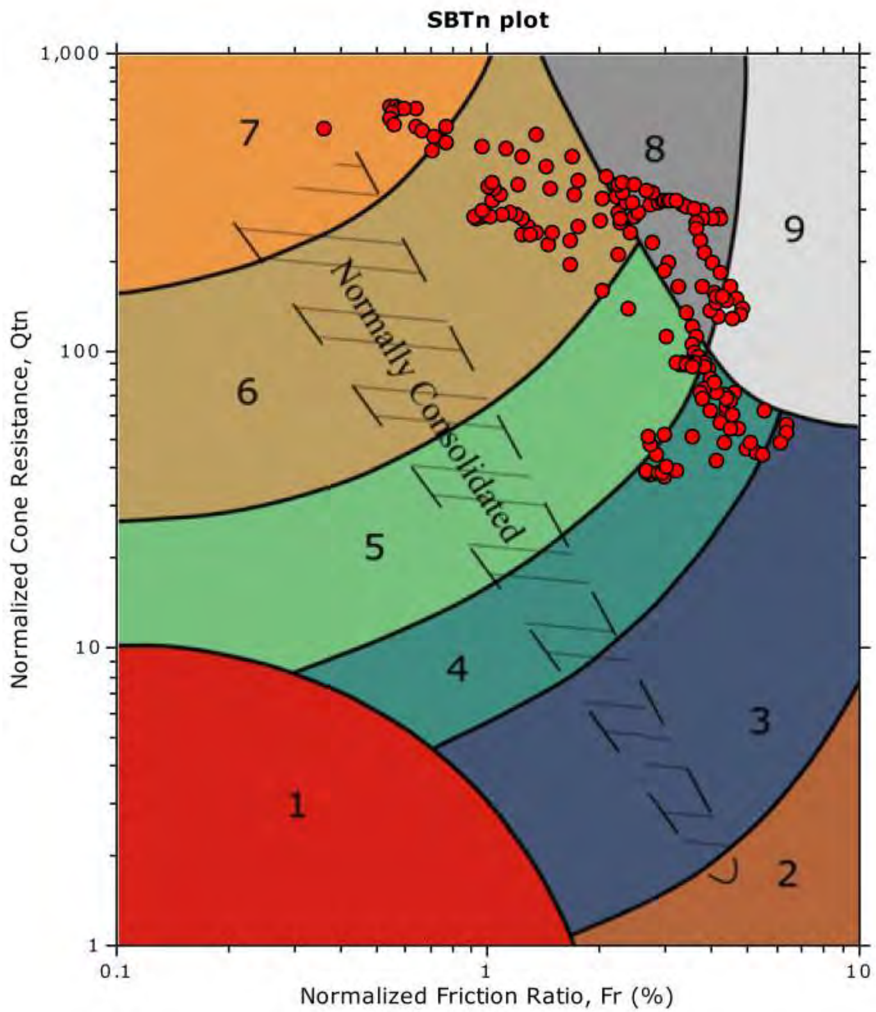


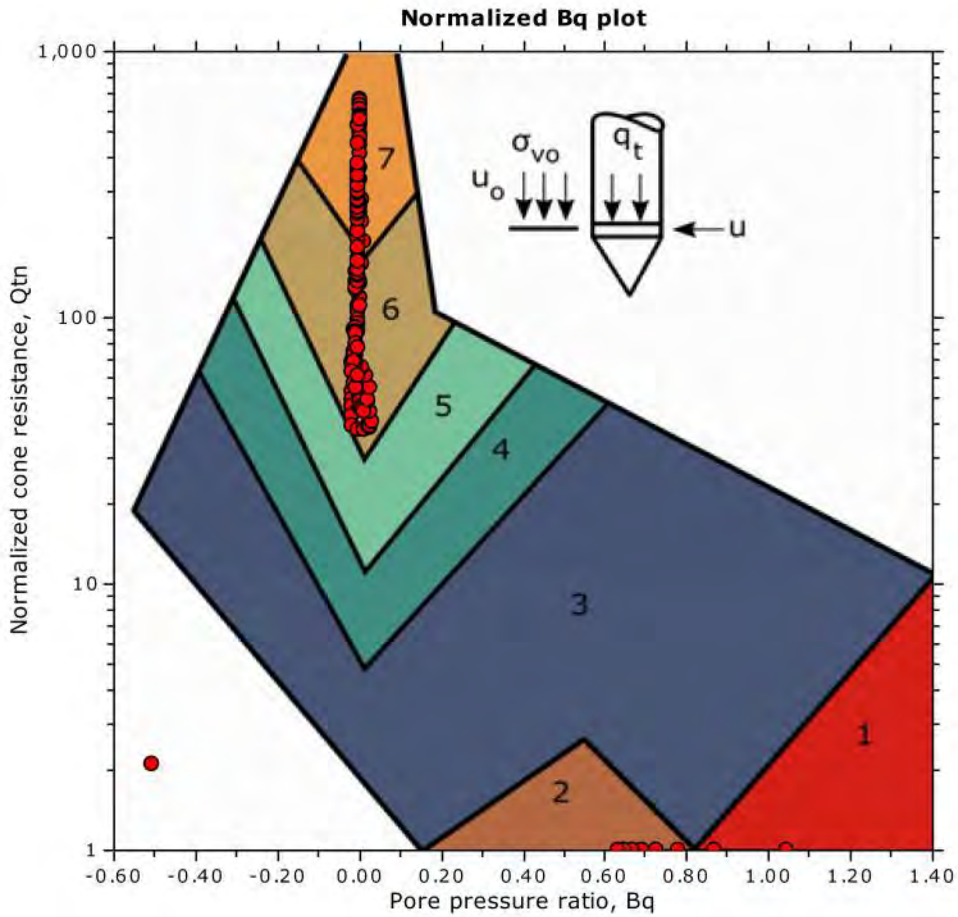


SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

SBT - Bq plots (normalized)

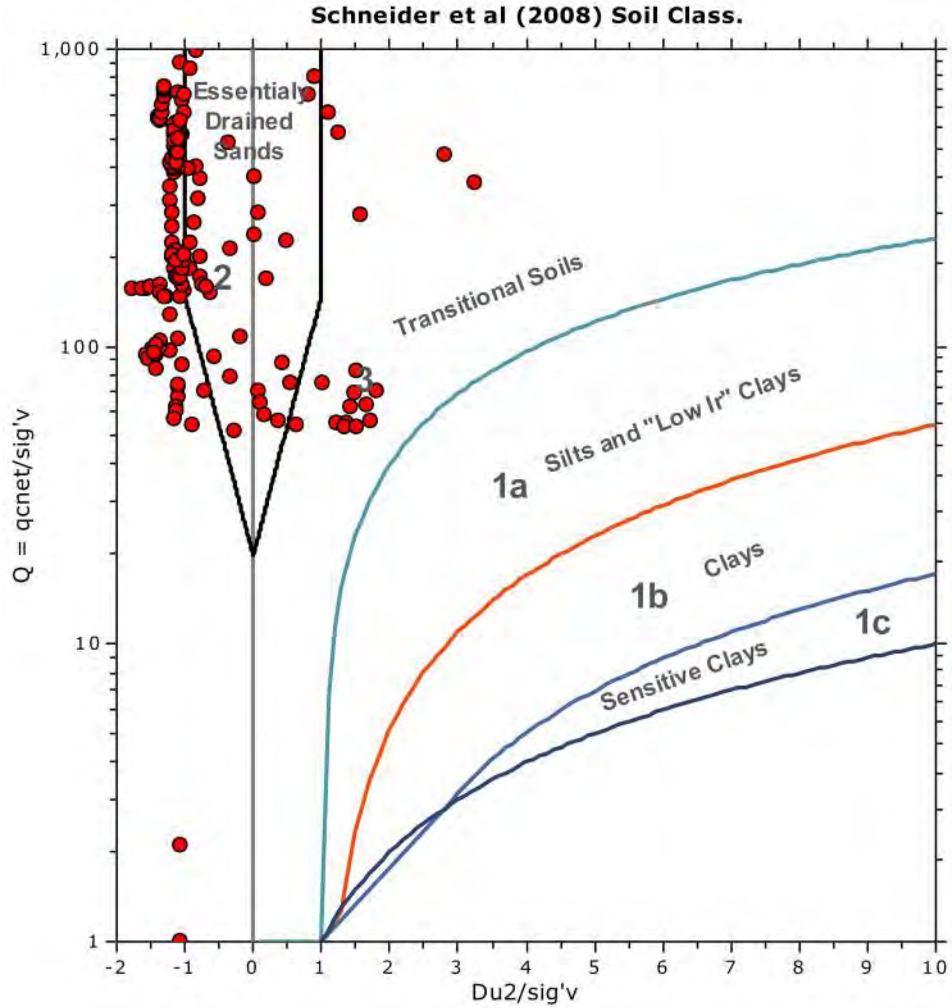


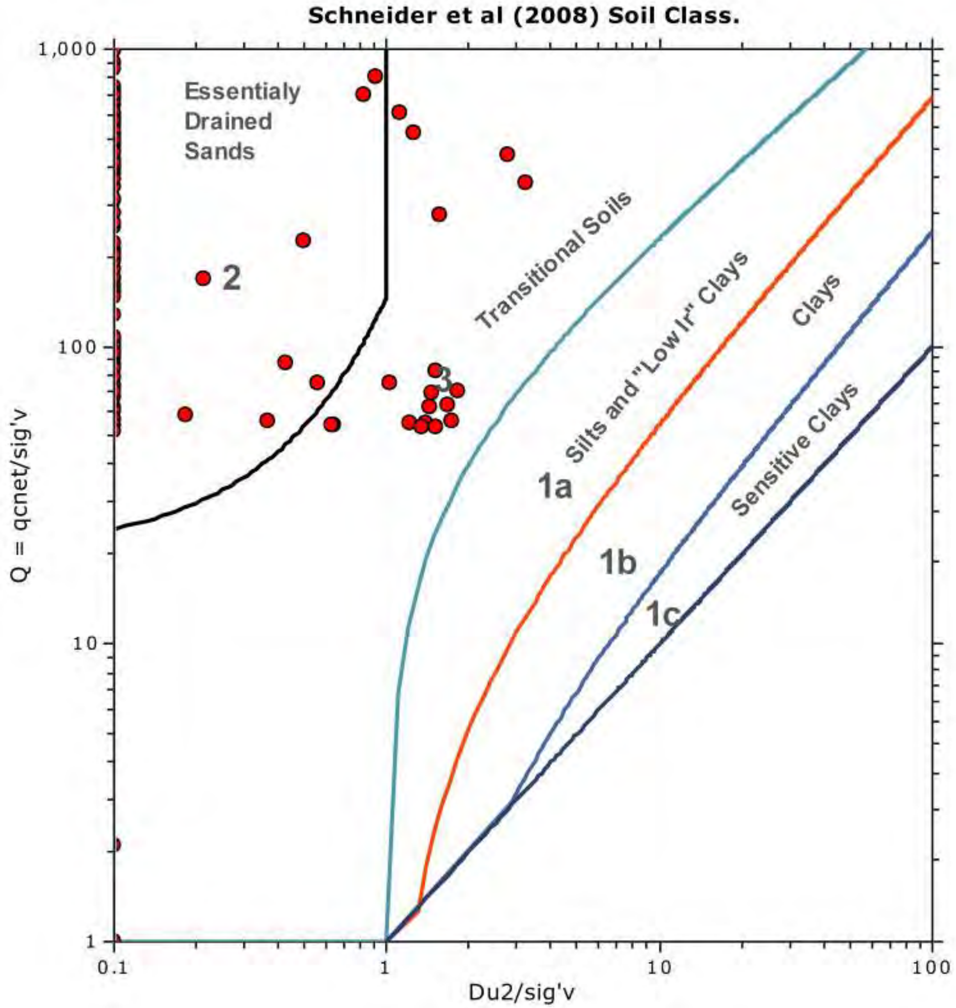


SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Bq plots (Schneider)





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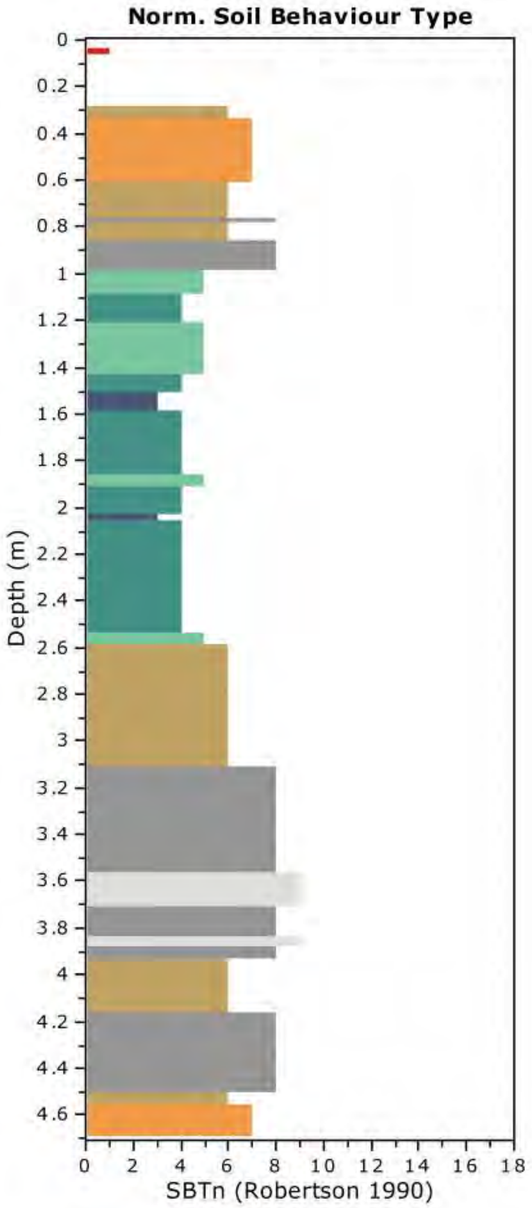
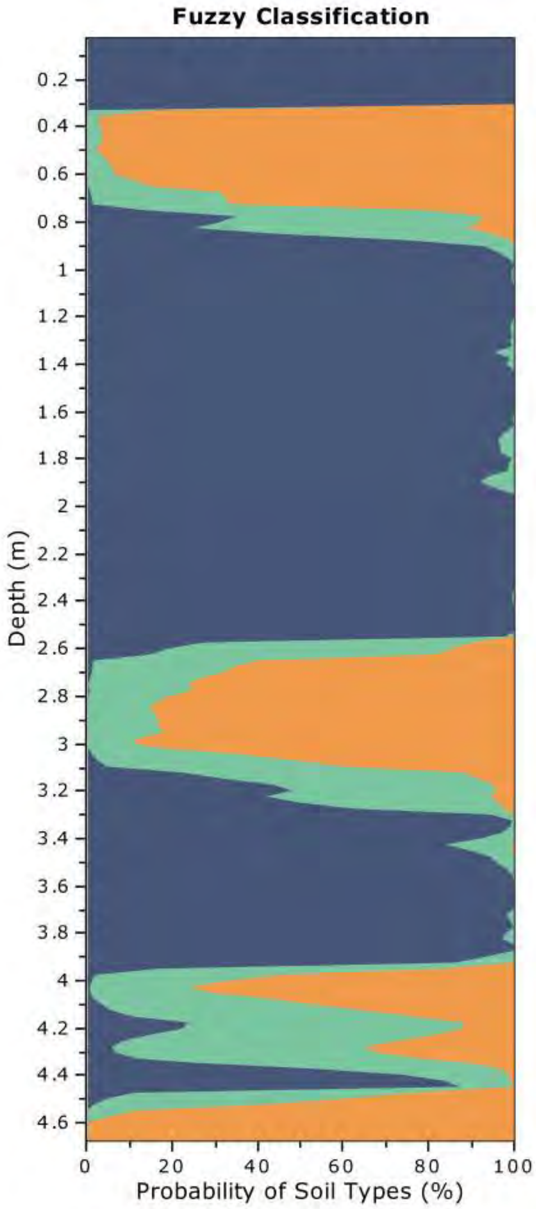
CPT: CPT-04
Total depth: 4.67 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:





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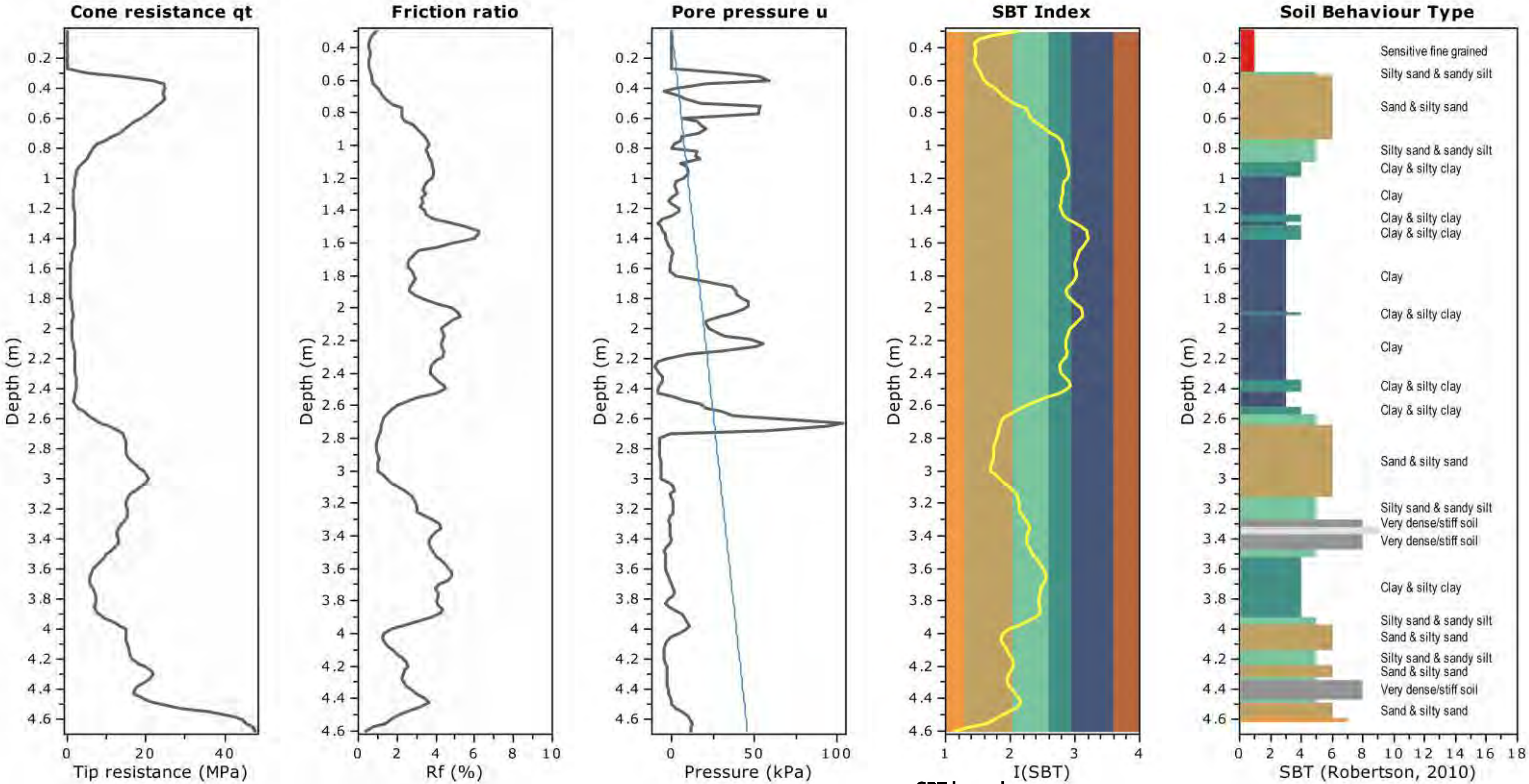
CPT: CPT-04
Total depth: 4.67 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



SBT legend

1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

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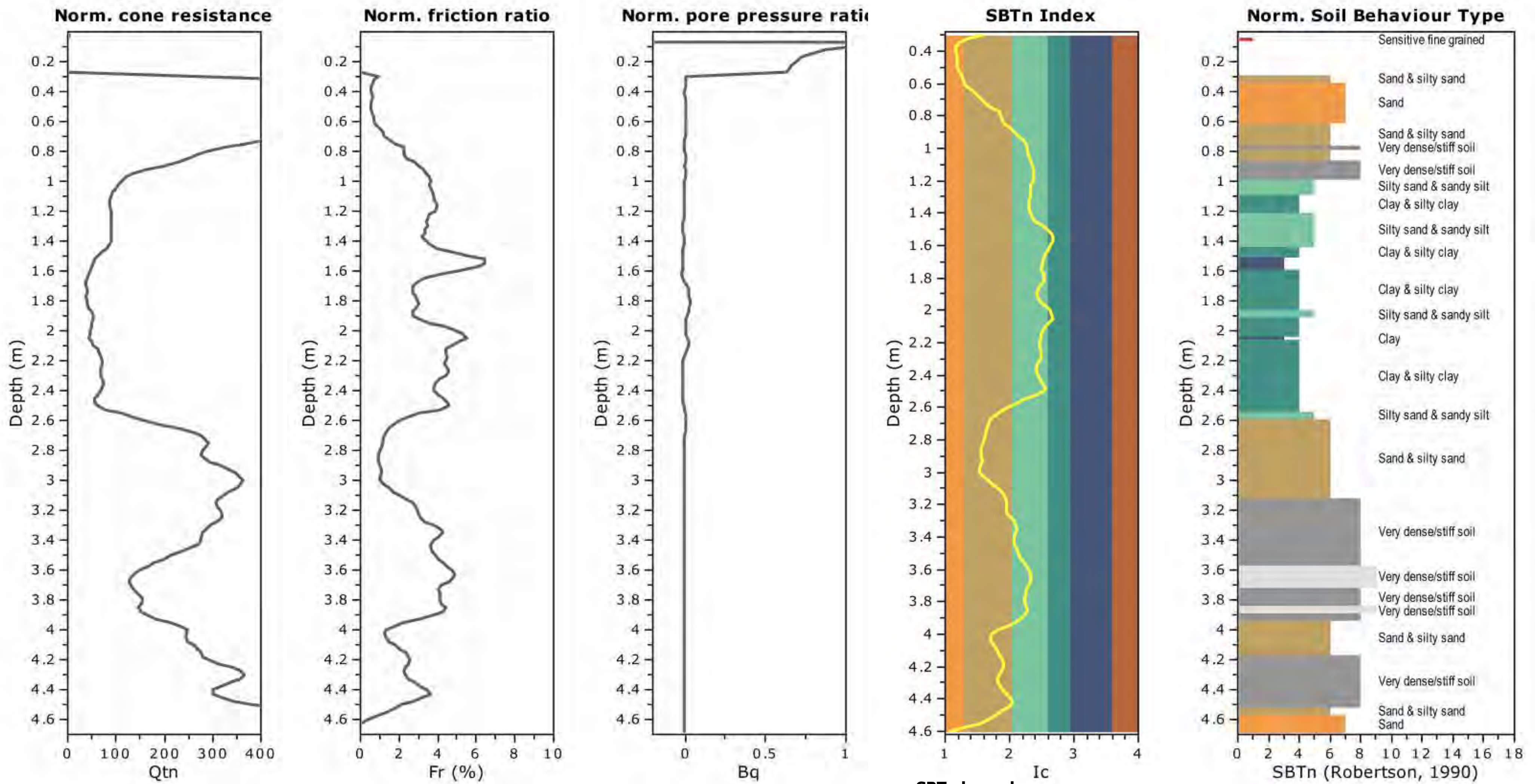
CPT: CPT-04
Total depth: 4.67 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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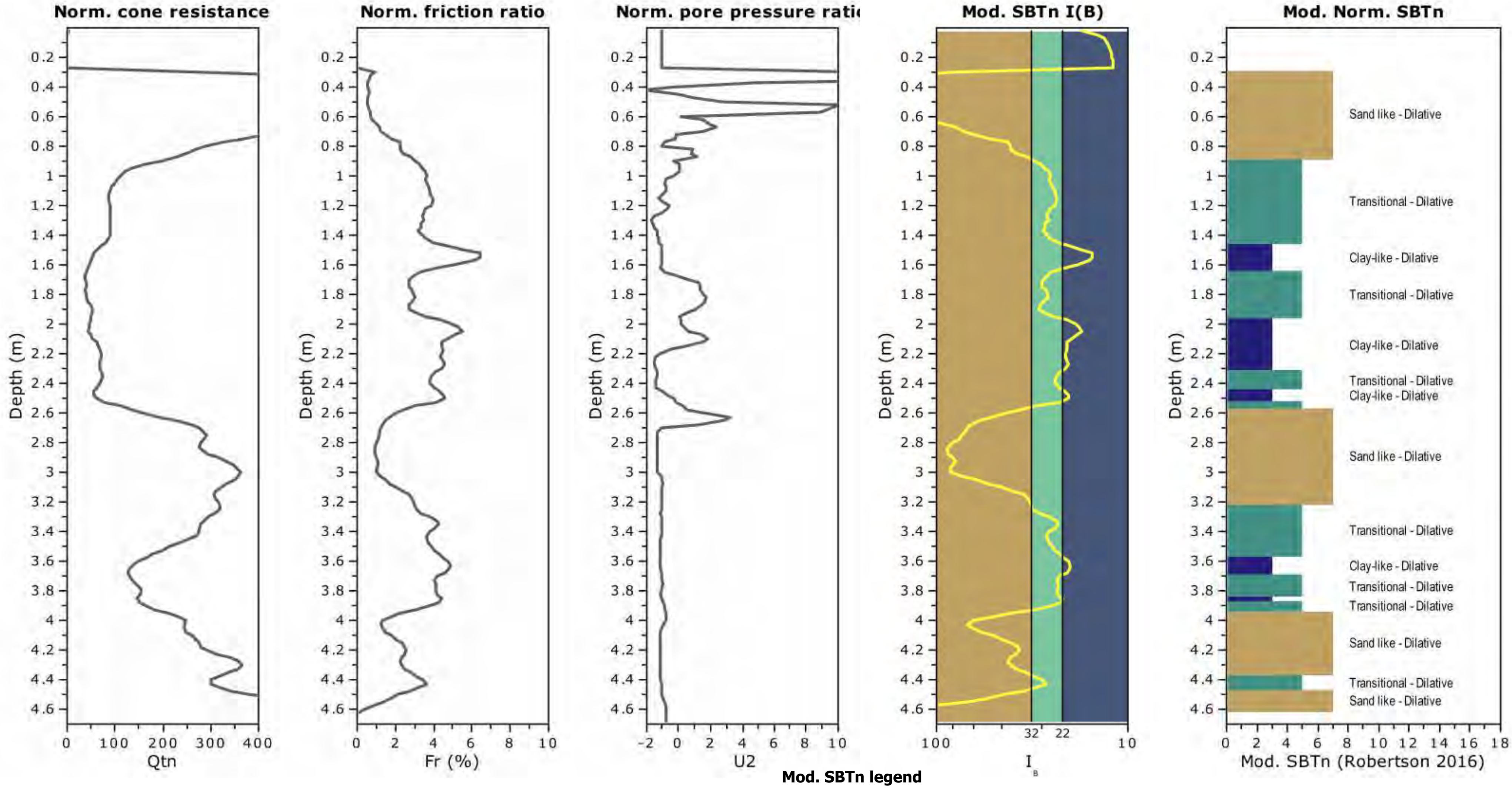
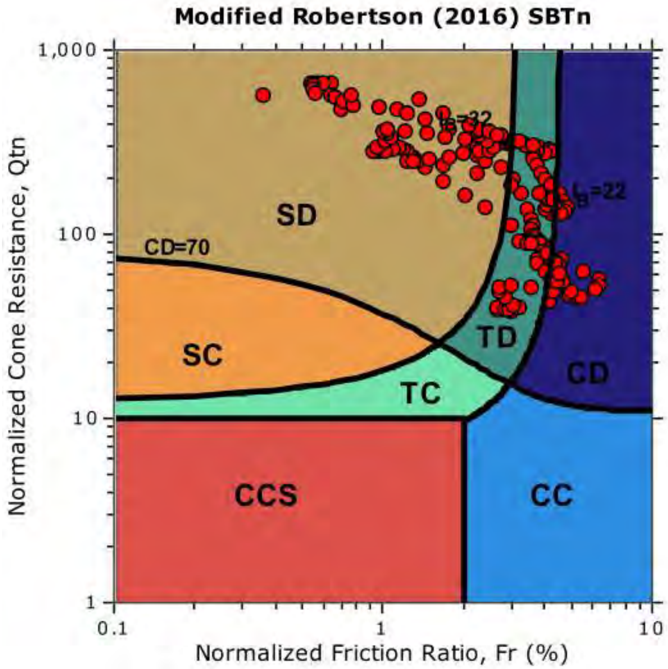
CPT: CPT-04
Total depth: 4.67 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

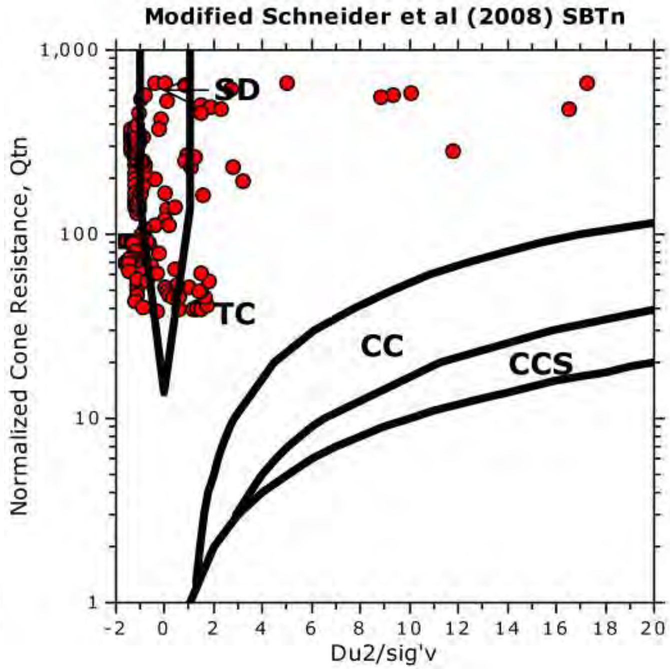
Project:
Location:

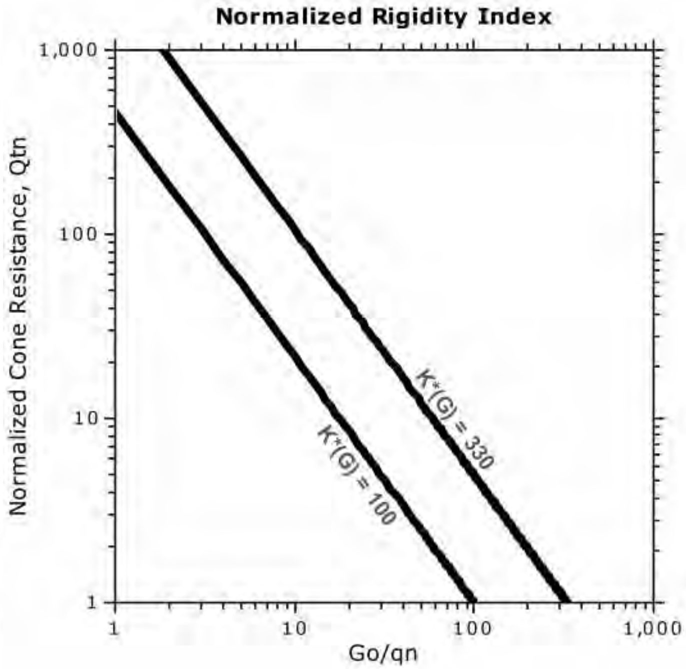


Mod. SBTn legend
1. CCS: ClayLike - Contractive, Sensitive
2. CC: Clay-like - Contractive
3. CD: Clay-Like: Dilative
4. TC: Transitional - Contractive
5. TD: Transitional - Dilative
6. SC: Sand-like - Contractive
7. SD: Sand-like - Dilative

Updated SBTn plots







CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
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TD: Transitional - Dilative
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K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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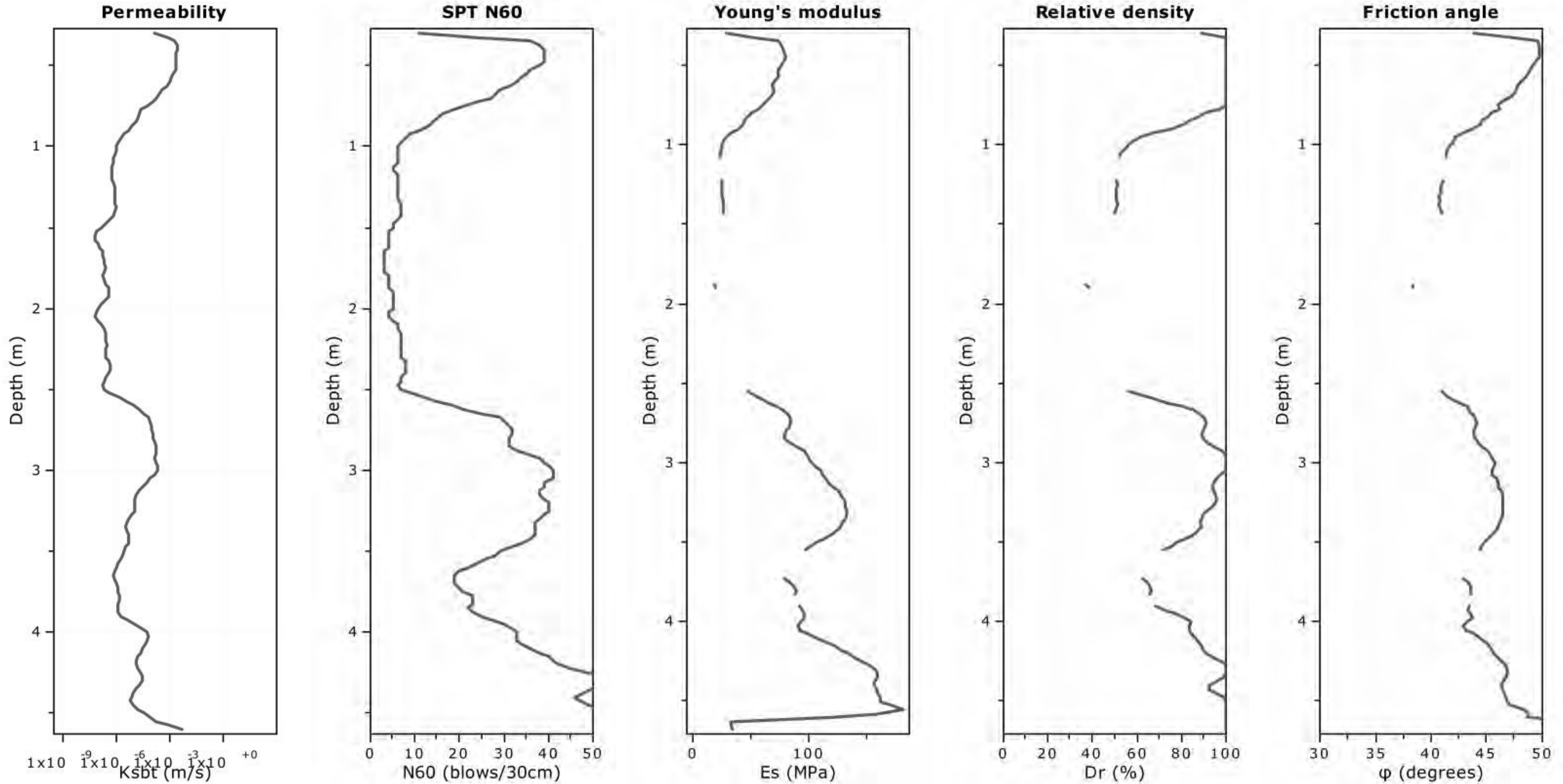
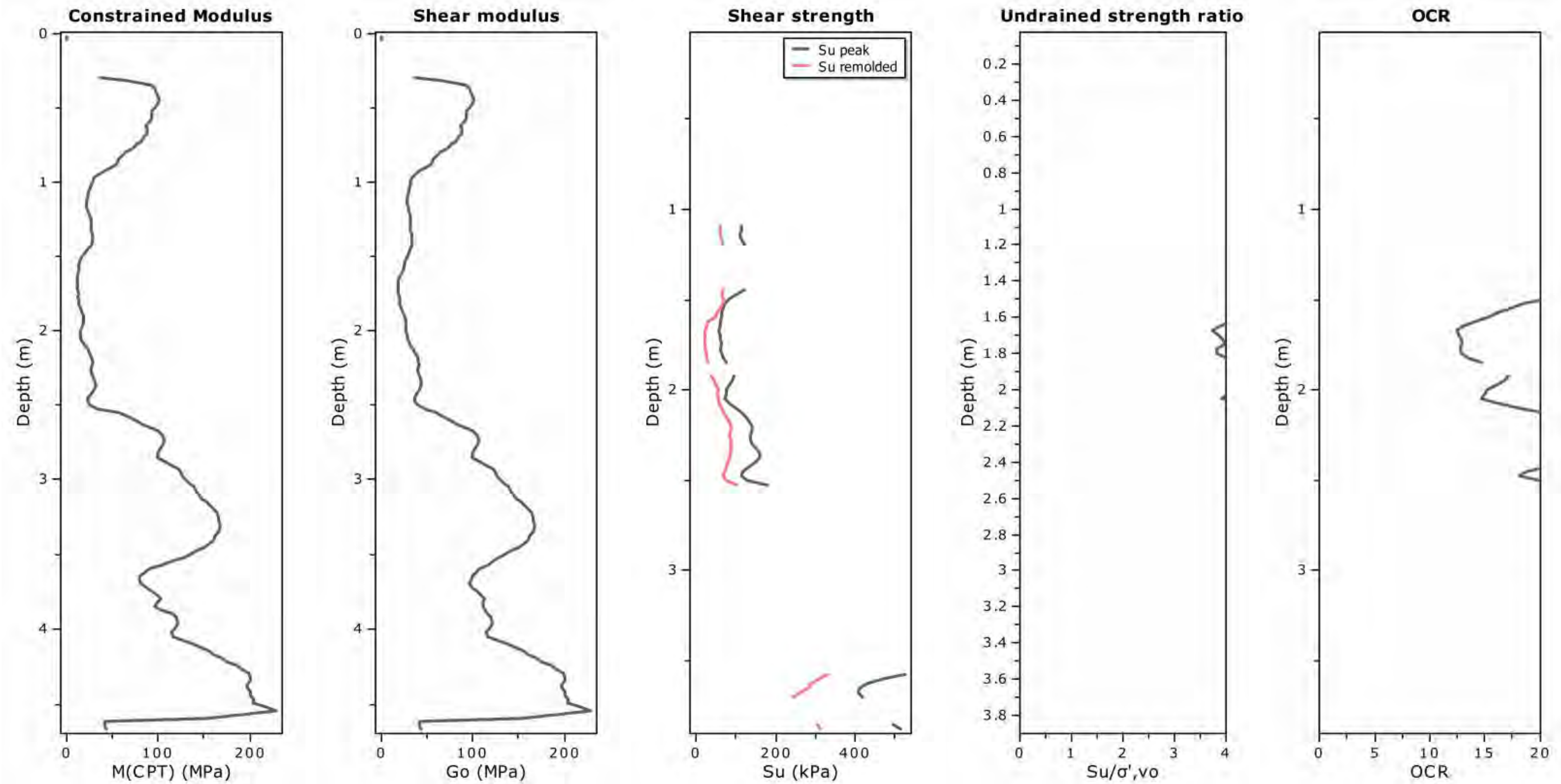
CPT: CPT-04
Total depth: 4.67 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



Calculation parameters

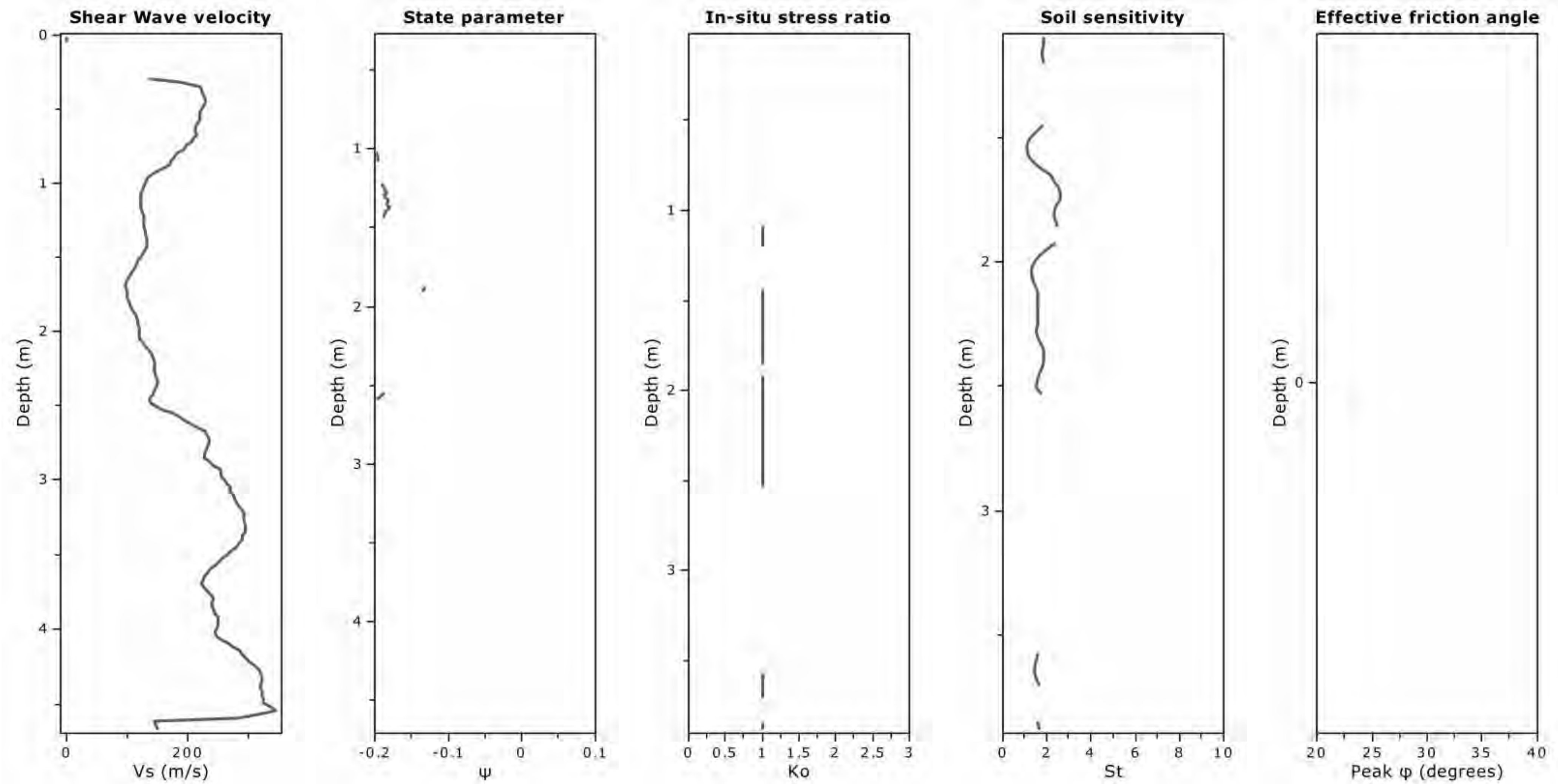
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data



Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

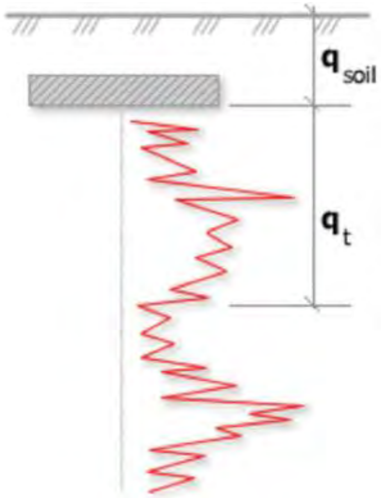
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

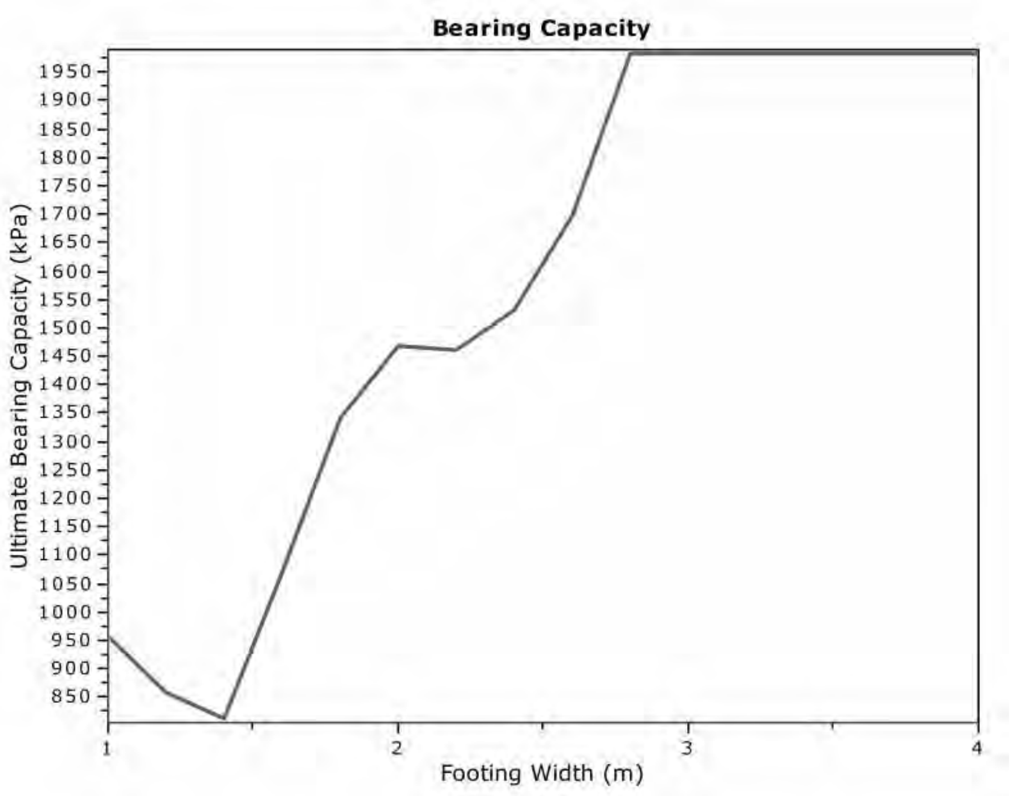
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	4.74	0.20	9.50	958.13
2	1.20	0.50	2.30	4.24	0.20	9.50	857.23
3	1.40	0.50	2.60	4.02	0.20	9.50	812.84
4	1.60	0.50	2.90	5.30	0.20	9.50	1068.90
5	1.80	0.50	3.20	6.65	0.20	9.50	1339.73
6	2.00	0.50	3.50	7.29	0.20	9.50	1466.51
7	2.20	0.50	3.80	7.25	0.20	9.50	1459.34
8	2.40	0.50	4.10	7.60	0.20	9.50	1530.20
9	2.60	0.50	4.40	8.44	0.20	9.50	1696.57
10	2.80	0.50	4.70	9.86	0.20	9.50	1982.22
11	3.00	0.50	5.00	9.86	0.20	9.50	1982.22
12	3.20	0.50	5.30	9.86	0.20	9.50	1982.22
13	3.40	0.50	5.60	9.86	0.20	9.50	1982.22
14	3.60	0.50	5.90	9.86	0.20	9.50	1982.22
15	3.80	0.50	6.20	9.86	0.20	9.50	1982.22
16	4.00	0.50	6.50	9.86	0.20	9.50	1982.22

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot +7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

• Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5 th Edition, November 2012

• Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

• N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

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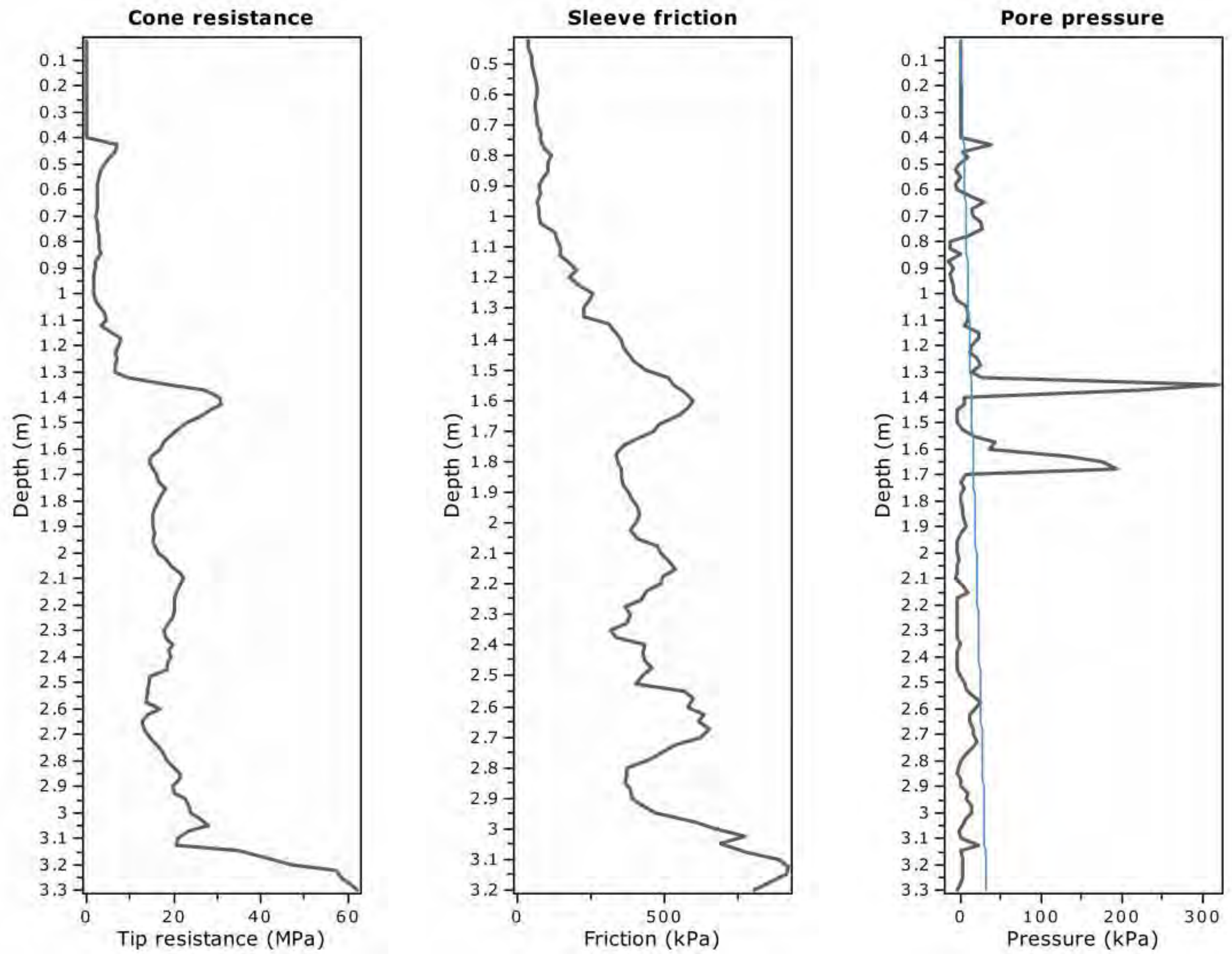
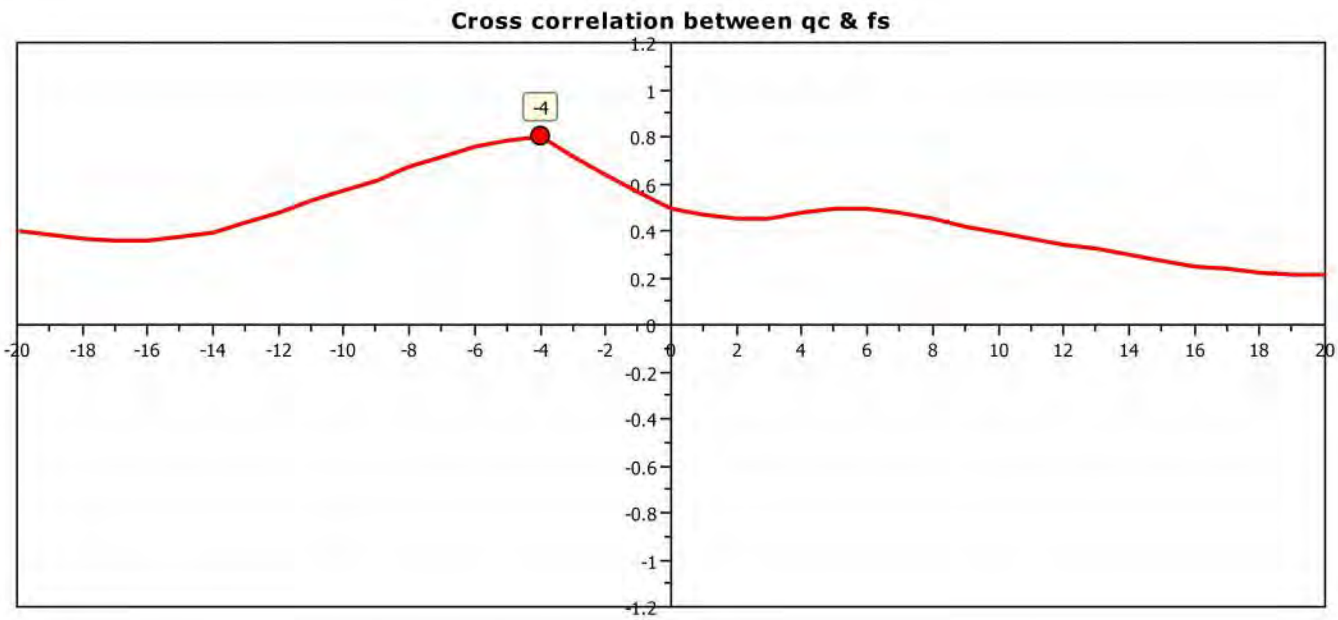
CPT: CPT-05
Total depth: 3.30 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

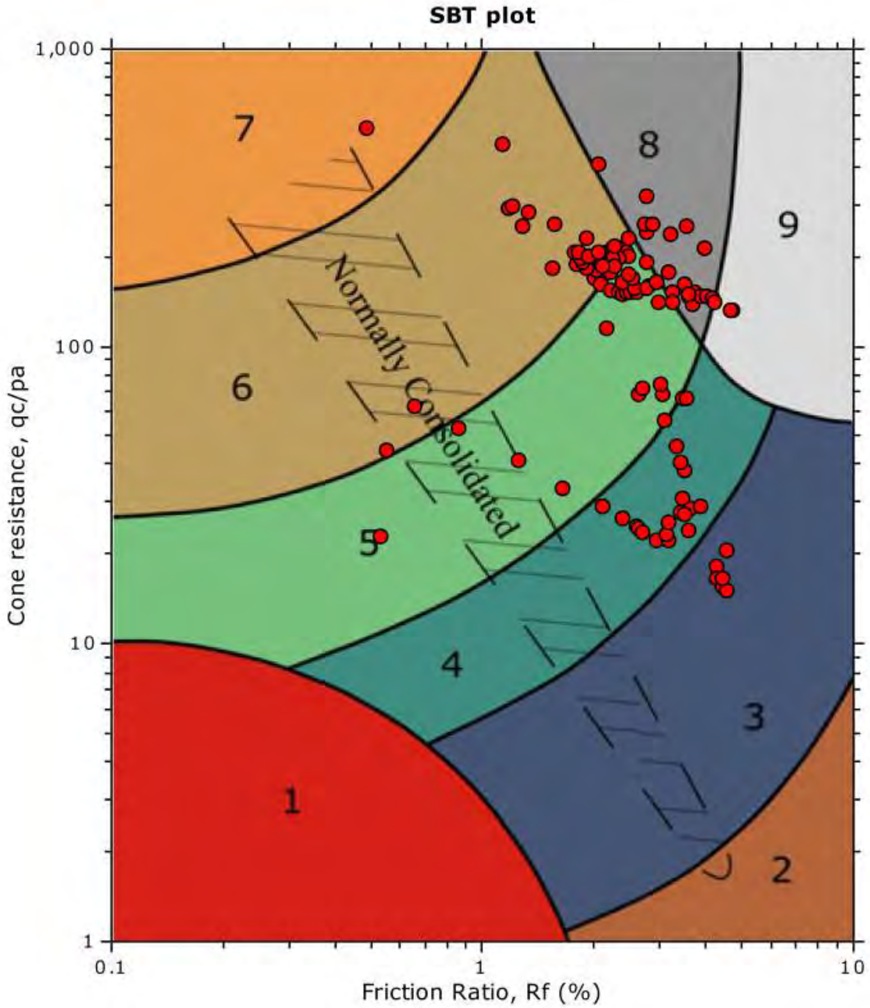
Project:
Location:

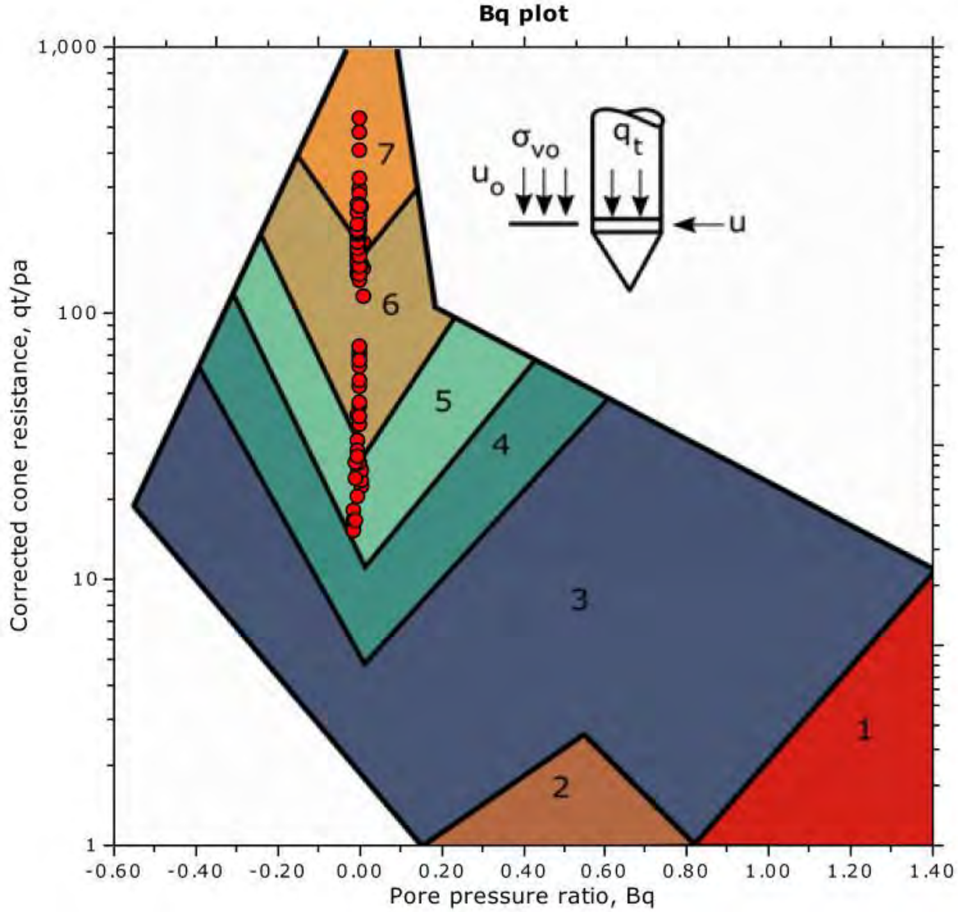


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots

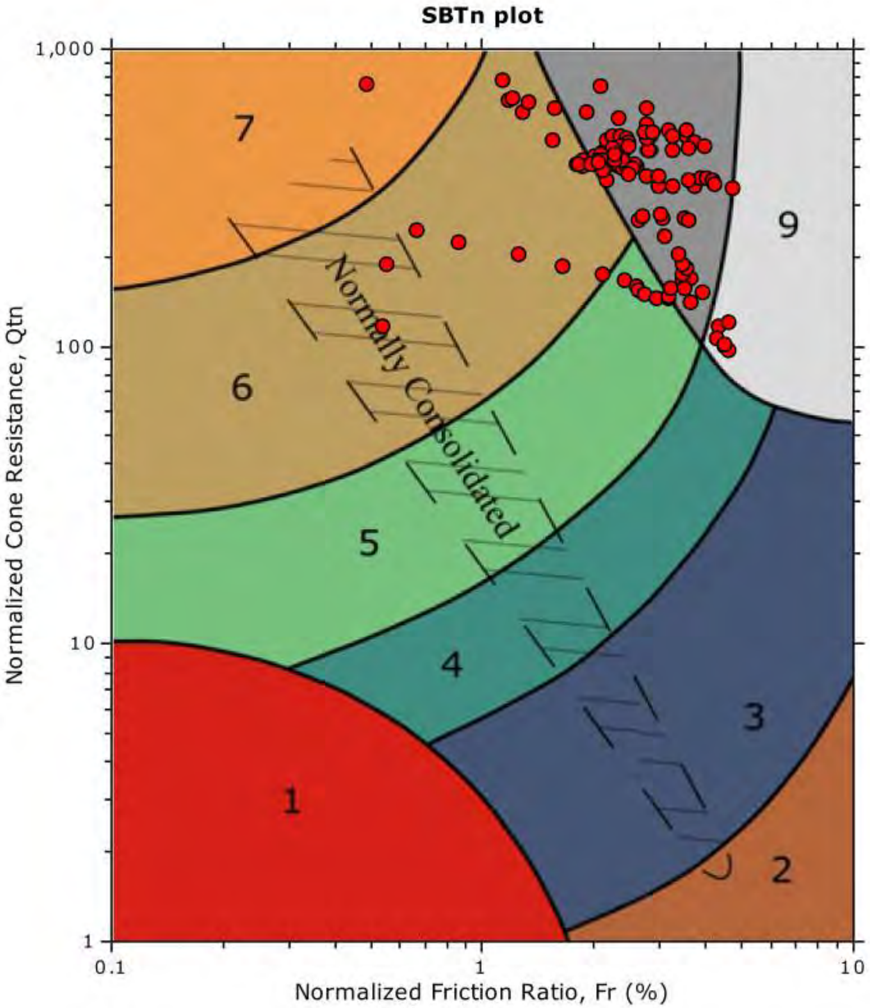


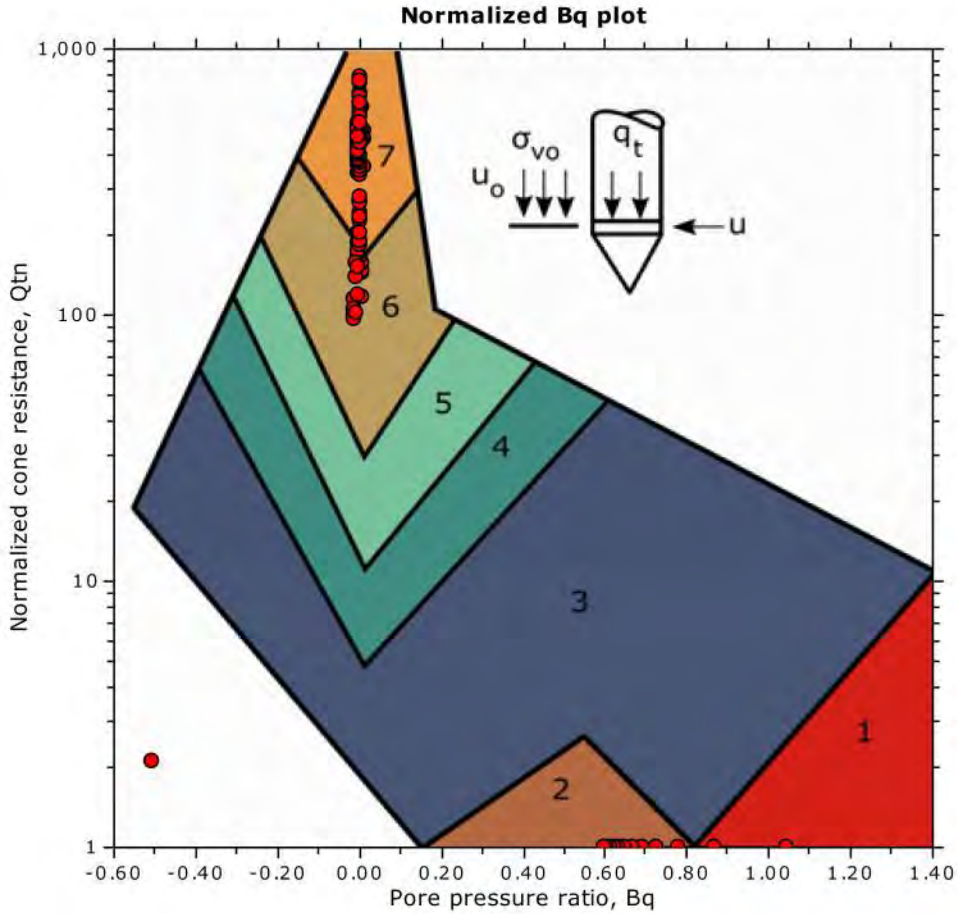


SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

SBT - Bq plots (normalized)

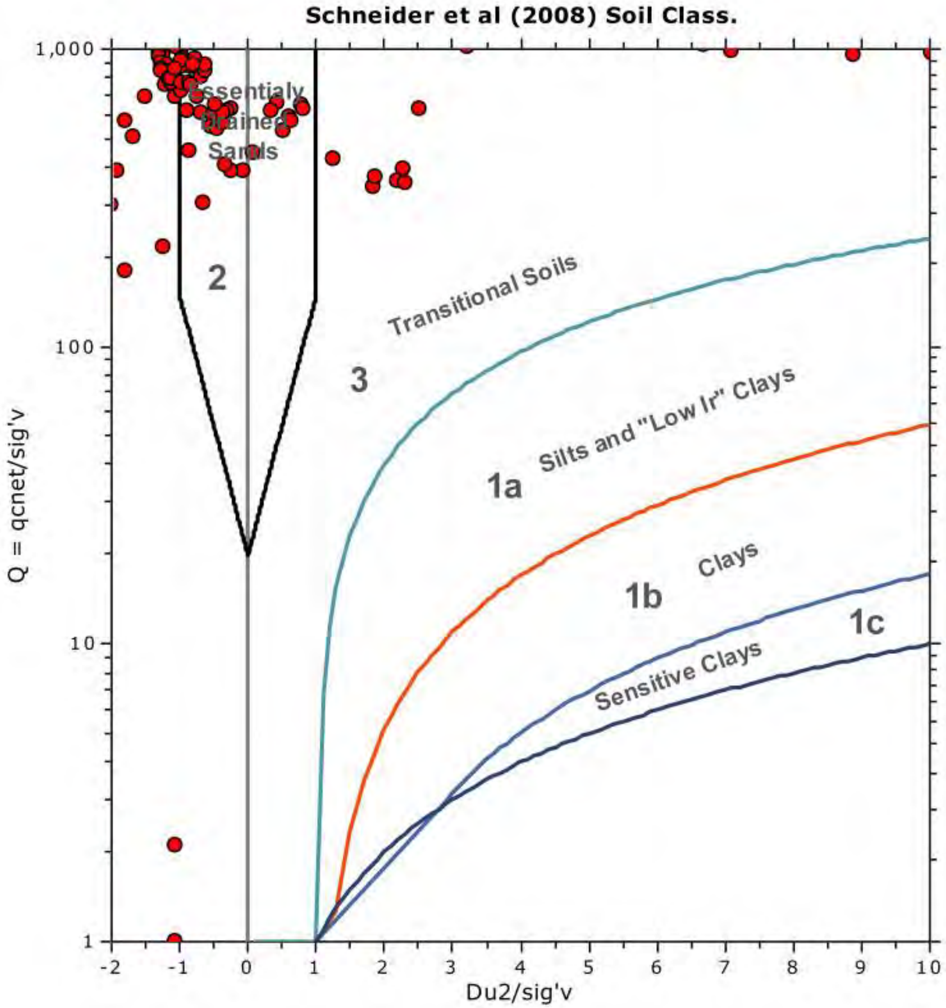


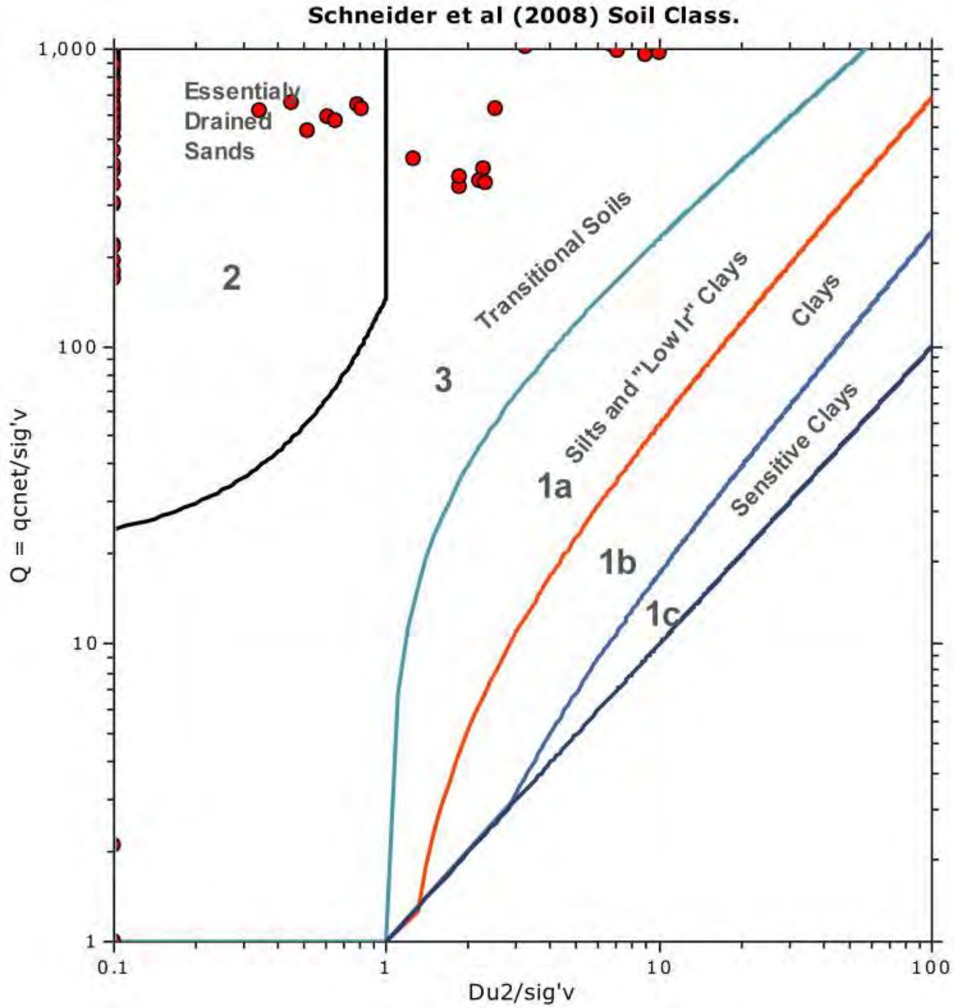


SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Bq plots (Schneider)





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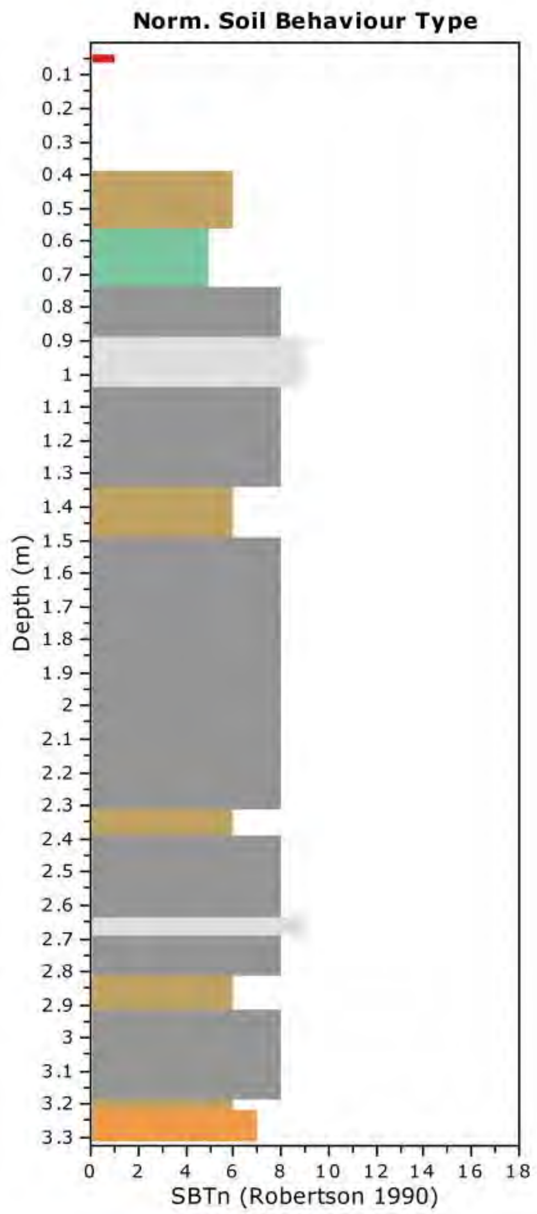
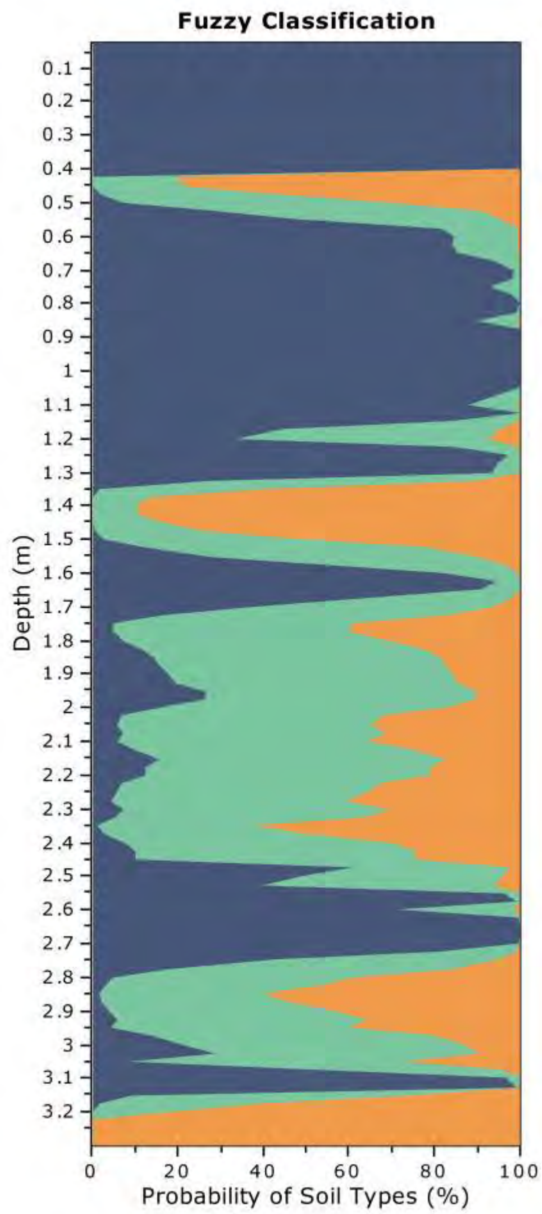
CPT: CPT-05
Total depth: 3.30 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:





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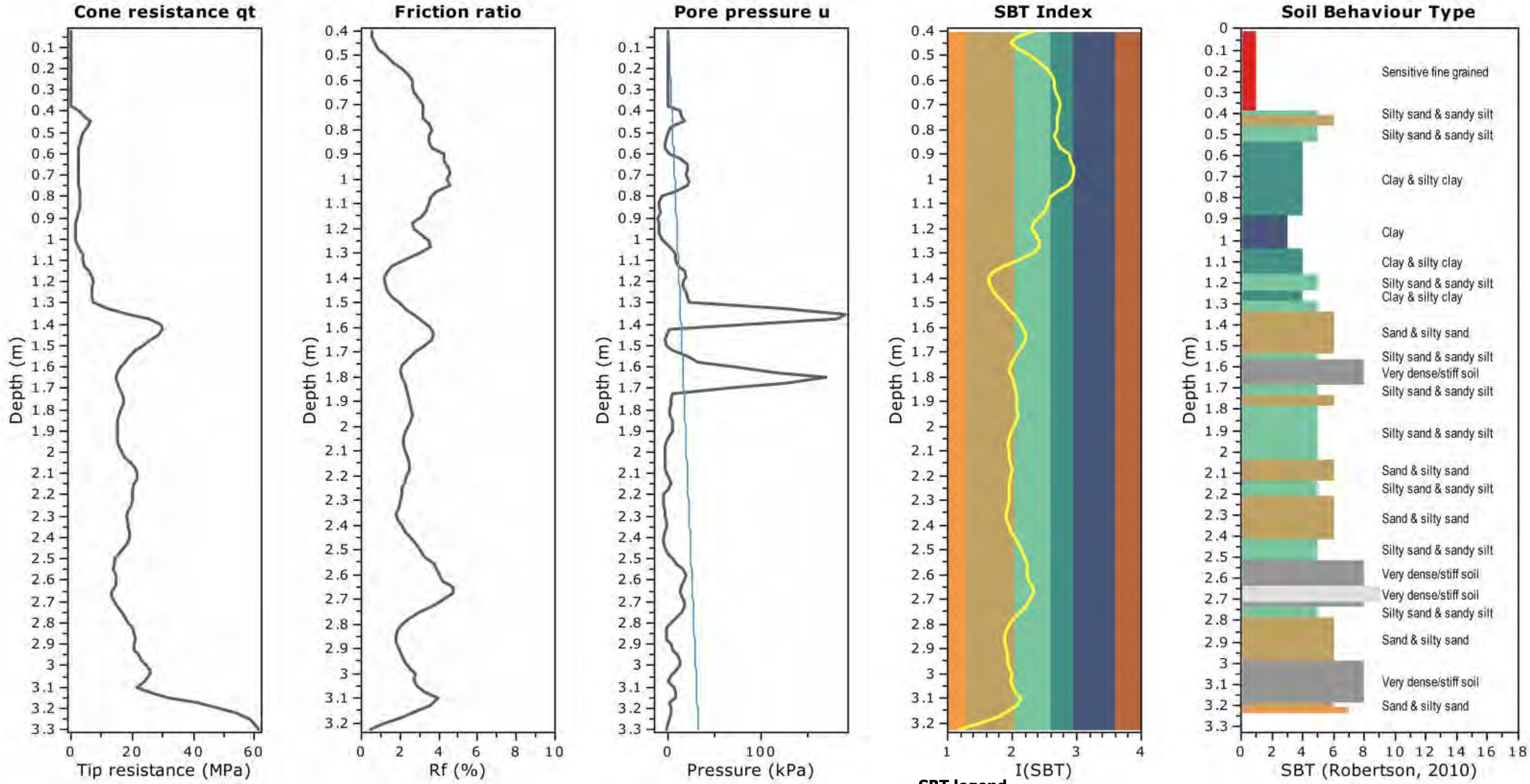
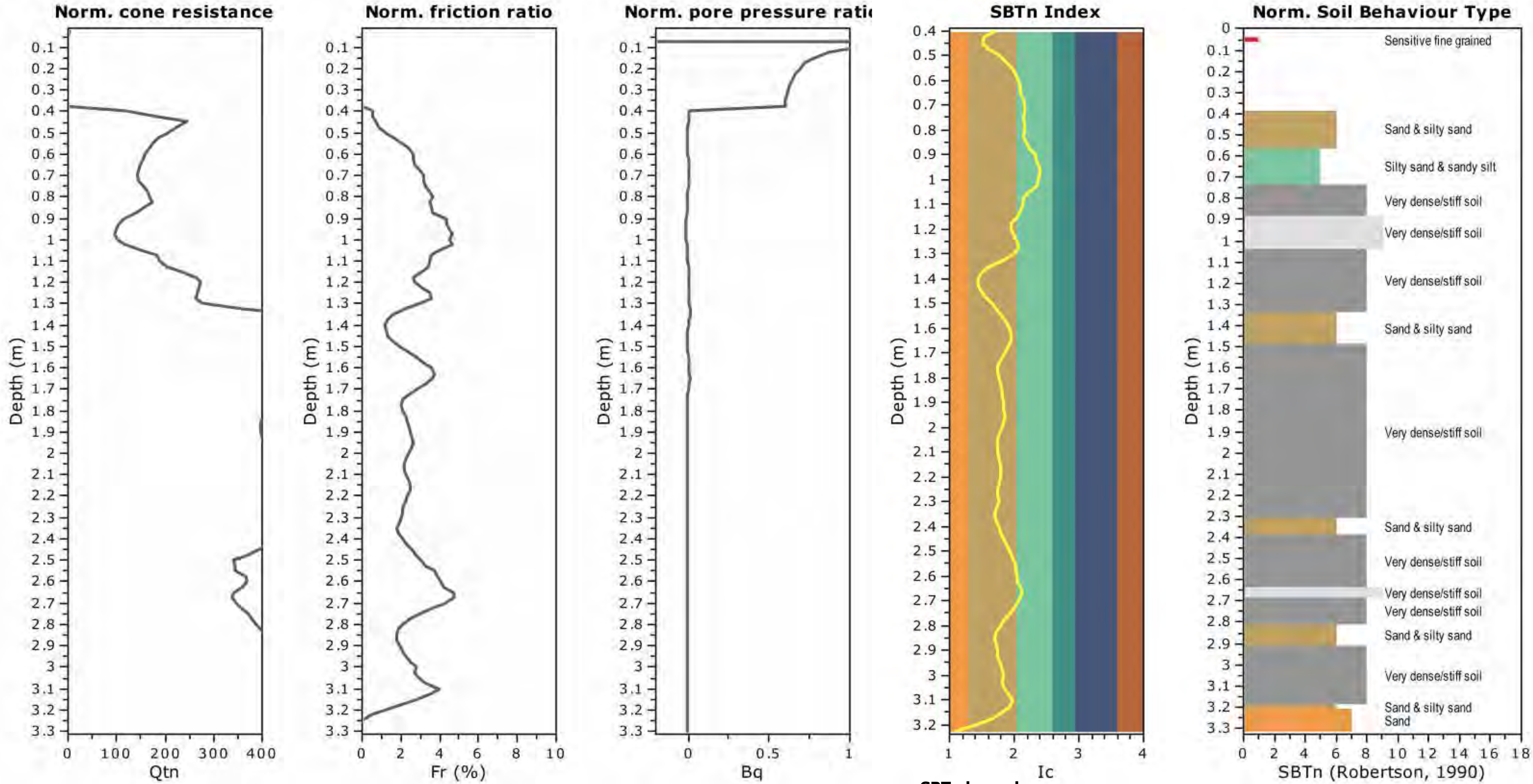
Total depth: 3.30 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained



SBTn legend

- 1. Sensitive fine grained
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CPT: CPT-05
Total depth: 3.30 m, Date: 4/18/2022
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Cone Type:
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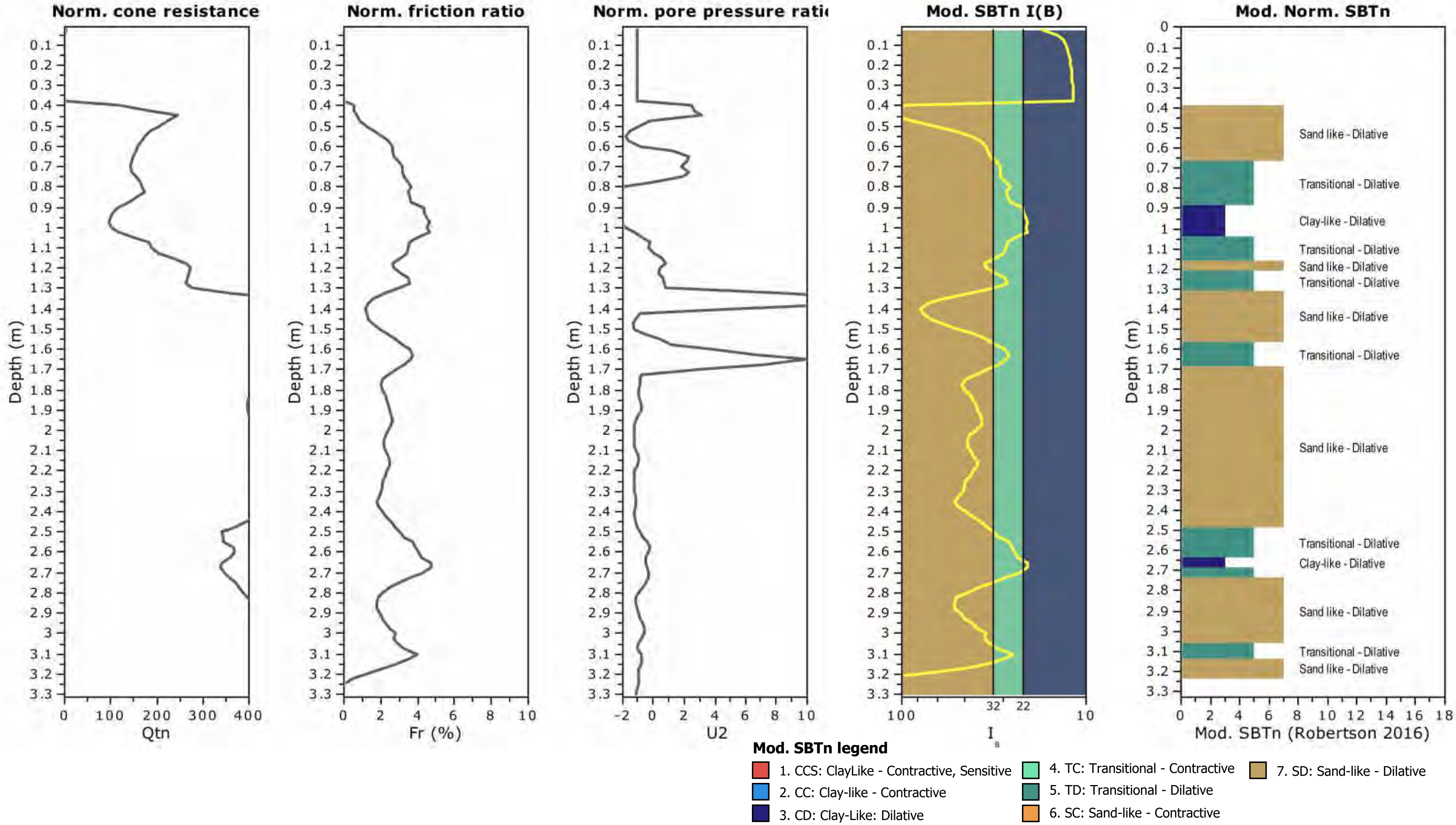
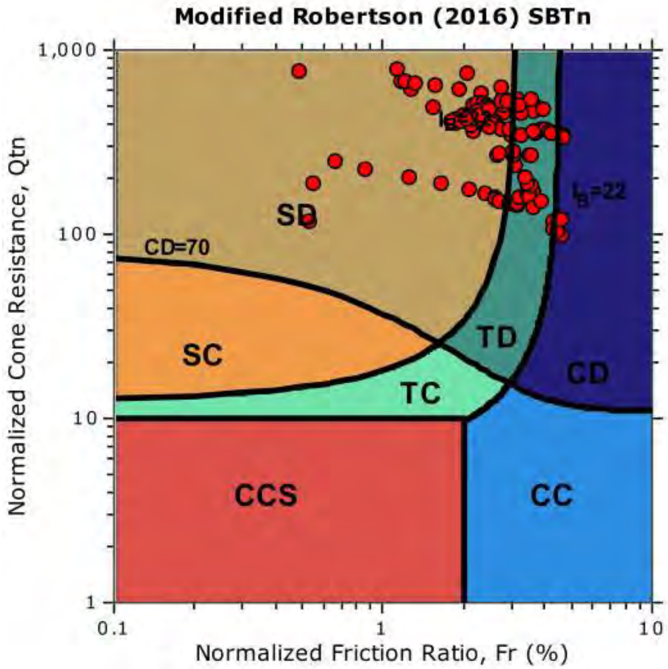
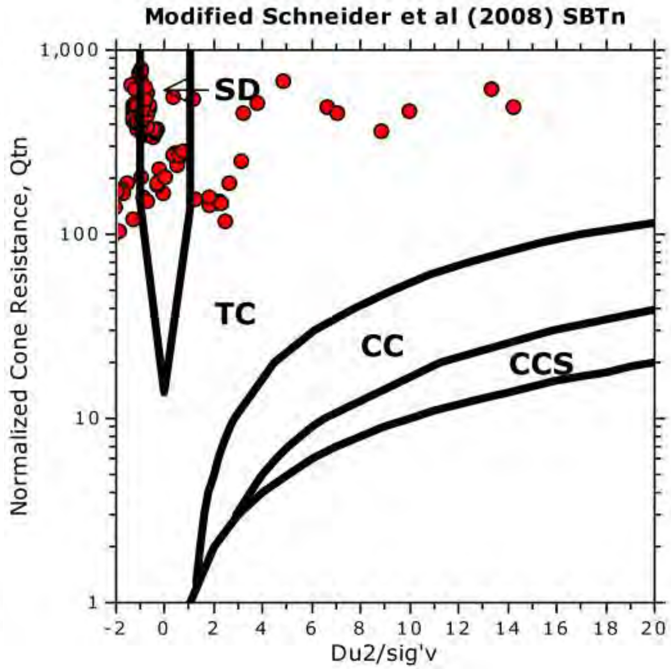
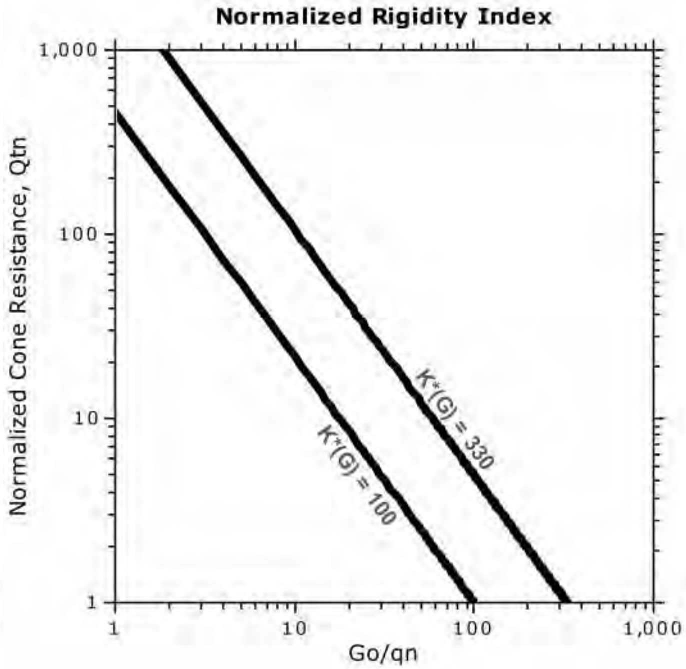
Project:
Location:



Updated SBTn plots

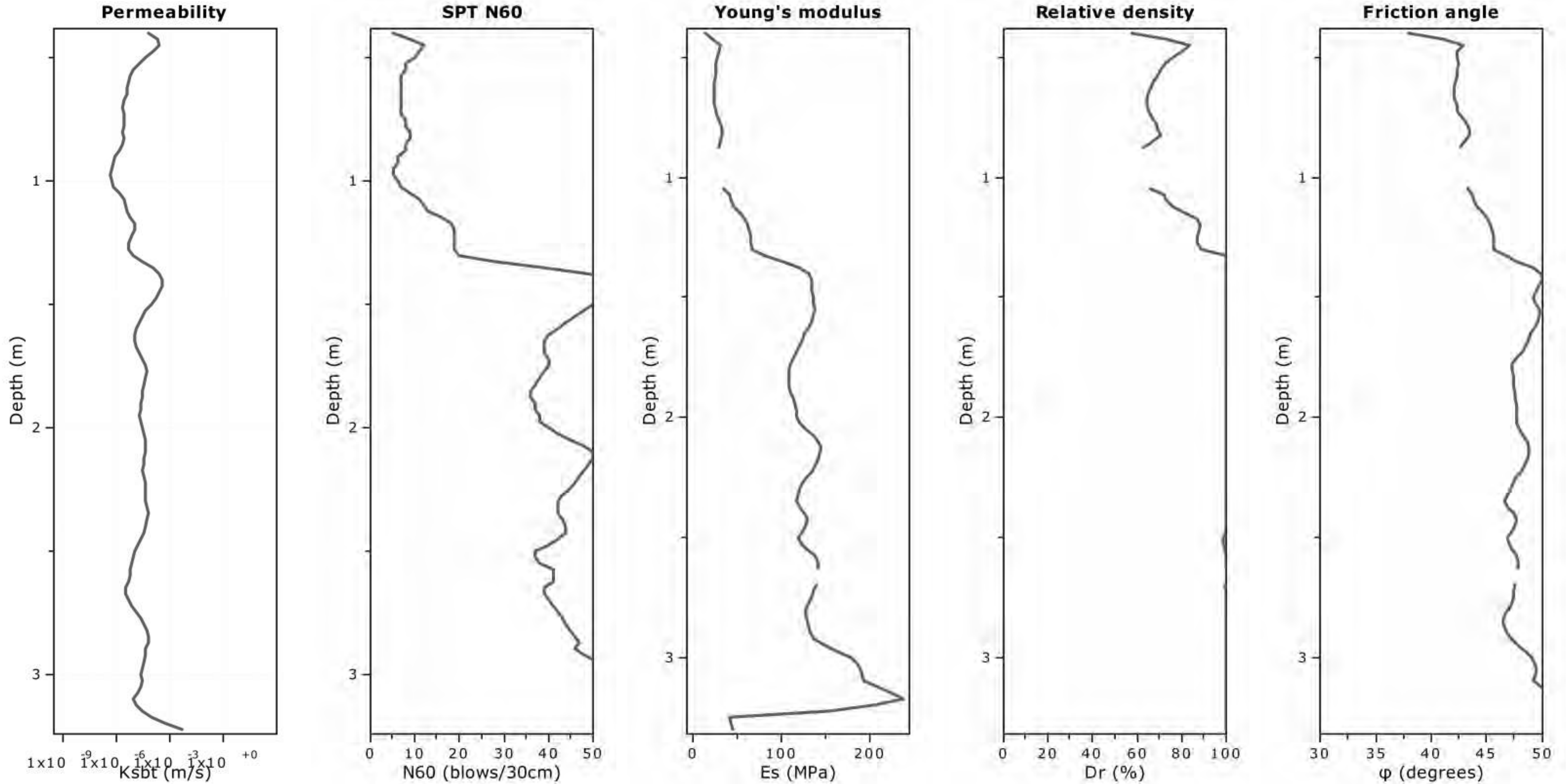






CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)

User defined estimation data

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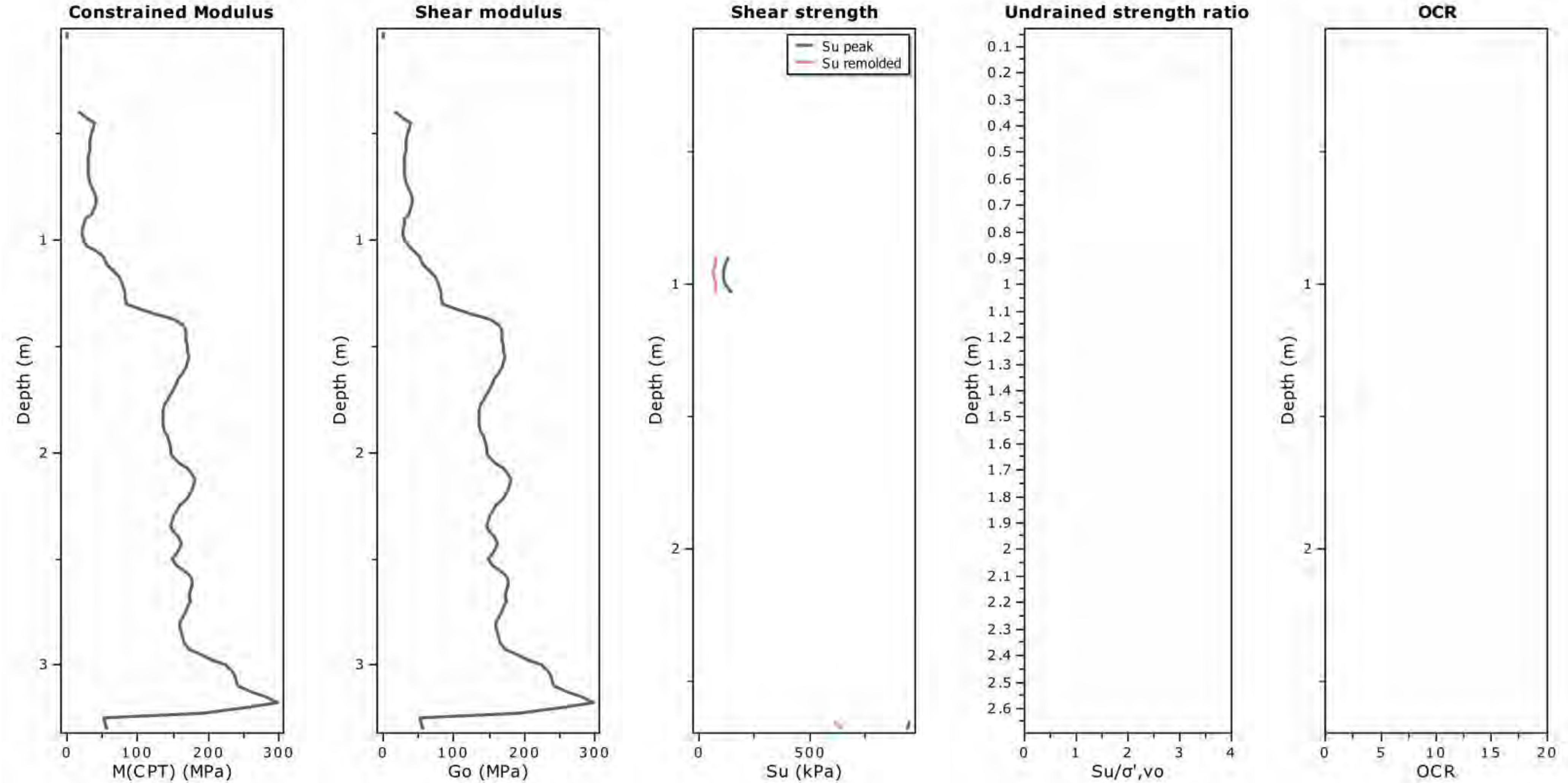
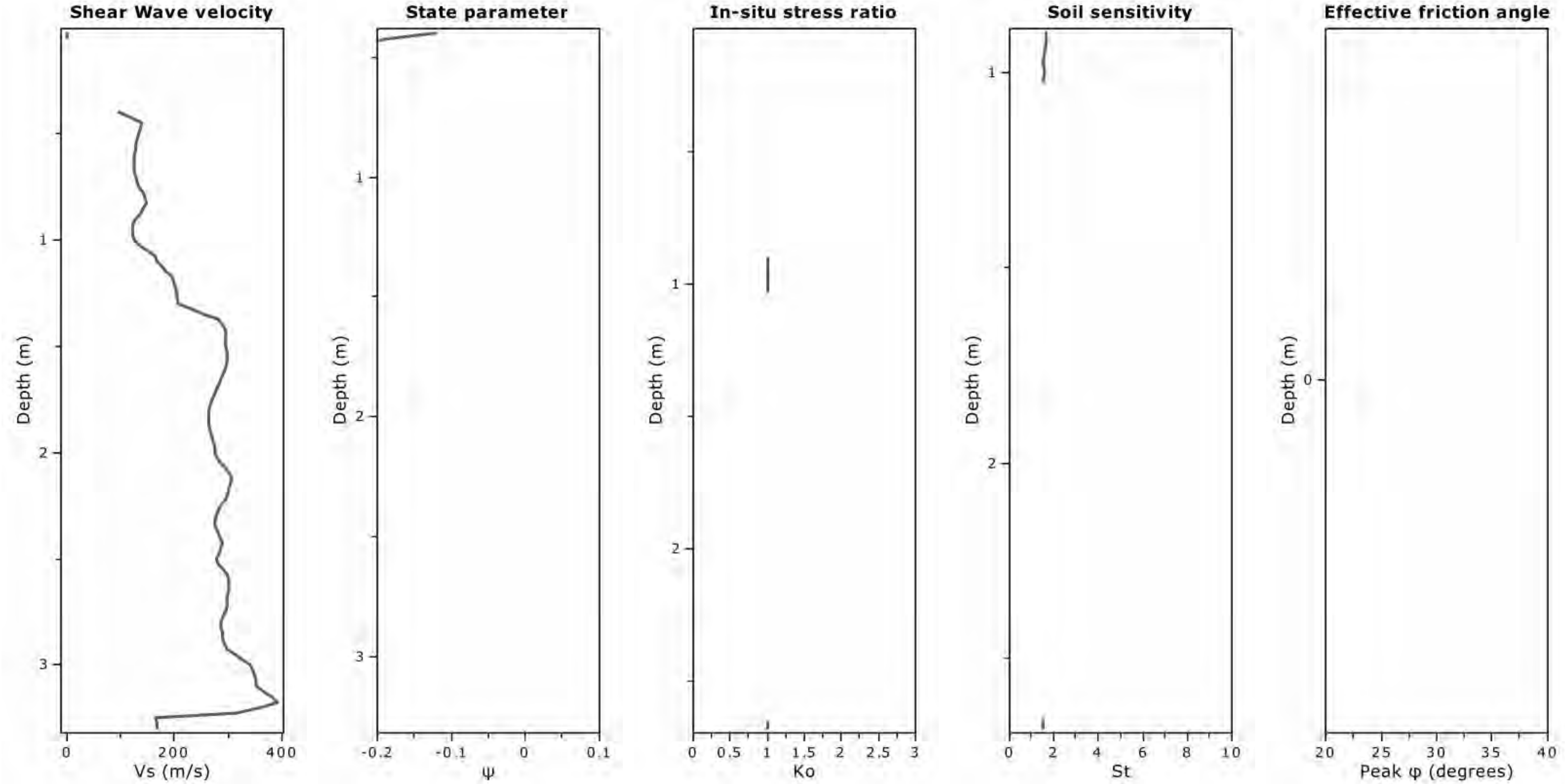
CPT: CPT-05
Total depth: 3.30 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:



Calculation parameters
Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

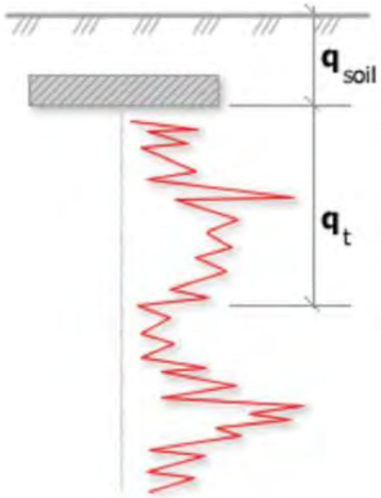
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

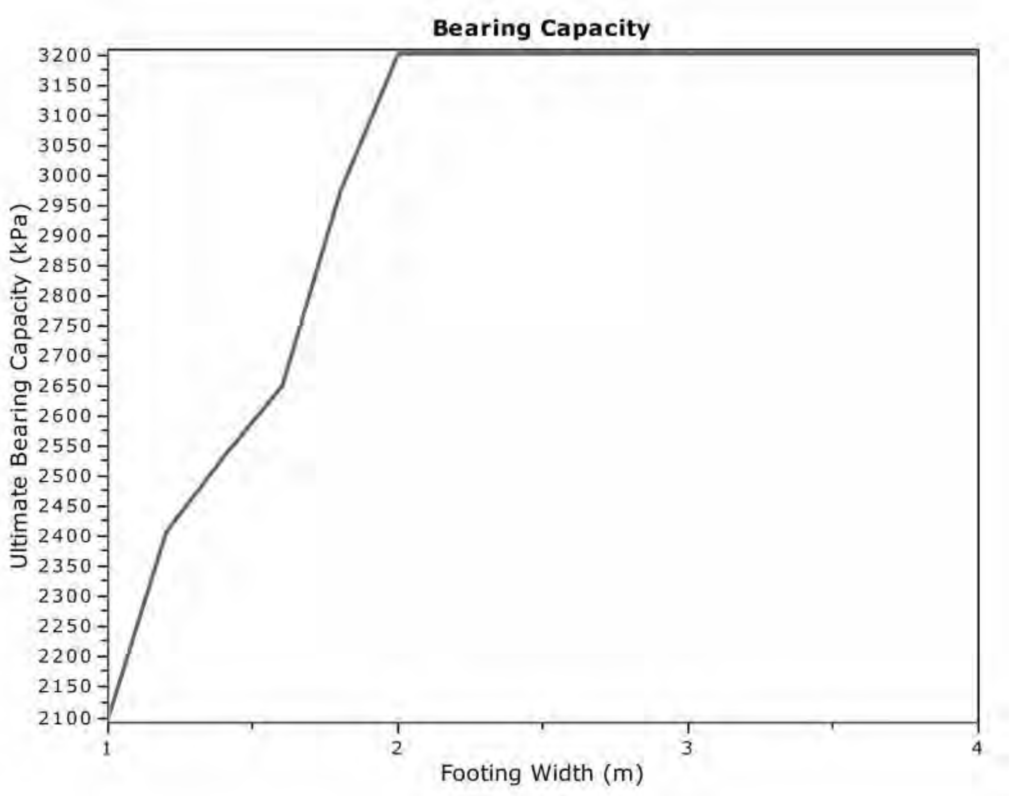
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	10.44	0.20	9.50	2098.36
2	1.20	0.50	2.30	12.00	0.20	9.50	2408.53
3	1.40	0.50	2.60	12.62	0.20	9.50	2533.53
4	1.60	0.50	2.90	13.20	0.20	9.50	2649.39
5	1.80	0.50	3.20	14.82	0.20	9.50	2973.04
6	2.00	0.50	3.50	15.96	0.20	9.50	3201.78
7	2.20	0.50	3.80	15.96	0.20	9.50	3201.78
8	2.40	0.50	4.10	15.96	0.20	9.50	3201.78
9	2.60	0.50	4.40	15.96	0.20	9.50	3201.78
10	2.80	0.50	4.70	15.96	0.20	9.50	3201.78
11	3.00	0.50	5.00	15.96	0.20	9.50	3201.78
12	3.20	0.50	5.30	15.96	0.20	9.50	3201.78
13	3.40	0.50	5.60	15.96	0.20	9.50	3201.78
14	3.60	0.50	5.90	15.96	0.20	9.50	3201.78
15	3.80	0.50	6.20	15.96	0.20	9.50	3201.78
16	4.00	0.50	6.50	15.96	0.20	9.50	3201.78

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_o = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

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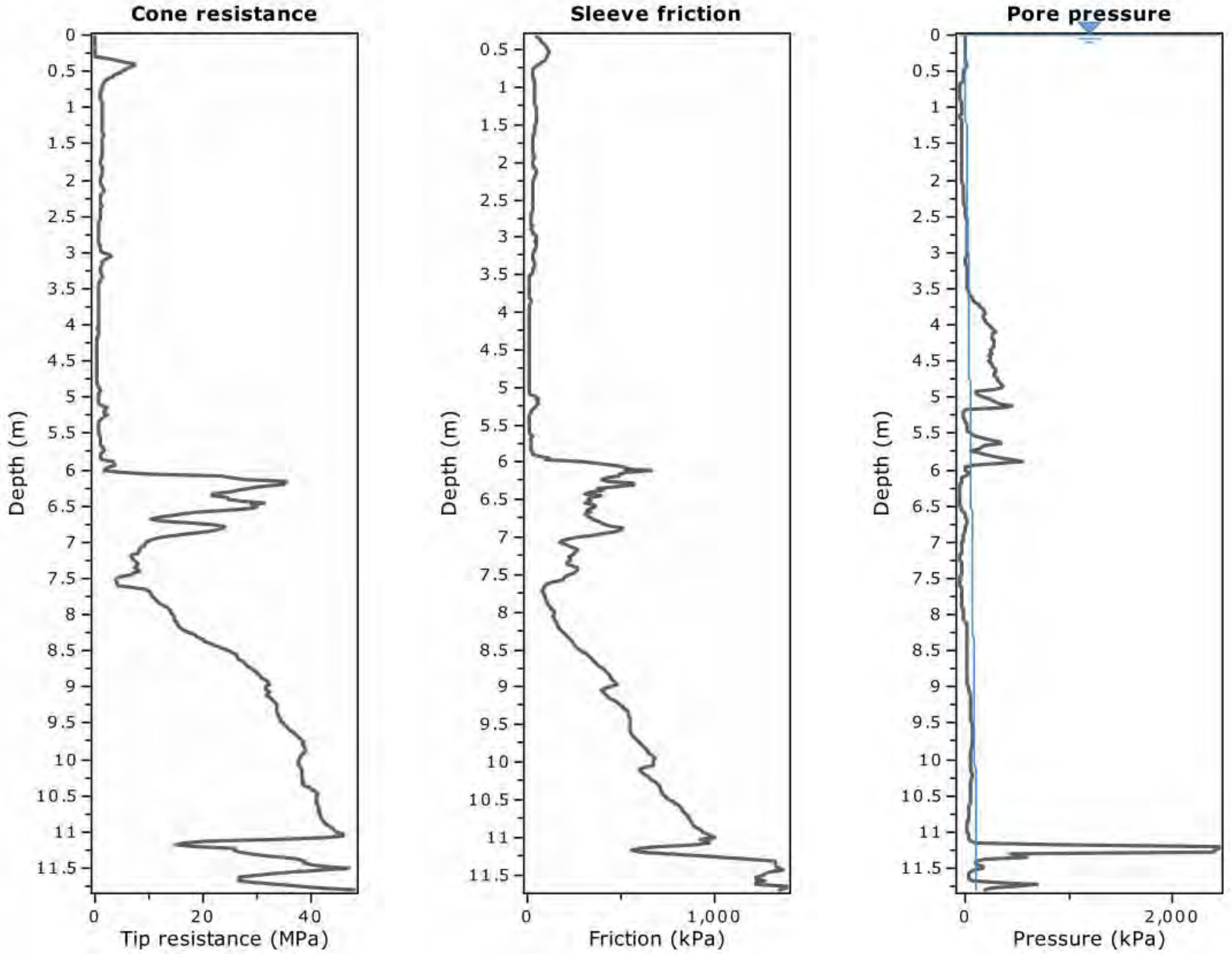
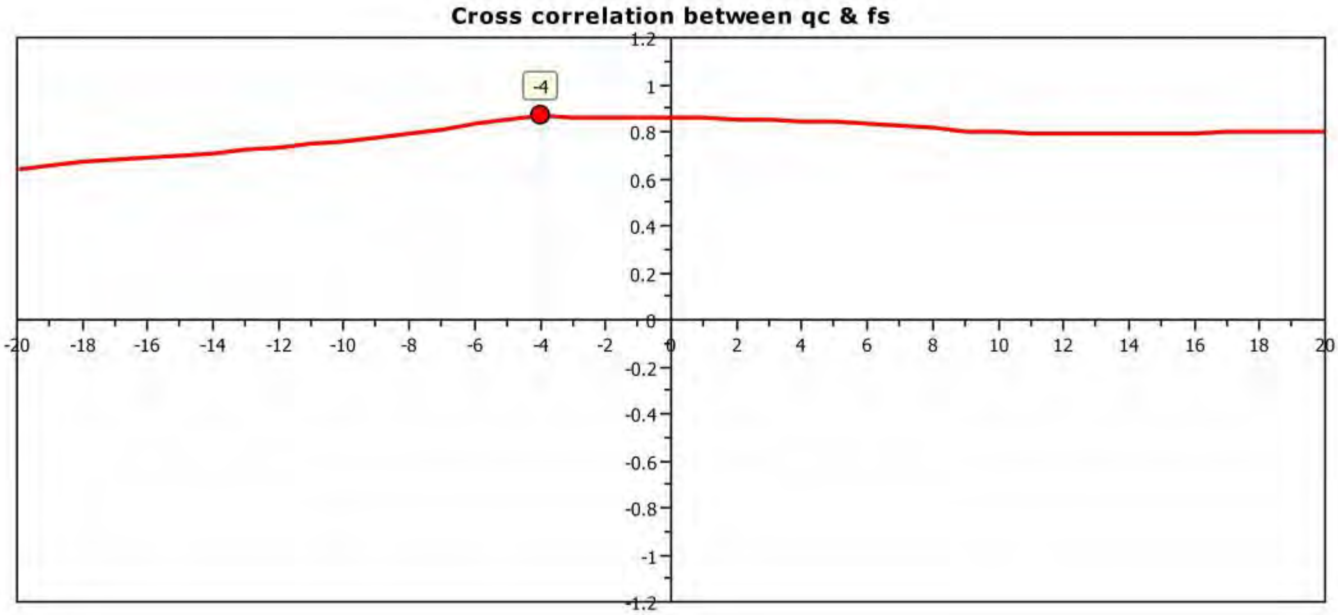
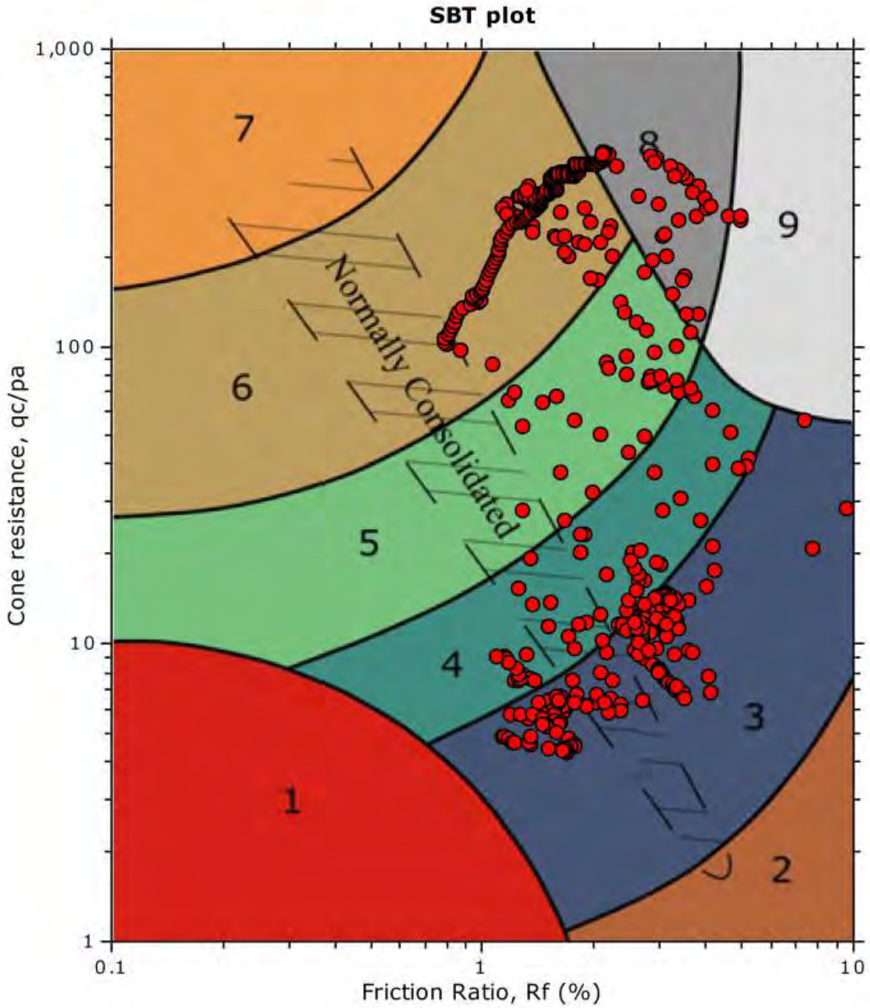
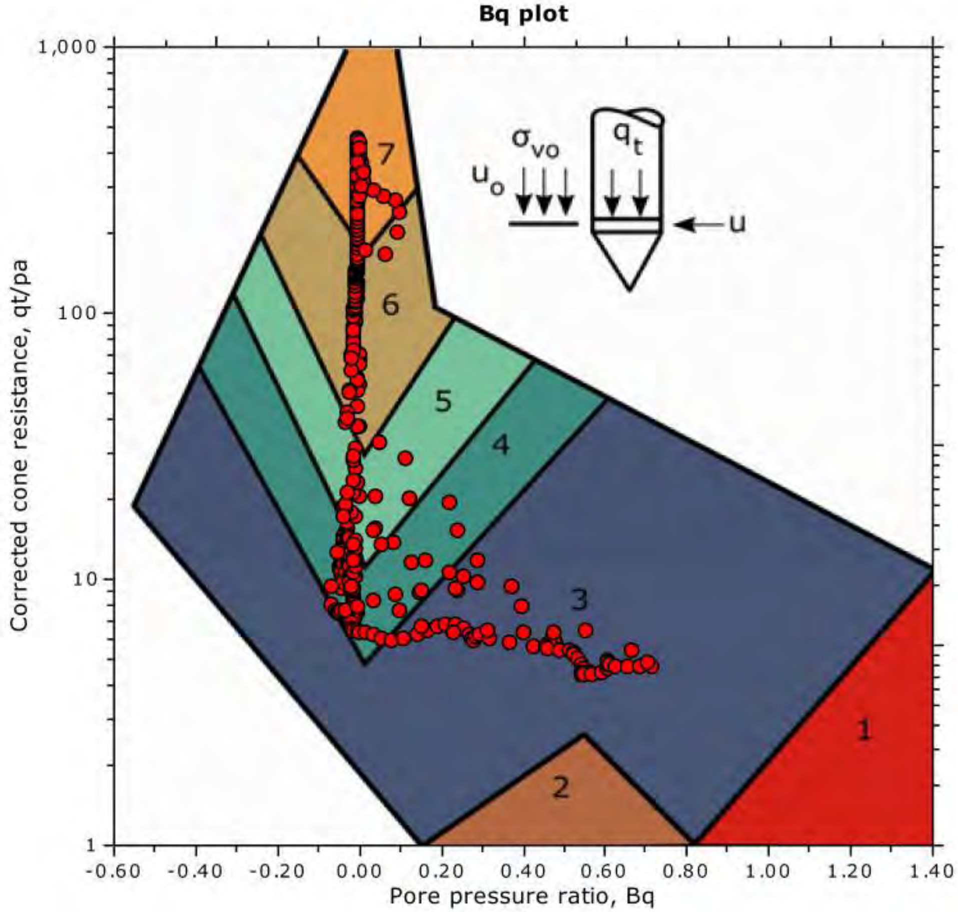


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots

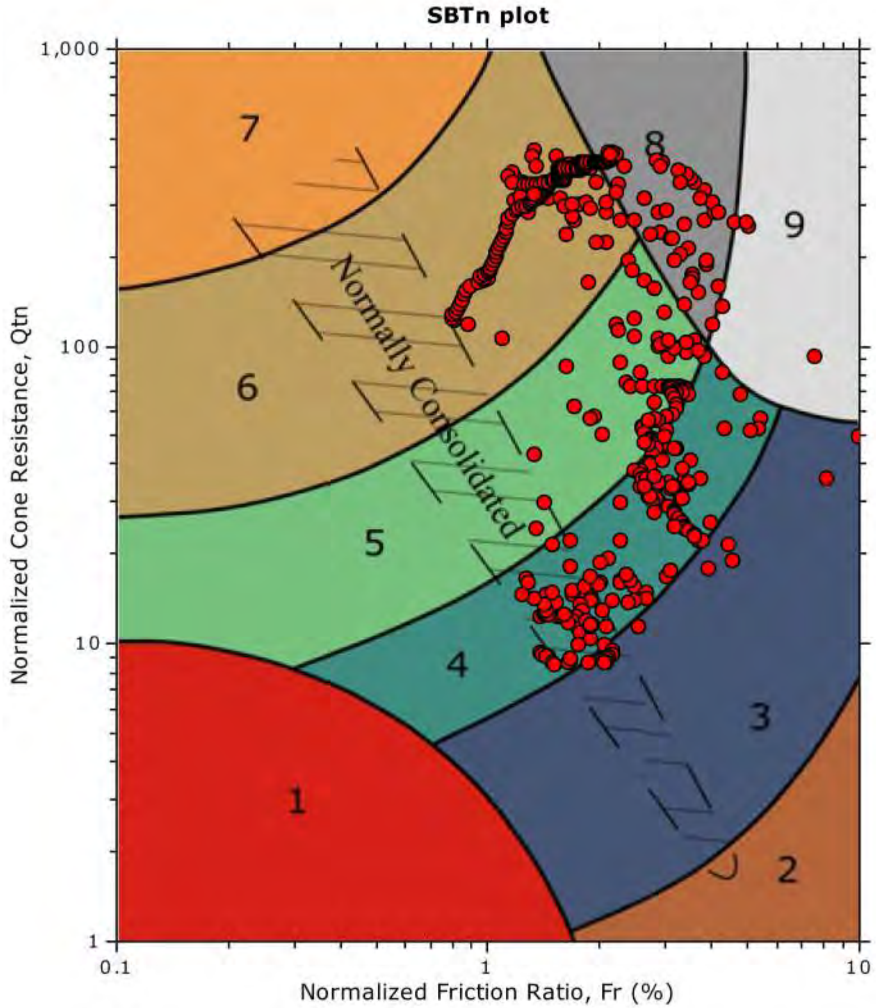


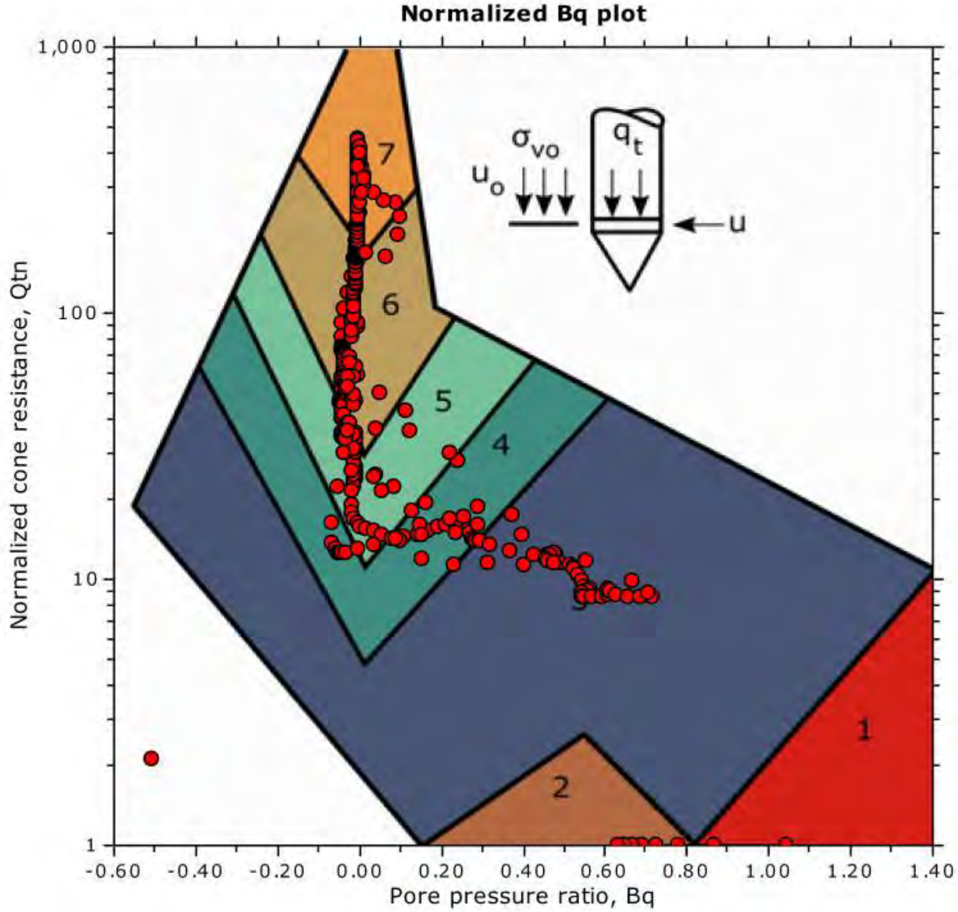


SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

SBT - Bq plots (normalized)





SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravelly sand to sand
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- 9. Very stiff fine grained

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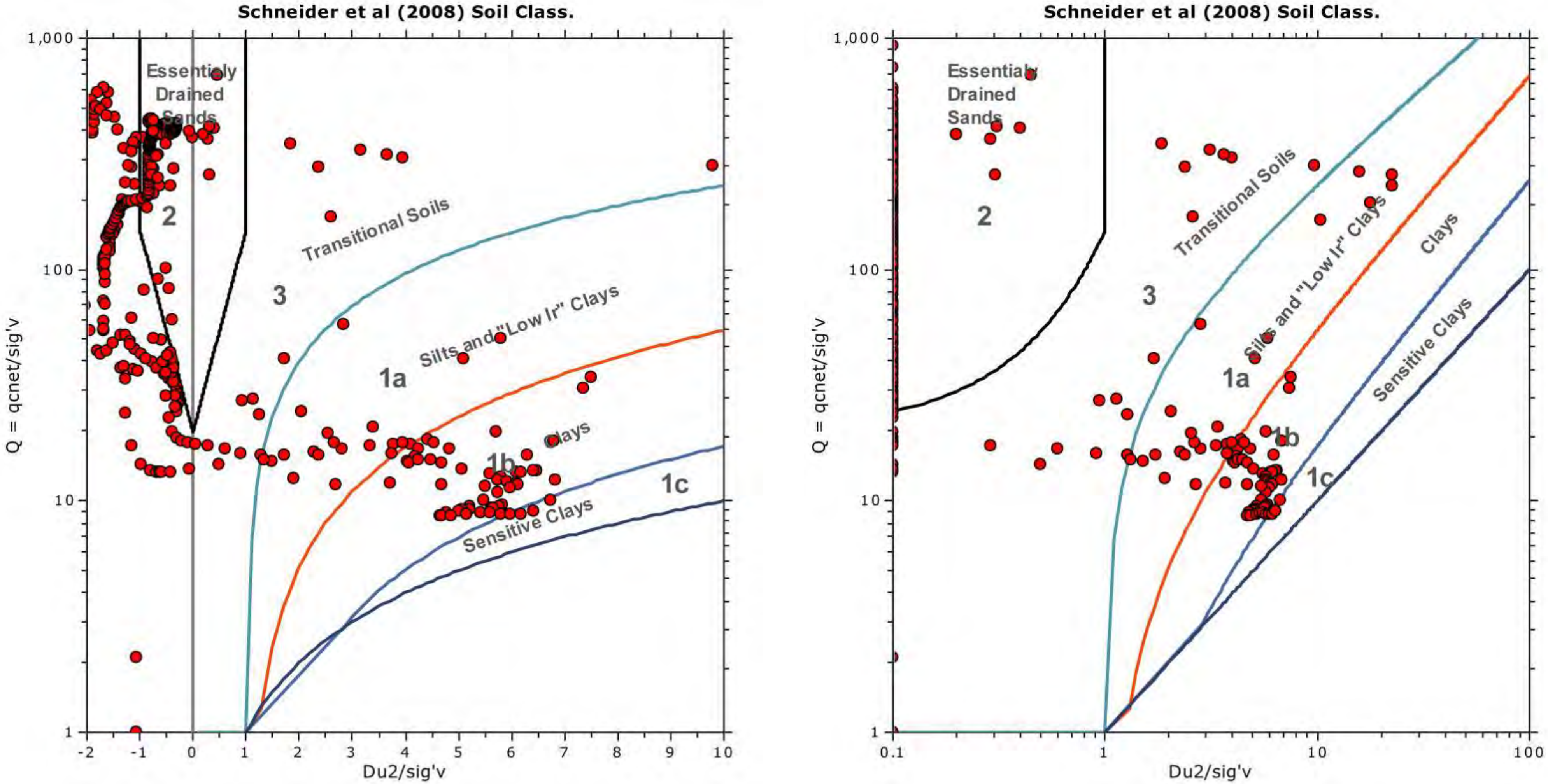
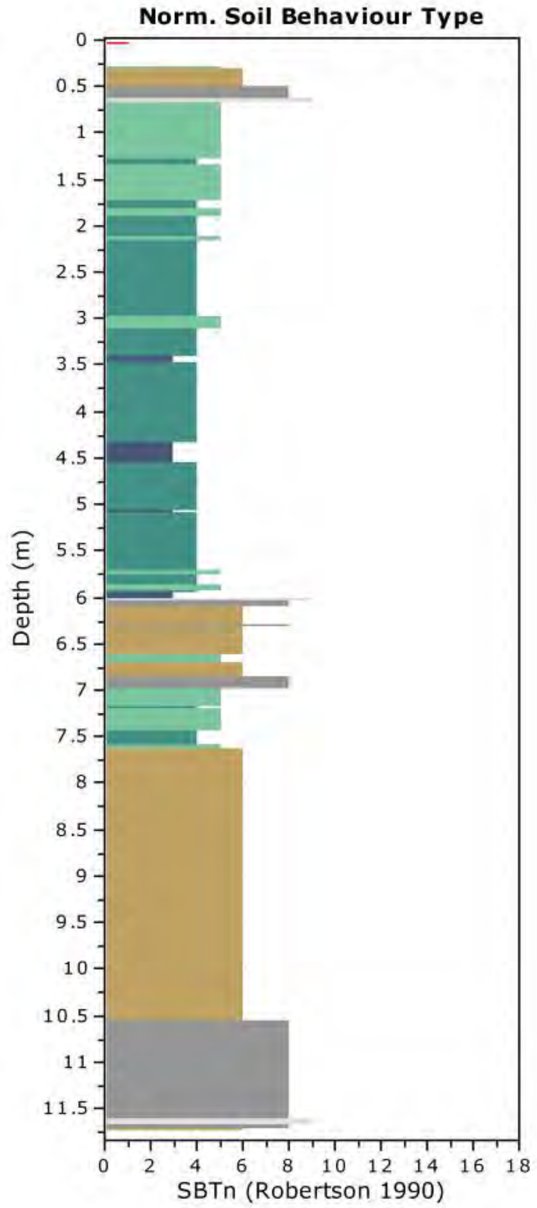
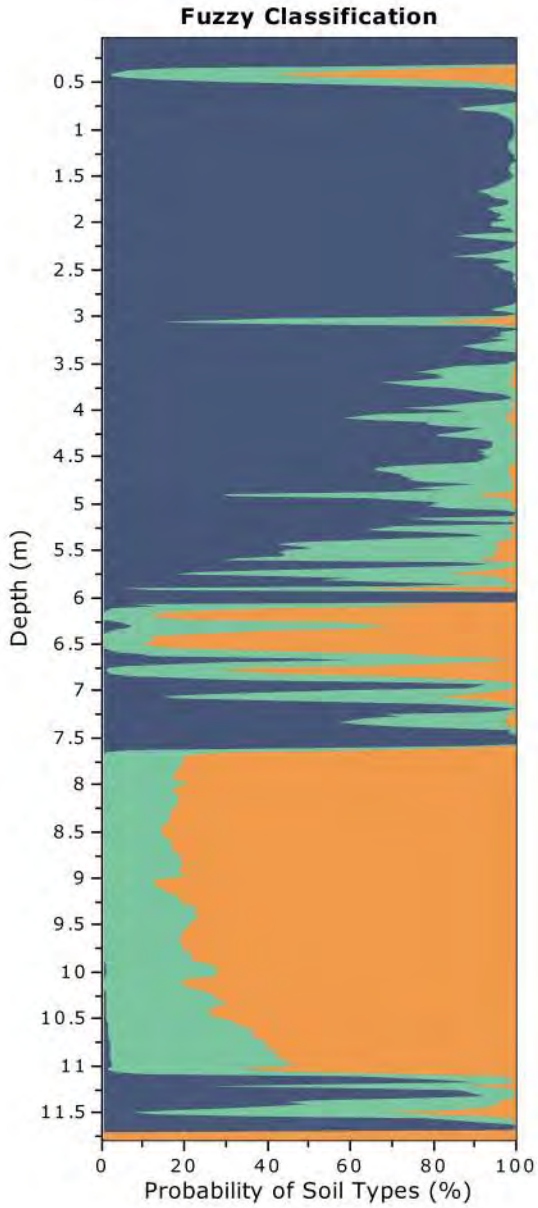
CPT: Sheet1
Total depth: 11.80 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

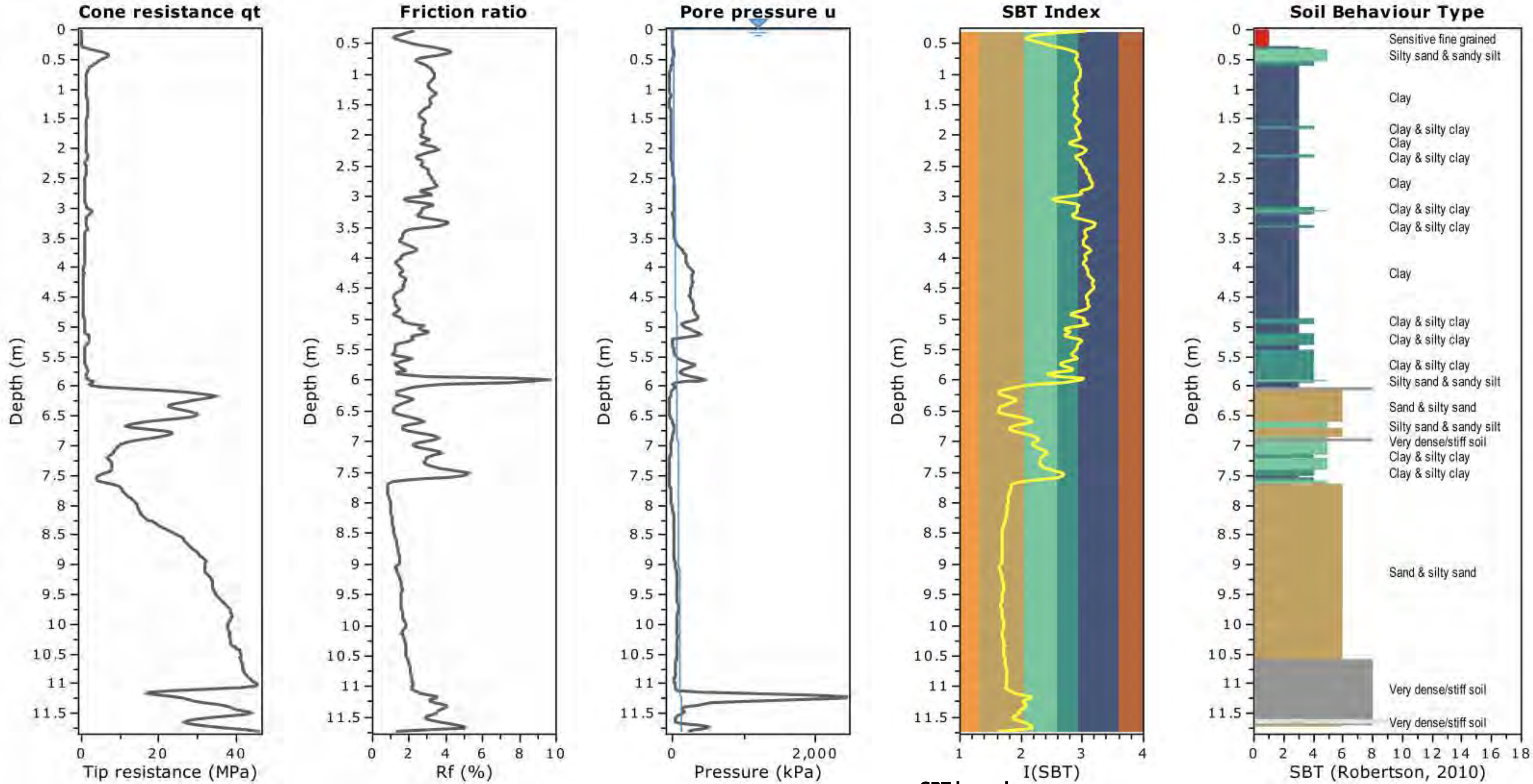
Project:
Location:

Bq plots (Schneider)




Norm. Soil Behaviour Type
Depth (m)
SBTn (Robertson 1990)


Fuzzy Classification
Depth (m)
Probability of Soil Types (%)



SBT legend

1. Sensitive fine grained
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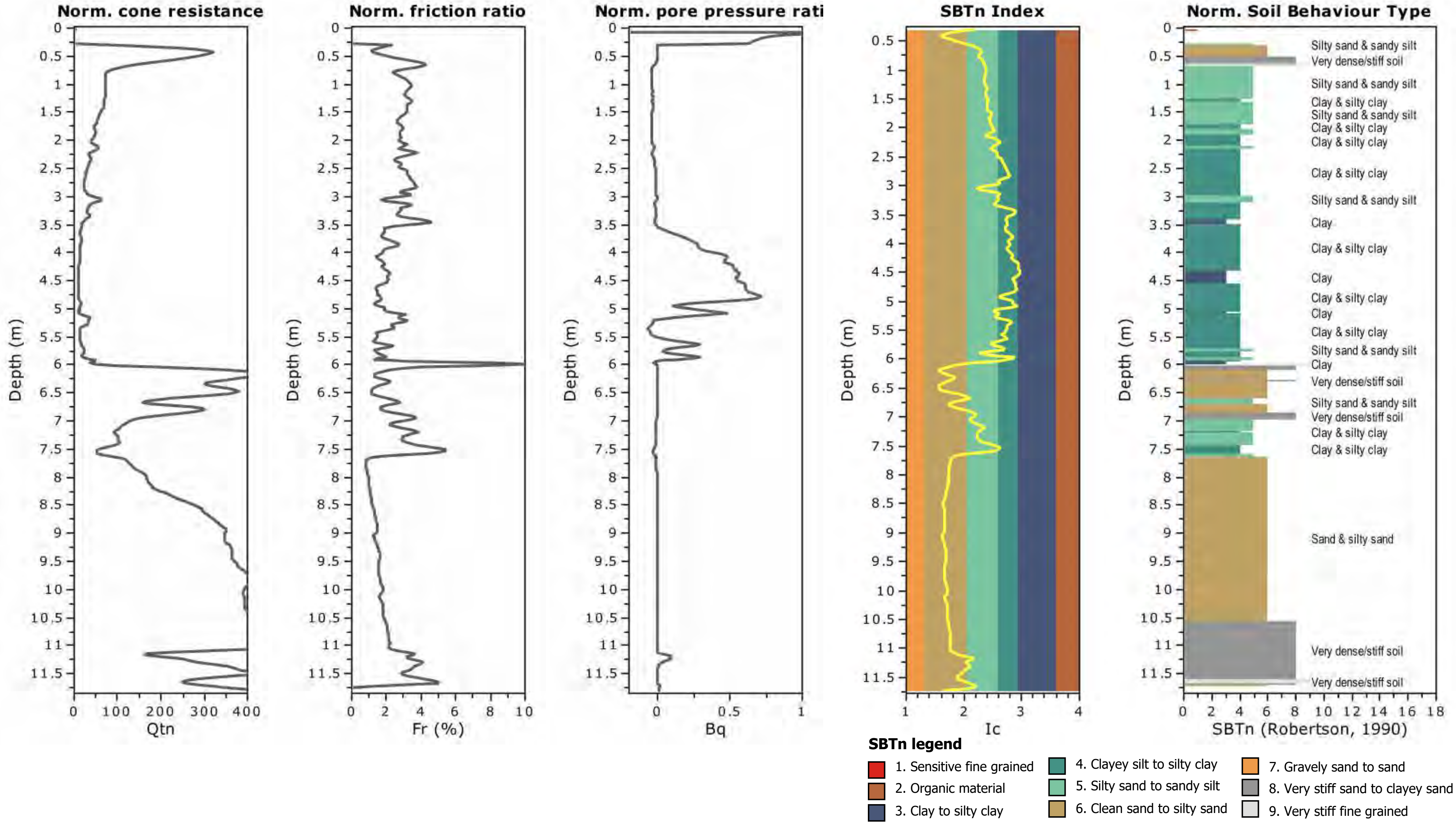
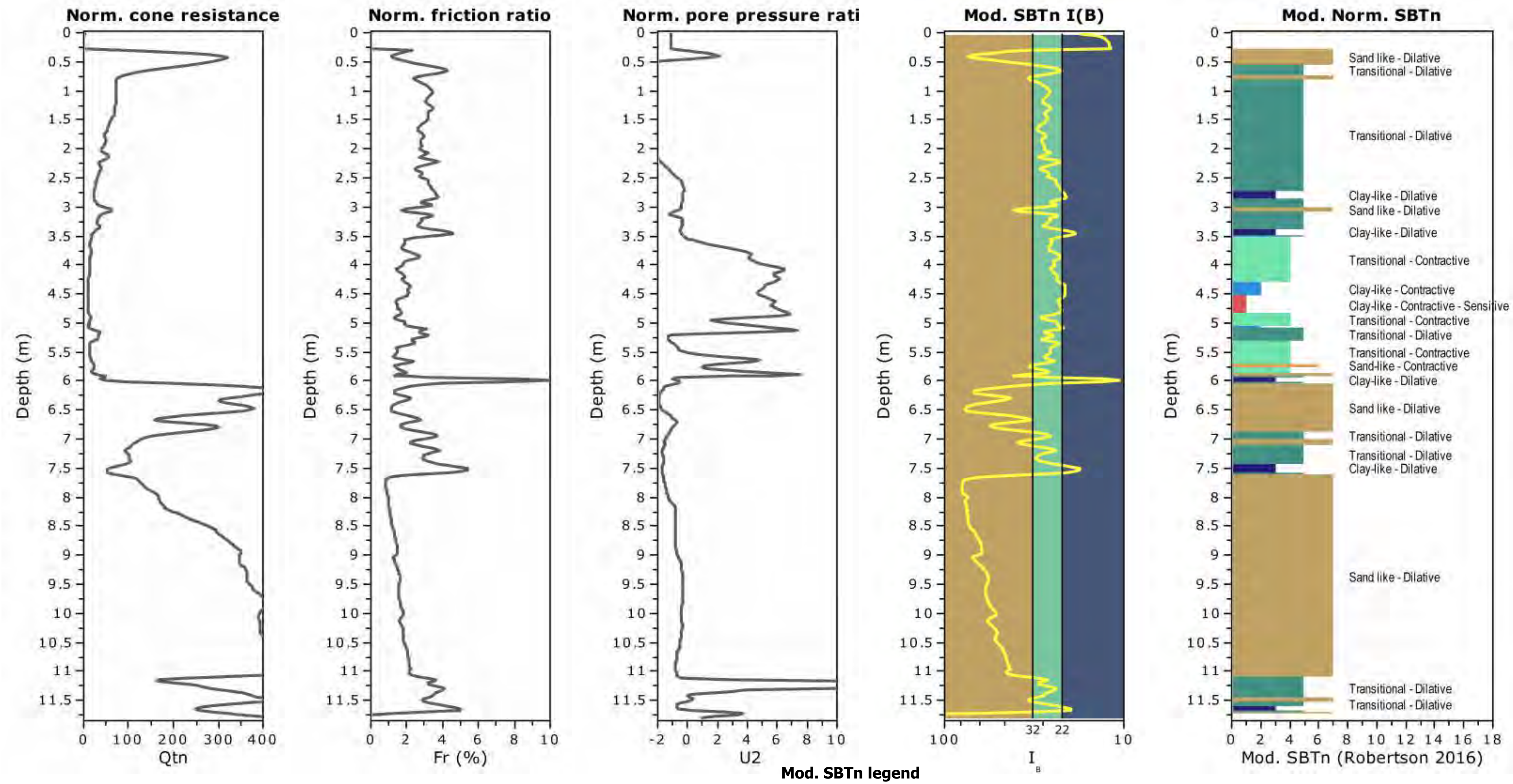


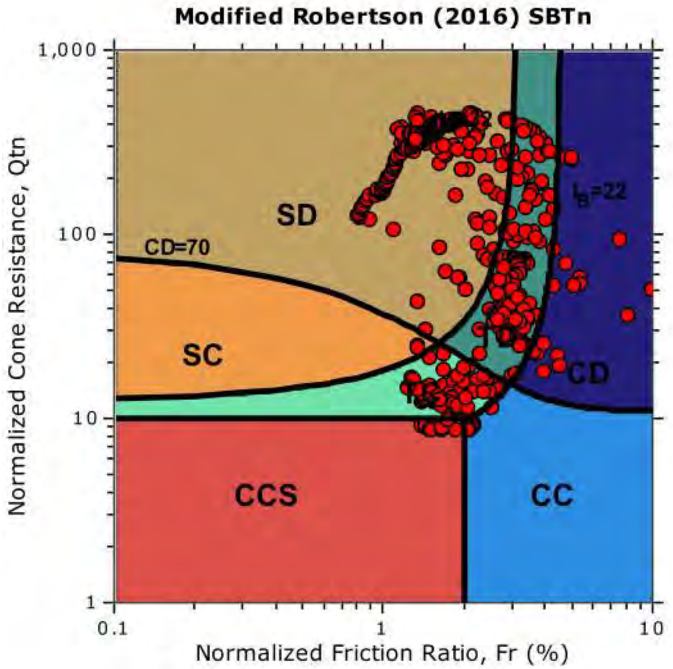
Project:
Location:

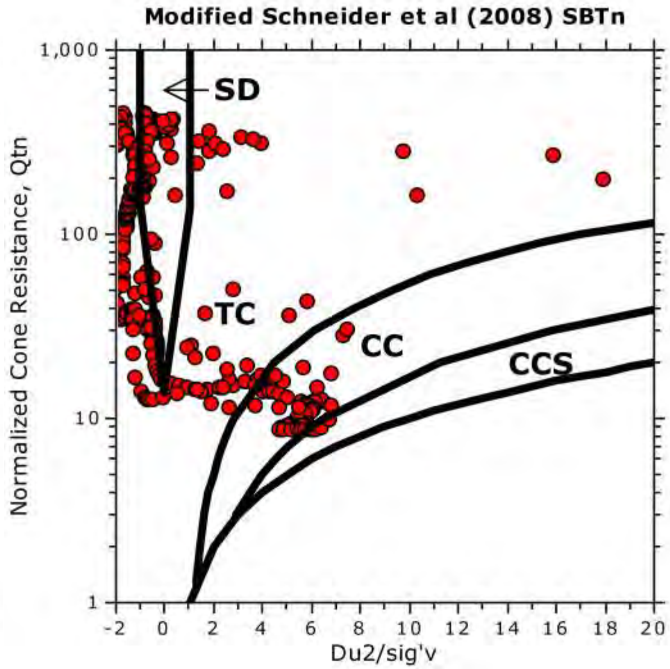


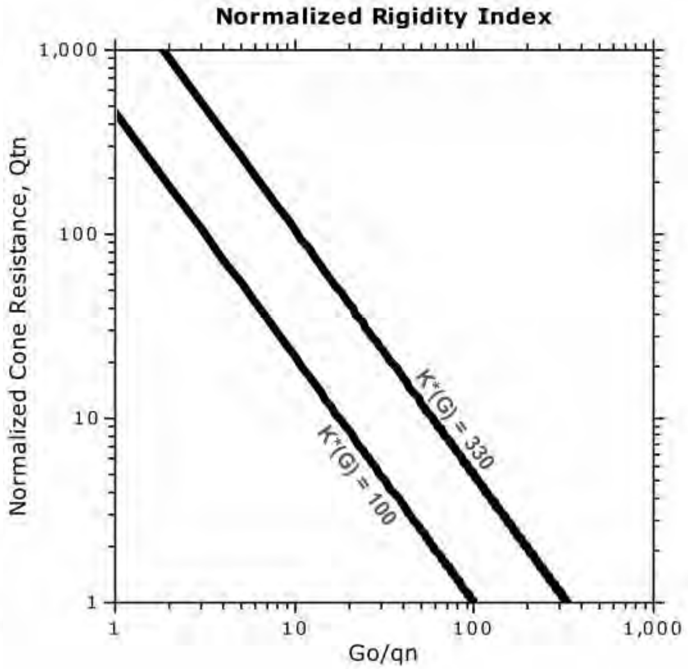
Mod. SBTn legend

- 1. CCS: ClayLike - Contractive, Sensitive
- 2. CC: Clay-like - Contractive
- 3. CD: Clay-Like: Dilative
- 4. TC: Transitional - Contractive
- 5. TD: Transitional - Dilative
- 6. SC: Sand-like - Contractive
- 7. SD: Sand-like - Dilative

Updated SBTn plots

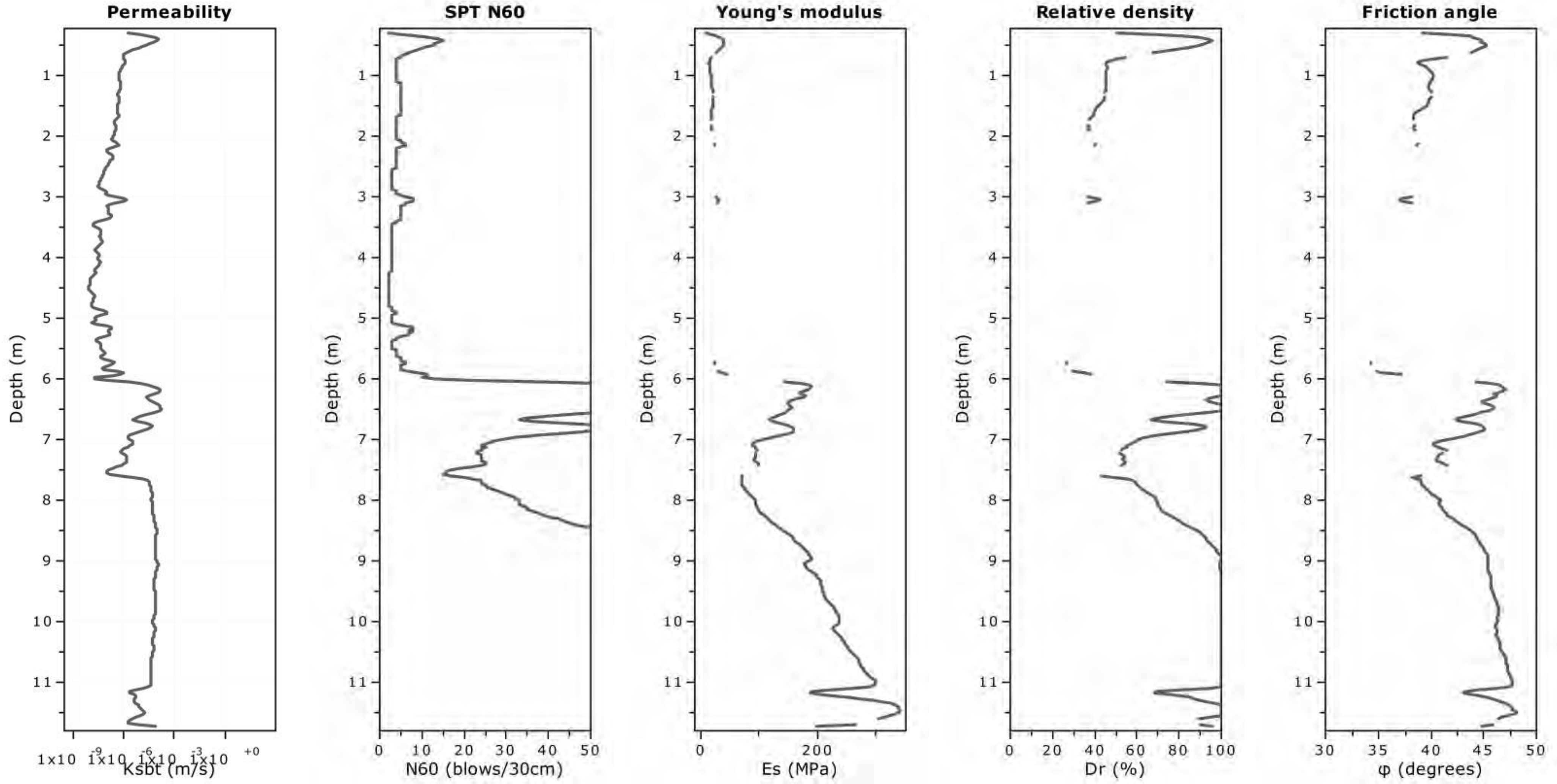






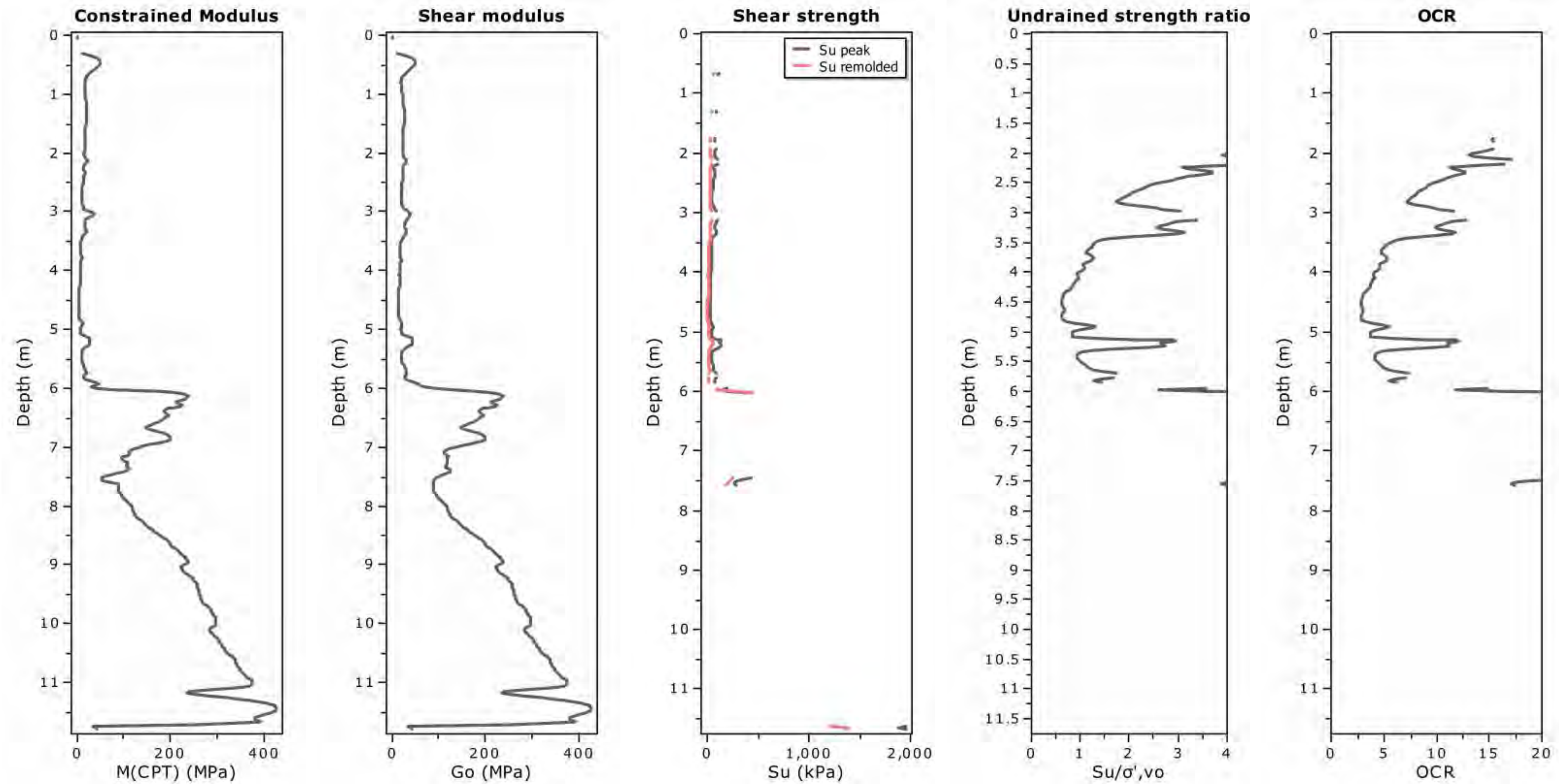
CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)



Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data



Calculation parameters

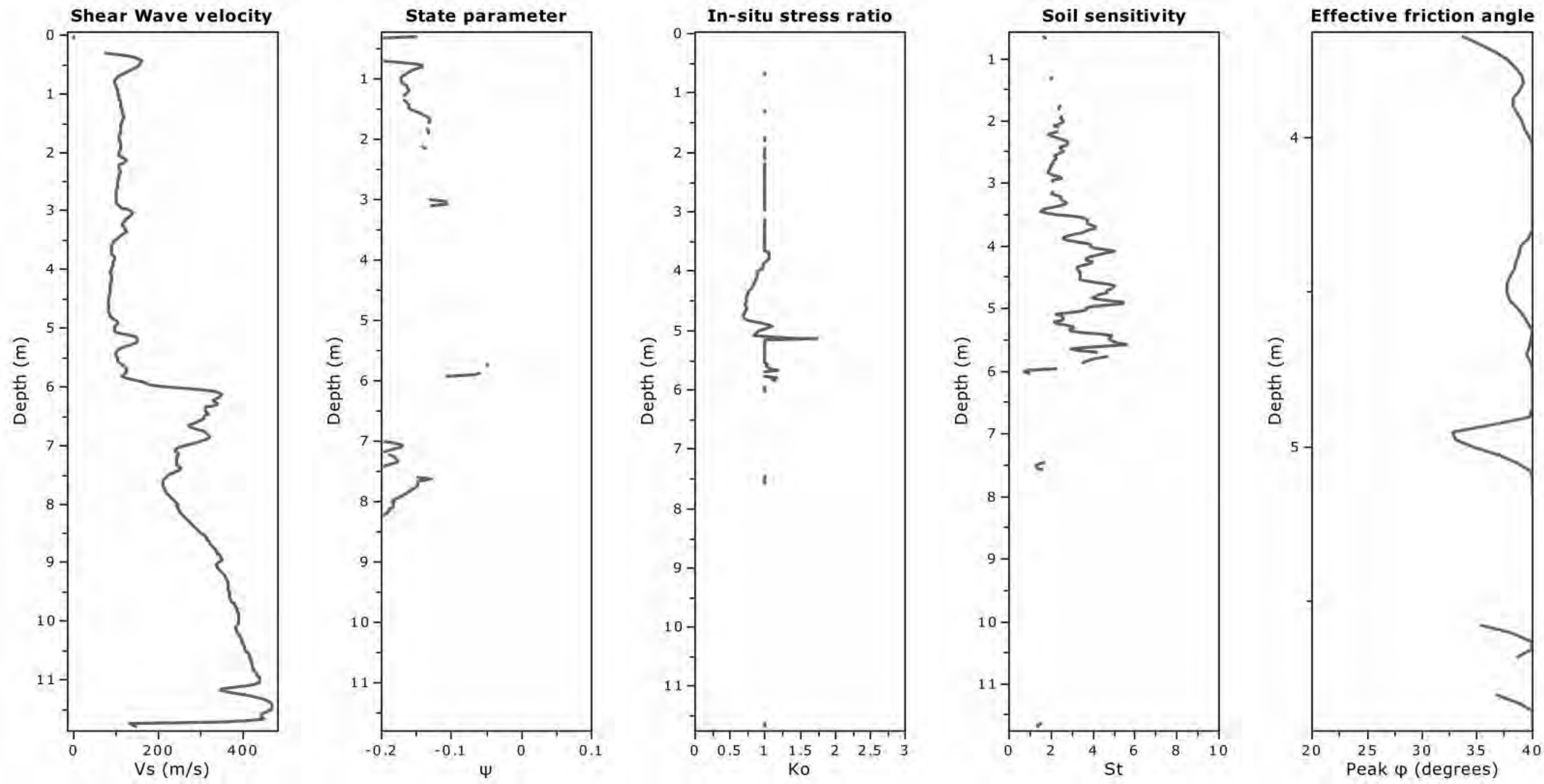
Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

CPT: Sheet1
Total depth: 11.80 m, Date: 4/18/2022
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

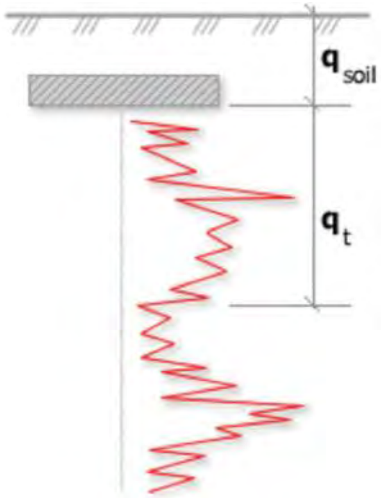
Project:
Location:



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

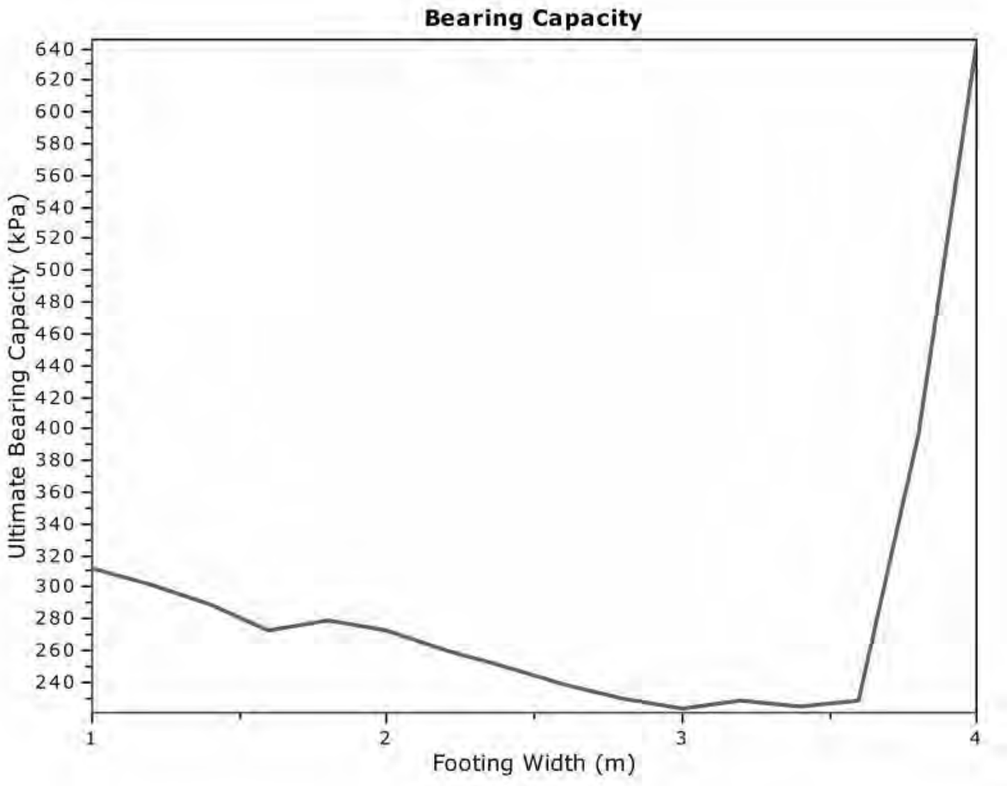
User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (m)	Start Depth (m)	End Depth (m)	Ave. q_t (MPa)	R_k	Soil Press. (kPa)	Ult. bearing cap. (kPa)
1	1.00	0.50	2.00	1.51	0.20	9.50	311.24
2	1.20	0.50	2.30	1.46	0.20	9.50	301.65
3	1.40	0.50	2.60	1.40	0.20	9.50	289.18
4	1.60	0.50	2.90	1.31	0.20	9.50	272.07
5	1.80	0.50	3.20	1.34	0.20	9.50	278.47
6	2.00	0.50	3.50	1.31	0.20	9.50	272.36
7	2.20	0.50	3.80	1.25	0.20	9.50	260.13
8	2.40	0.50	4.10	1.20	0.20	9.50	249.33
9	2.60	0.50	4.40	1.15	0.20	9.50	238.91
10	2.80	0.50	4.70	1.10	0.20	9.50	229.19
11	3.00	0.50	5.00	1.07	0.20	9.50	223.22
12	3.20	0.50	5.30	1.09	0.20	9.50	227.77
13	3.40	0.50	5.60	1.08	0.20	9.50	224.65
14	3.60	0.50	5.90	1.10	0.20	9.50	228.64
15	3.80	0.50	6.20	1.94	0.20	9.50	396.52
16	4.00	0.50	6.50	3.17	0.20	9.50	643.39

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot +7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

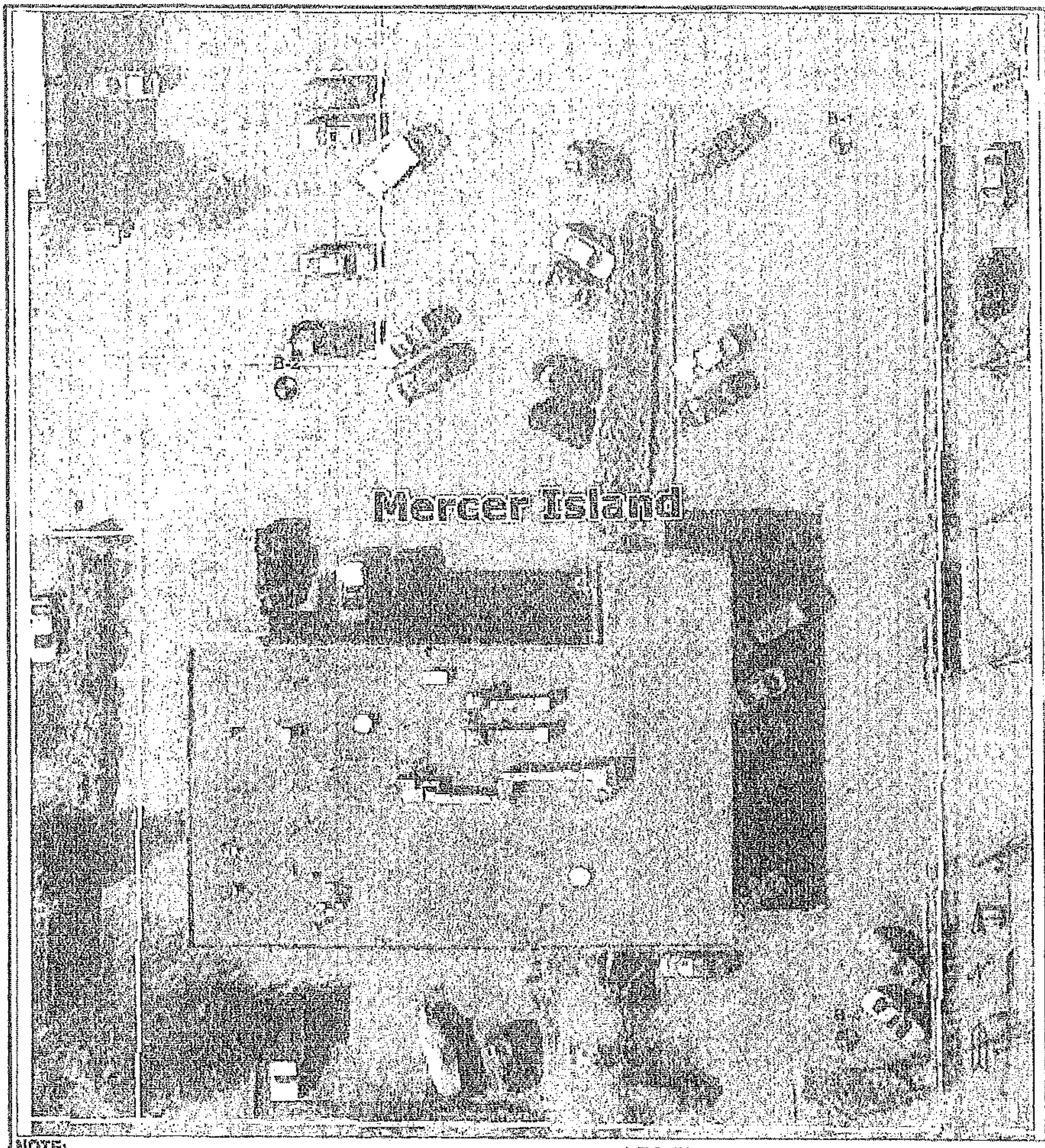
References

• Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012

• Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

• N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

APPENDIX B
Historical Explorations



NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

LEGEND:

APPROXIMATE BORING LOCATION

REFERENCE:

SITE PLAN PROVIDED BY KING COUNTY IMAP

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

EXPLORATION LOCATION PLAN
MERCER ISLAND NORTH
MERCER ISLAND, WASHINGTON

Proj. No.T-6714	Date MAY 2012	Figure 2

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS More than 50% material larger than No. 200 sieve size	GRAVELS More than 50% of coarse fraction is larger than No. 4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS More than 50% of coarse fraction is smaller than No. 4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands or gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS More than 50% material smaller than No. 200 sieve size	SILTS AND CLAYS Liquid limit is less than 50%		ML	Inorganic silts, rock flour, clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, (lean clay).
			OL	Organic silts and organic clays of low plasticity.
	SILTS AND CLAYS Liquid limit is greater than 50%		MH	Inorganic silts, elastic.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat.

DEFINITION OF TERMS AND SYMBOLS

COHESIONLESS

Density	Standard Penetration Resistance in Blows/Foot
Very loose	0-4
Loose	4-10
Medium dense	10-30
Dense	30-50
Very dense	>50

COHESIVE

Consistency	Standard Penetration Resistance in Blows/Foot
Very soft	0-2
Soft	2-4
Medium stiff	4-8
Stiff	8-16
Very stiff	16-32
Hard	>32

- I — 2" OUTSIDE DIAMETER SPLIT SPOON SAMPLER
- I — 2.4" INSIDE DIAMETER RING SAMPLER OR SHELBY TUBE SAMPLER
- ▽ — WATER LEVEL (DATE)
- Tr — TORVANE READINGS, tsf
- Pp — PENETROMETER READING, tsf
- DD — DRY DENSITY, pounds per cubic foot
- LL — LIQUID LIMIT, percent
- PI — PLASTIC INDEX
- N — STANDARD PENETRATION, blows per foot

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

UNIFIED SOIL CLASSIFICATION SYSTEM
MERCER ISLAND NORTH
MERCER ISLAND, WASHINGTON

Proj. No. T-6714	Date MAY 2012	Figure A-1

LOG OF BORING NO. B-1

Figure No. A-2

Project: Mercer Island North | Project No: T-6714 | Date Drilled: 4-25-12

Client: PMF Investments | Driller: BORETEC | Logged By: CSD

Location: Mercer Island, Washington | Approx. Elev: N/A

Depth (ft)	Soil Description	Consistency/ Relative Density	Moisture Content % (Wp |----x----| Wl; 10 20 30 40)	Pocket Penetrometer TSF (1 2 3 4) / SPT (N) Blows/ft (10 20 30 40)	Observ. Well
0–2	(4 inches ASPHALT)				
2–4.5	FILL: brown sand with silt and gravel, fine to course grained, moist.	Medium Dense		18	
4.5–6	FILL: brown and gray silty sand with gravel, fine to medium grained, moist.	Loose	18.3	3	
6–7	Dark brown SILT with organics, fine grained, moist.	Loose			
7–9.5	Gray SILT, fine grained, moist, sand pockets, slight mottling.	Stiff	40.0	14	
9.5–15.5	Brown SILT, fine grained, moist to wet, sand pockets.	Medium Stiff / Hard	28.2; 22.5	4; 42	
15.5–20	Gray silty SAND, fine to medium grained, moist to wet. (SM)	Very Dense	14.5		
20	*See Next Page				

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

Terra Associates, Inc.
Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

LOG OF BORING NO. B-1

Figure No. A-2

Project: Mercer Island North | Project No: T-6714 | Date Drilled: 4-25-12

Client: PMF Investments | Driller: BORETEC | Logged By: CSD

Location: Mercer Island, Washington | Approx. Elev: N/A

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |----x----| Wl 10 20 30 40	Pocket Penetrometer TSF 1 2 3 4; SPT (N) Blows/ft 10 20 30 40	Observ. Well
21		Gray silty SAND, fine to medium grained, moist to wet. (SM)	Very Dense	12.5	50/4"	
25				15.2	50/6"	
28–31		Gray SILT, fine grained, moist. (ML)	Hard	12.4	50/4"	
31		Test boring terminated at 31 feet. Perced groundwater observed at 13 feet during drilling. Boring converted to 2-inch monitoring well.				

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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LOG OF BORING NO. B-2

Figure No. A-3

Project: Mercer Island North Project No: T-6714 Date Drilled: 4-25-12

Client: PMF Investments Driller: BORETEC Logged By: CSD

Location: Mercer Island, Washington Approx. Elev: N/A

Depth (ft)	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |---x---| Wl 10 30 50 70 90	Pocket Penetrometer TSF 1 2 3 4 / SPT (N) Blows/ft 10 20 30 40
0–2	(4 inches ASPHALT) FILL: brown gravel, fine to course grained, saturated.		43.0	13
2–9.5	Gray sandy SILT, fine grained, moist to wet, mottled. (ML) LL=33 PL=26 PI=7	Soft	40.0; 43.7; 56.3	6; 4; 2
9.5–19.5	Gray SILT, fine grained, moist to wet. (ML)	Hard	17.5	41
19.5–31.5	Gray SAND, fine to medium grained, saturated. (SP)	Loose; Medium Dense; Dense	25.1; 23.2; 20.6	8; 29; 80/5"
31.5	Test boring terminated at 31.5 feet. Groundwater observed at 19.5 feet during drilling.			

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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LOG OF BORING NO. B-3

Figure No. A-4

Project: Mercer Island North | Project No: T-6714 | Date Drilled: 4-25-12

Client: PMF Investments | Driller: BORETEC | Logged By: CSD

Location: Mercer Island, Washington | Approx. Elev: N/A

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |----x----| Wl (10 30 50 70 90)	Pocket Penetrometer TSF (1 2 3 4) / SPT (N) Blows/ft (10 20 30 40)
1–3		(4 inches ASPHALT) FILL: gray silty sand with gravel, fine to medium grained, moist.	Medium Dense		
5	I	Gray SILT, fine grained, moist, occasional brown sand pocket, mottled. (ML)	Medium Stiff	46.4	8
10	I	LL=34 PL=27 PI=7		46.2	4
15	I	*At 15 feet soil becomes wet, no sand pockets		43.4	4
20	I			20.8 / 17.2	39
21 (groundwater)					
25	I	Gray SAND, fine to medium grained, saturated. (SP)	Dense	21.0	33
30	I			26.7	80/5"
32–40		Test boring terminated at 31.5 feet. Groundwater observed at 21 feet during drilling. Groundwater observed at 15.5 feet after drilling.			

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

Terra Associates, Inc.
Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

LOG OF BORING NO. B-4

Figure No. A-5

Project: Mercer Island North | Project No: T-6714 | Date Drilled: 4-25-12

Client: PMF Investments | Driller: BORETEC | Logged By: CSD

Location: Mercer Island, Washington | Approx. Elev: N/A

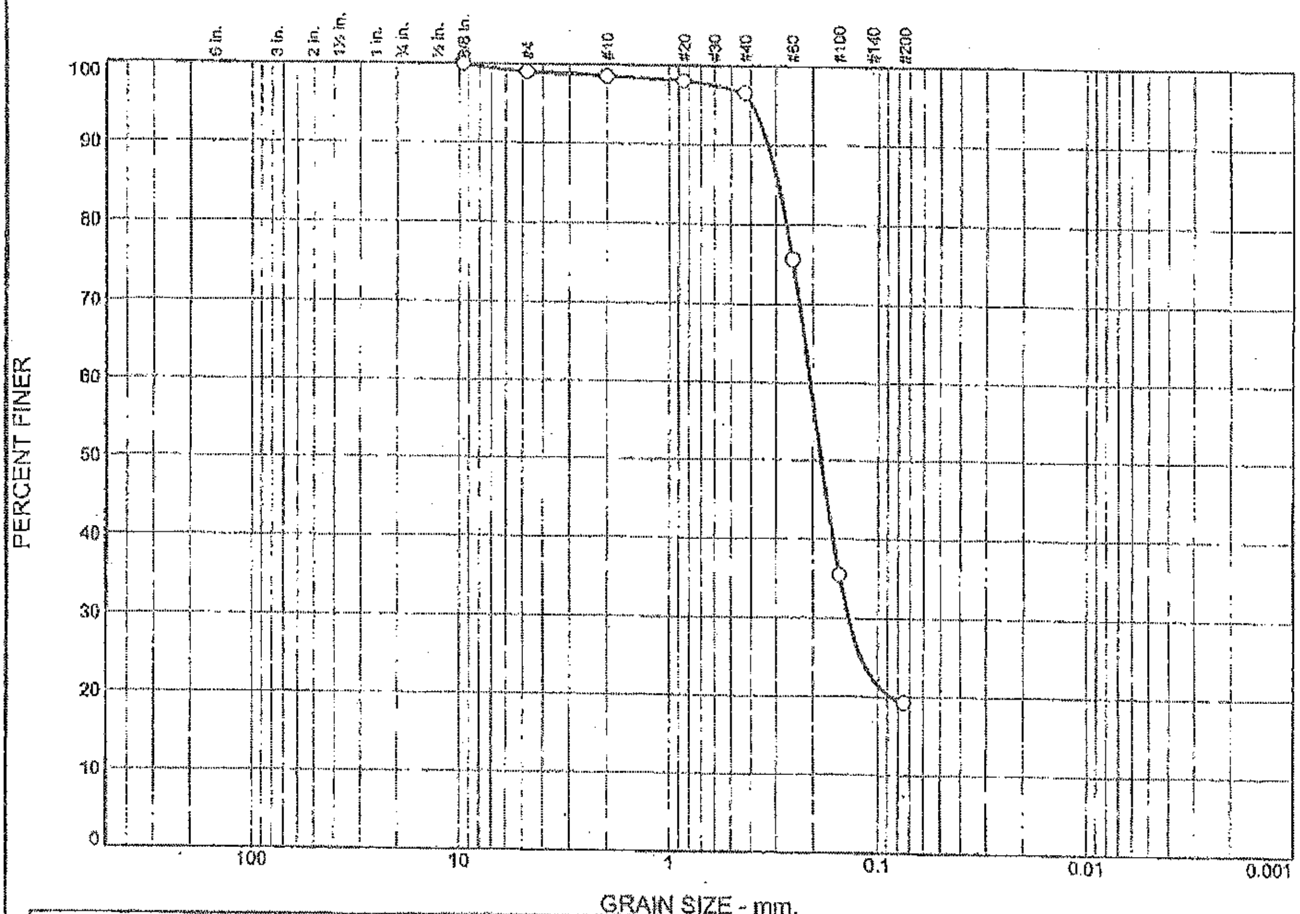
Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % (Wp |----x----| Wl; 10 30 50 70 90)	Pocket Penetrometer TSF (1 2 3 4) / SPT (N) Blows/ft (10 20 30 40)
0–1		(3.5 inches ASPHALT)			
1–8.5	5–6.5	FILL; mix of brown sand with silt and gravel and gray silty sand with gravel, fine to coarse grained, moist.	Medium Dense	16.9	24
8.5–13.5	10–11.5	Brown silty SAND, fine to medium grained, moist. (SM)	Very Dense	15.2	56
13.5–27	15–16.5	Gray SILT, fine grained, moist. (ML)	Hard	17.8	90/4"
	20–21.5			11.2	50/5"
	25–26.5			23.7	57
27–31.5	30–31.5	Gray SAND, fine to medium grained, saturated. (SP)	Medium Dense	21.9	19

Test boring terminated at 31.5 feet.
Groundwater observed at 28 feet during drilling.
Groundwater observed at 22 feet after drilling.

Note: This borehole log has been prepared for geotechninal purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

Terra Associates, Inc.
Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	1.0	0.5	1.9	77.3	19.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.2925	0.2051	0.1822	0.1354				

Material Description	USCS	AASHTO
○ Silty SAND	SM	

Project No. T-6714 **Client:** PMF Investements

Project: Mercer Island North

Mercer Island, Washington

○ **Location:** Test Boring B-1 **Depth:** -25' **Sample Number:** 7

Terra Associates, Inc.

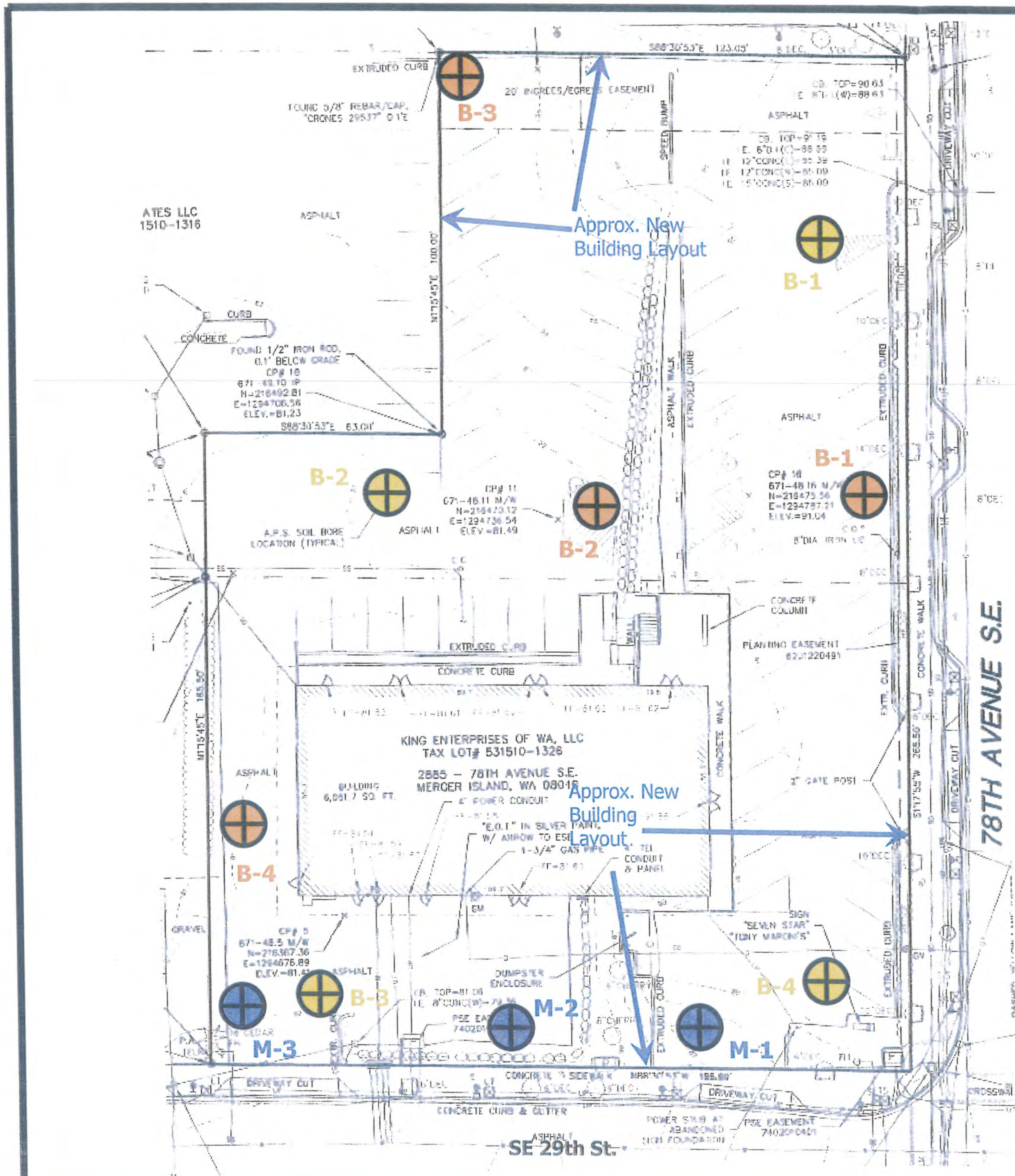
Kirkland, WA

Remarks:

○ Tested on 4/27/2012

Figure A-6

Tested By: BS



LEGEND

B-1 Approx. Boring Location and Number

B-1 Approx. Boring Locations by Terra Associates (4-12)

M-1 Monitoring Well Locations

Approx. Scale

0 20' 40' 60' 80' 100'

1" = 31'

Ref: Pace Inc. Survey dated October 11, 2012

ABPB Consulting
Geotechnical Consultants
Kirkland, Wash.

Exploration Location Plan
Mercer Island Multi-Family Project
Mercer Island, Washington

Proj. No. 1250	Date : October 2012	Figure 2

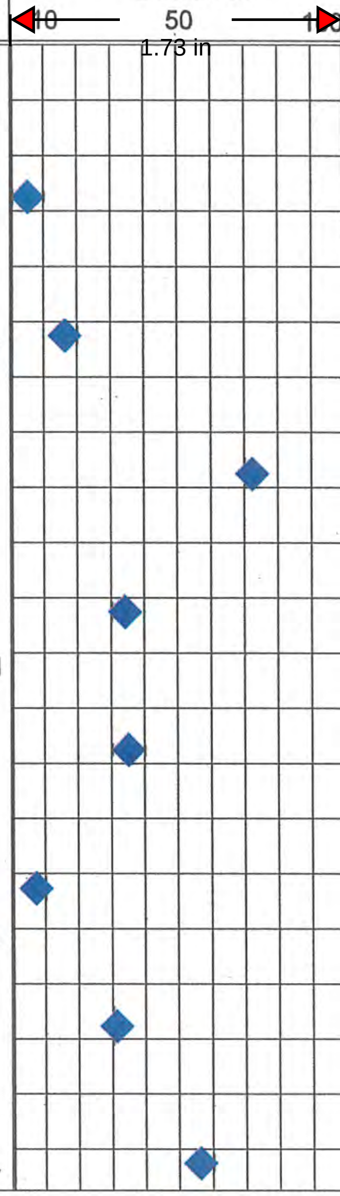
Project :	**Mercer Island Multi-Family**		
Project No.	1350	**Date :**	10-2-12
Client :	Continental Pacific	**Elevation**	92 Feet
Location:	East Side	**Logged By:**	Paul Bonifaci

Boring No. B-1

Depth (ft)	Soil Description	USCS	Moisture Content
0 to -4.5	Fill: (3" Asphalt over basecourse) Brown to black tan, mixed silty SAND with some wood and SILT, medium stiff to loose, moist (FILL)	ML	
-5			25.0%
-4.5 to -29	SILT: Tan mottled grey to grey, interbedded clayey SILT and sandy SILT, occasional pebbles top ten feet, very stiff grading to hard to dense, moist grades to wet and damp below 15 feet		
-10		ML	15.6%
-15			10.3%
-20			25.0%
-22		SM	
-25			21.3%
-29 to -38	SAND: Blue grey, clean to slightly silty, fine to medium SAND, grades to silty SAND, wet to saturated, medium dense to dense, waterbearing		
-30			28.2%
-32		SP	
-35			25.0%
-38 to -41	SILT: Blue grey, SILT with clay to clayey SILT, hard , moist to damp		
-40	Groundwater level at about 14 feet at time drilling	ML	25.6%

SUBSURFACE PROFILE: Depth (ft), Soil Lithology, Soil Description. SAMPLE: Water Level, Sample, USCS.

△ Standard Penetration Resistance (SPT)



Laboratory Results

Moisture Content

ABPB Consulting
Geotechnical Consultants

12525 Willows Road, Suite 80, Kirkland, Washington (425) 820-2544

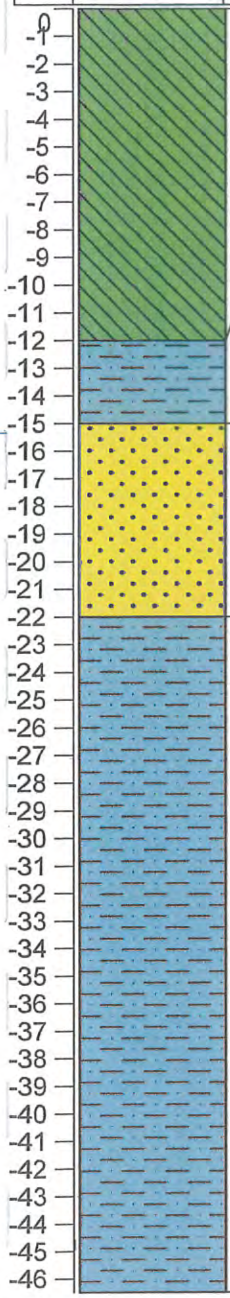
Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 3

Project :	**Mercer Island Multi-Family**		
Project No.	1350	**Date :**	10-2-12
Client :	Continental Pacific	**Elevation**	82 Feet
Location:	East Side	**Logged By:**	Paul Bonifaci

Boring No. B-2

SUBSURFACE PROFILE | SAMPLE

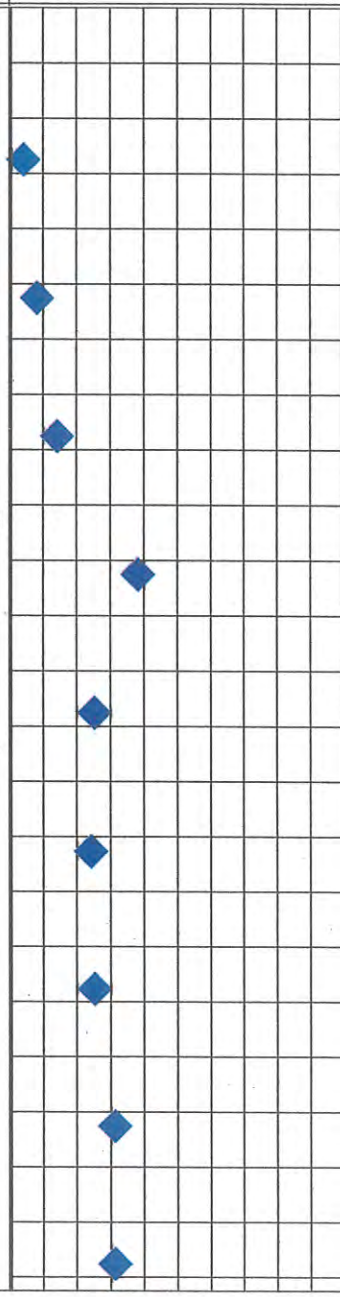
Depth (ft) | Soil Lithology | Soil Description | Water Level | Sample | USCS



Soil Description	USCS
CLAY: (3" Asphalt over basecourse) Brown to grey tan, plastic clayey SILT to silty CLAY, soft to stiff, wet to damp	ML CL
SILT: Tan blue grey to grey, interbedded clayey SILT, to SILT, dense to hard, moist	
SAND: Tan blue grey to grey, interbedded clean to silty SAND, some lenses of sandy SILT, water-bearing, wet to saturated, medium dense to dense	SP SM
SILT: Blue grey, interbedded SILT with clay to clayey SILT, non-bedded, some disturbed layers, hard , moist to damp, a few very silty sand layers. Water table at about 7 feet at time drilling	ML

△ Standard Penetration Resistance (SPT)

Blows/Foot



Laboratory Results

Moisture Content

58.8%

13.9%

19.2%

25.6%

33.3%

28.2%
qu=+4.5tsf

30.6%

28.6%
qu=+4.5tsf

32.0%
qu=3.0tsf

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Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 4

Project :	**Mercer Island Multi-Family**		
Project No. 1350		**Date :**	10-2-12
Client :	Continental Pacific	**Elevation**	85 Feet
Location:	NW corner	**Logged By:**	Paul Bonifaci

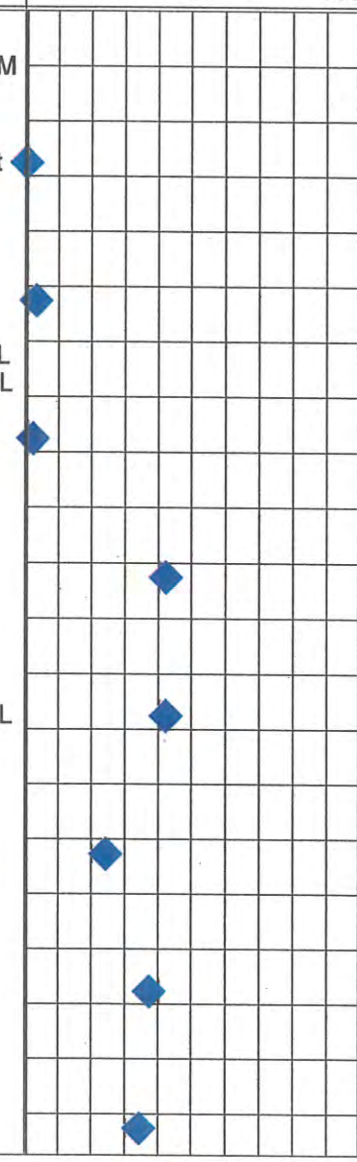
Boring No. B-3

Depth (ft)	Soil Description	USCS	Laboratory Results – Moisture Content
0 to -2	FILL: (3" Asphalt over basecourse) Brown to grey tan, silty gravelly SAND (FILL), loose, moist	SM	
-2 to -9	PEAT: Interbedded brown, organic PEAT, mixed with silty organic CLAY, very soft, wet	Pt	65.2%
-9 to -20	CLAY: Tan blue grey, interbedded silty CLAY and clayey SILT, scattered organic fragments, very soft to soft, damp to wet	CL ML	45.0% qu=0.75tsf; 44.7% qu=0.25tsf
-20 to -41	SILT: Blue grey, interbedded SILT with clay to clayey SILT, non-bedded, occasional sandy SILT layers, hard , moist to damp, a few very silty sand layers. Water table at about 7 feet at time drilling	ML	26.5%; 21.9% qu=2.5tsf; 37.9%; 20.0% qu=+4.5tsf; 29.0

SUBSURFACE PROFILE: Depth (ft), Soil Lithology, Soil Description — SAMPLE: Water Level, Sample, USCS

△ Standard Penetration Resistance (SPT)

Blows/Foot



Laboratory Results

Moisture Content

Sample depth (approx. ft)	Moisture Content
-6	65.2%
-11	45.0% qu=0.75tsf
-16	44.7% qu=0.25tsf
-21	26.5%
-26	21.9% qu=2.5tsf
-31	37.9%
-36	20.0% qu=+4.5tsf
-41	29.0

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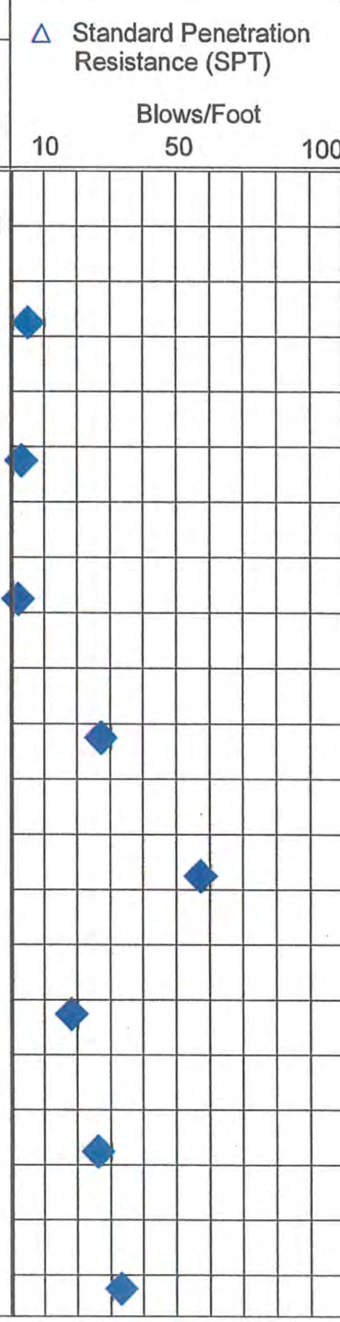
Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 5

Project :	**Mercer Island Multi-Family**		
Project No. 1350		**Date :**	10-2-12
Client :	Continental Pacific	**Elevation**	81 Feet
Location:	SW side	**Logged By:**	Paul Bonifaci

Boring No. B -4

Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS	Laboratory Results: Moisture Content
0 to -3		FILL: (3" Asphalt over basecourse) Brown to grey tan, silty gravelly SAND (FILL), loose, moist			SM	
-3 to -19		CLAY: Interbedded grey to blue grey, layers of silty CLAY and clayey SILT, scattered organic fragments, very soft to soft, damp to wet	▼ (-8)		ML CL	
-5				▥		40.5% qu=1.25tsf
-10				▥		37.9% qu=0.2tsf
-15				▥		52.6%
-19 to -41		SILT: Tan blue grey, interbedded sandy SILT with occasional pebbles, to mostly clayey SILT, some silty CLAY lenses, very stiff grading to hard, damp to moist			ML	
-20				▥		26.9% qu=3.5tsf
-25				▥		15.6%
		Groundwater at about 8 feet at time of drilling				
-30				▥		27.8% qu=+4.5tsf
-35				▥		35.7%
-40				▥		28.2%



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Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 6

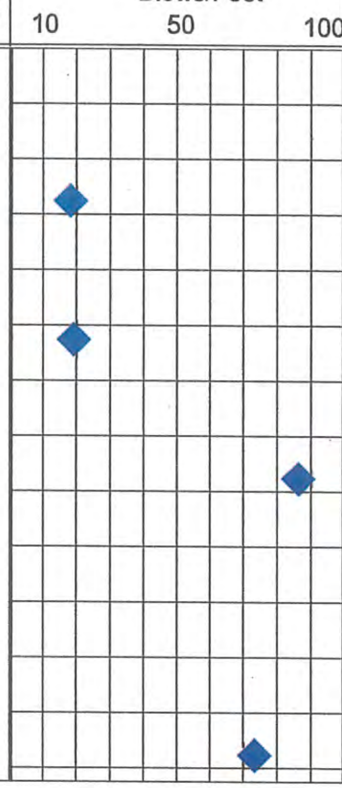
Project :	**Mercer Island Multi-Family**			Boring No. M-1
Project No.	1350	**Date :**	10-19-12	
Client :	Continental Pacific	**Elevation**	87 feet	
Location:	South Side	**Logged By:**	Terry Bukowsky	

SUBSURFACE PROFILE | SAMPLE

Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS
0 to -26		Fill: (3" Asphalt over basecourse) Brown tan grading to grey, layers of sandy SILT to SILT, stiff grading to hard/very dense, moist grading to damp Monitoring Well installed to 25 feet.			ML

△ Standard Penetration Resistance (SPT)

Blows/Foot



Laboratory Results

Moisture Content

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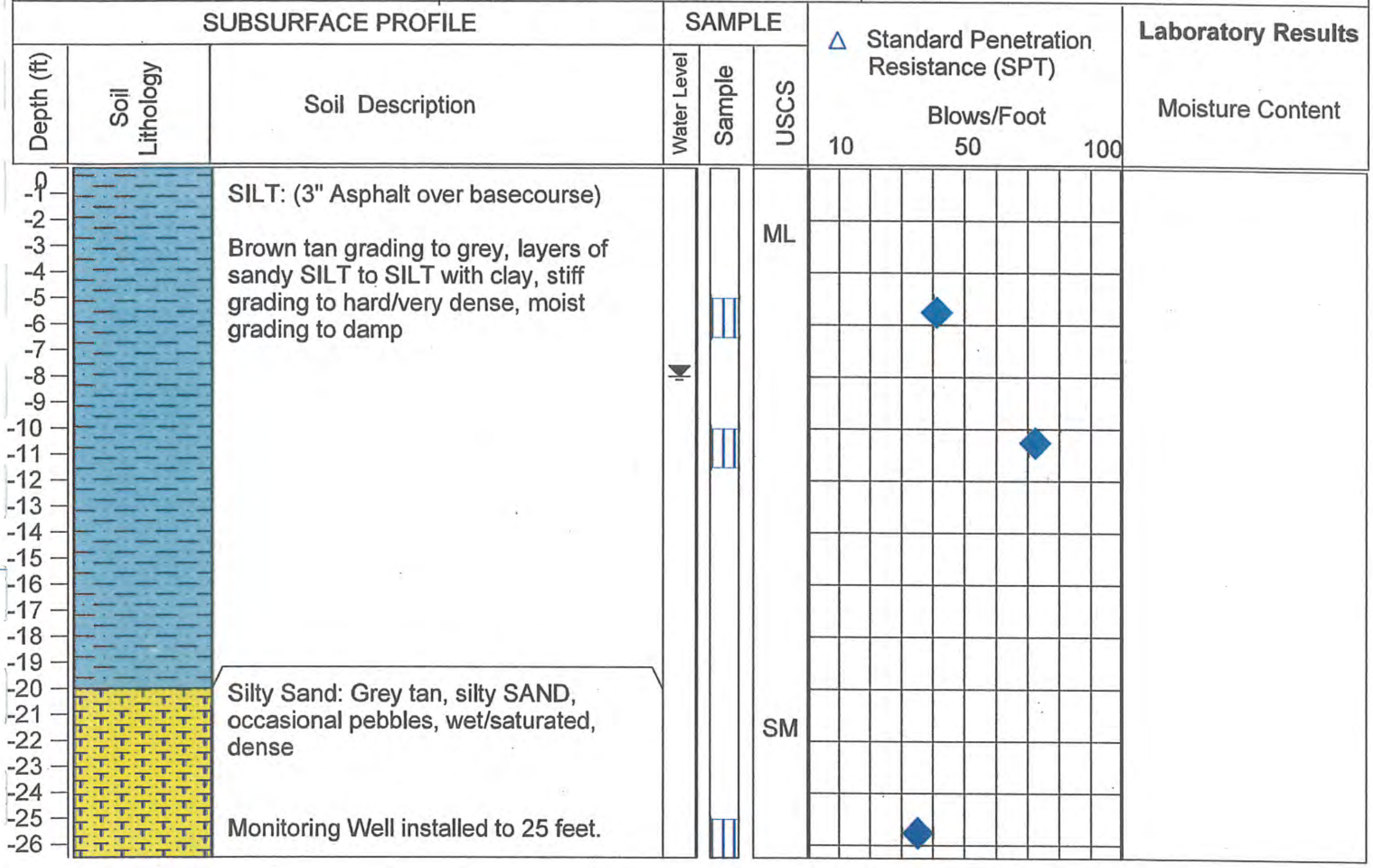
Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 7

Project :	**Mercer Island Multi-Family**			Boring No. M-2
Project No. 1350		**Date :**	10-20-12	
Client :	Continental Pacific	**Elevation**	83 feet	
Location:	South Side	**Logged By:**	Terry Bukowsky	



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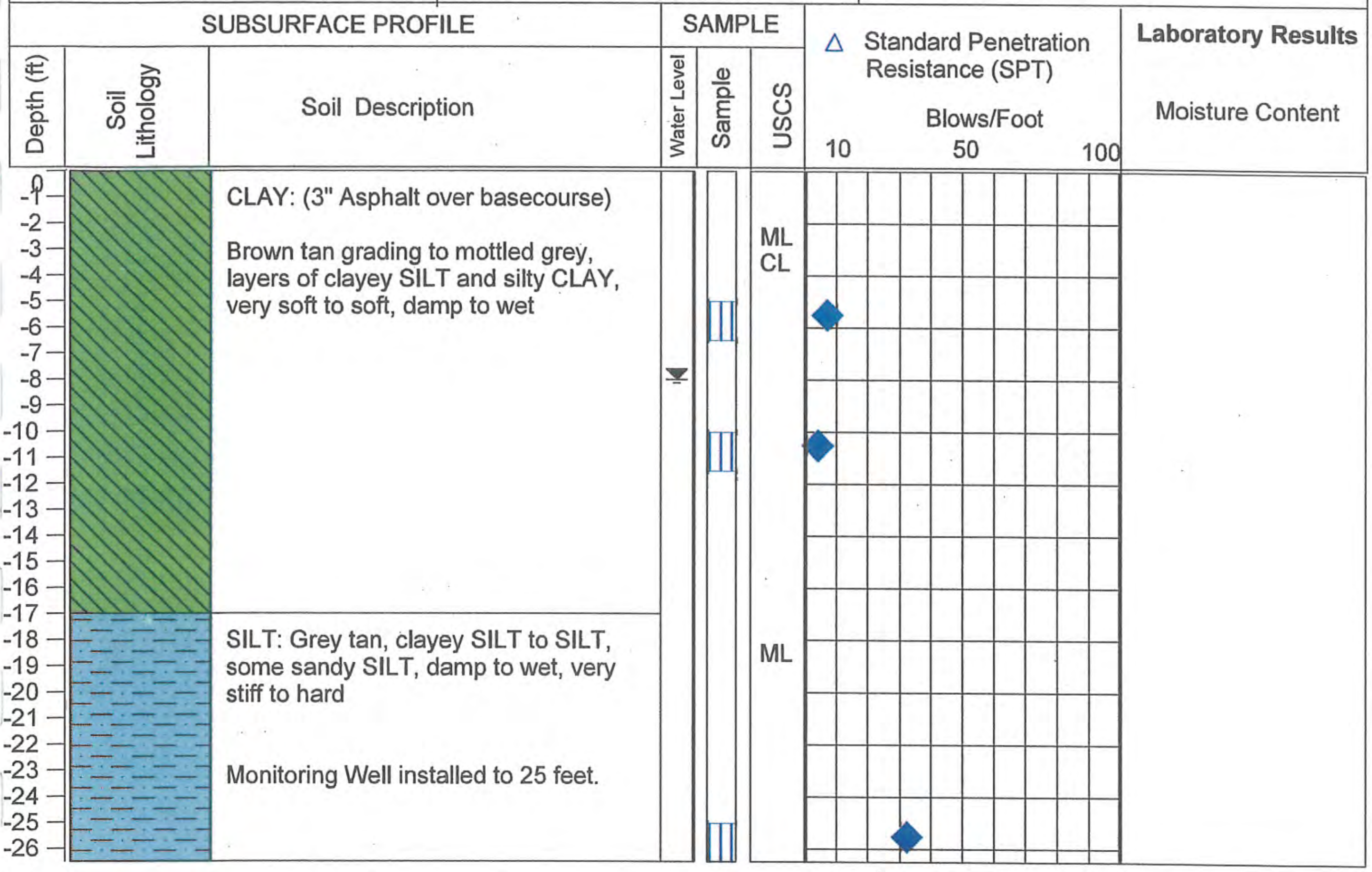
Date : Oct. 2012	Project Name: Mercer Island Multi-Family	Figure 8

Project :	**Mercer Island Multi-Family**		
Project No.	1350	**Date :**	10-20-12
Client :	Continental Pacific	**Elevation**	82 feet
Location:	South Side	**Logged By:**	Terry Bukowsky

Boring No. M-3



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APPENDIX C
Slug Test Results

MEMORANDUM

DATE: December 12, 2014

TO: Hines

FROM: Angie Goodwin, LHG
Roy Jensen, LHG

RE: **Summary of Mercer Island Multi-Family Development Slug Test Results**
Mercer Island, Washington
17984-01

This technical memorandum presents the results of slug testing that was conducted for the Mercer Island Multi-Family Development in Mercer Island, Washington. The development is located on the northwest corner of the intersection of SE 29th Street and 78th Avenue SE. We understand that current development plans include one to two stories of below grade parking and five levels of housing and mixed-use space plus rooftop mechanical equipment. Slug tests were performed to determine hydraulic conductivity of formation for use in estimating flow rates during dewatering.

Slug tests are performed by suddenly inserting or removing a solid PVC rod in a well and measuring the recovery of the water levels during the test. A test conducted by the insertion of the PVC rod into the well is referred to as a falling head test and the following removal of the rod is called a rising head test. The water level data generated from the tests were analyzed using the commercial software Aquifer^{Win32} Version 3 (Environmental Simulations, Inc., 2003). The slug test analysis is based on the Bouwer and Rice method (Bouwer and Rice 1976; Bouwer 1989) to obtain an estimated value of hydraulic conductivity of the aquifer.

Slug Testing Results

Slug testing was conducted in wells HC-1, HC-2, ABPB-M3, and Terra-B1 on November 17, 2014. A summary of monitoring well construction details is provided in Table 1. Shallow soils at the project site consist of Fill, silty Sand, and Silt units. The wells were screened in two stratigraphic units and are summarized below:

- HC-1 was screened in the Silt and silty Sand units;
- HC-2 was screened in the silty Sand unit;

- ABPB-M3 boring log did not identify the screened interval, but it was assumed the well was screened in the Silt and silty Sand units; and
- Terra-B1 was screened in the Silt unit.

A summary of slug testing results is provided in Table 2. The slug test plots are provided as Figures 1 through 6. Multiple sets of falling and rising head tests were performed on each well. The results of the falling and rising head tests compare favorably. Average hydraulic conductivities determined from slug tests range from 9.0×10^{-5} to 8.3×10^{-4} cm/sec (0.3 to 2.4 feet/day). This hydraulic conductivity range is typical for silt and silty sand (Freeze and Cherry 1979).

References

Bouwer H. 1989. The Bouwer and Rice Slug Test – An Update. Ground Water 27(3): 304-309.

Bouwer H. and R.C. Rice 1976. A Slug Test for Determining Hydraulic Conductivity of Unconfined Aquifers with Completely or Partially Penetrating Wells. Water Resources Research 12(3): 423-428.

Environmental Simulations, Inc. 2003. Guide to Using Aquifer^{Win32} Version 3.

Freeze, R.A. and J.A. Cherry 1979. Groundwater. Prentice-Hall, Englewood Cliffs, New Jersey.

Attachments:
Table 1 – Monitoring Well Construction Summary
Table 2 – Summary of Slug Test Results
Figure 1 – HC-1 and HC-2 Hydrographs
Figure 2 – ABPB-M3 and Terra-B1 Hydrographs
Figure 3 – HC-1 Representative Slug Tests Results
Figure 4 – HC-2 Representative Slug Tests Results
Figure 5 – ABPB-M3 Representative Slug Tests Results
Figure 6 – Terra-B1 Representative Slug Tests Results

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Table 1 - Monitoring Well Construction Summary

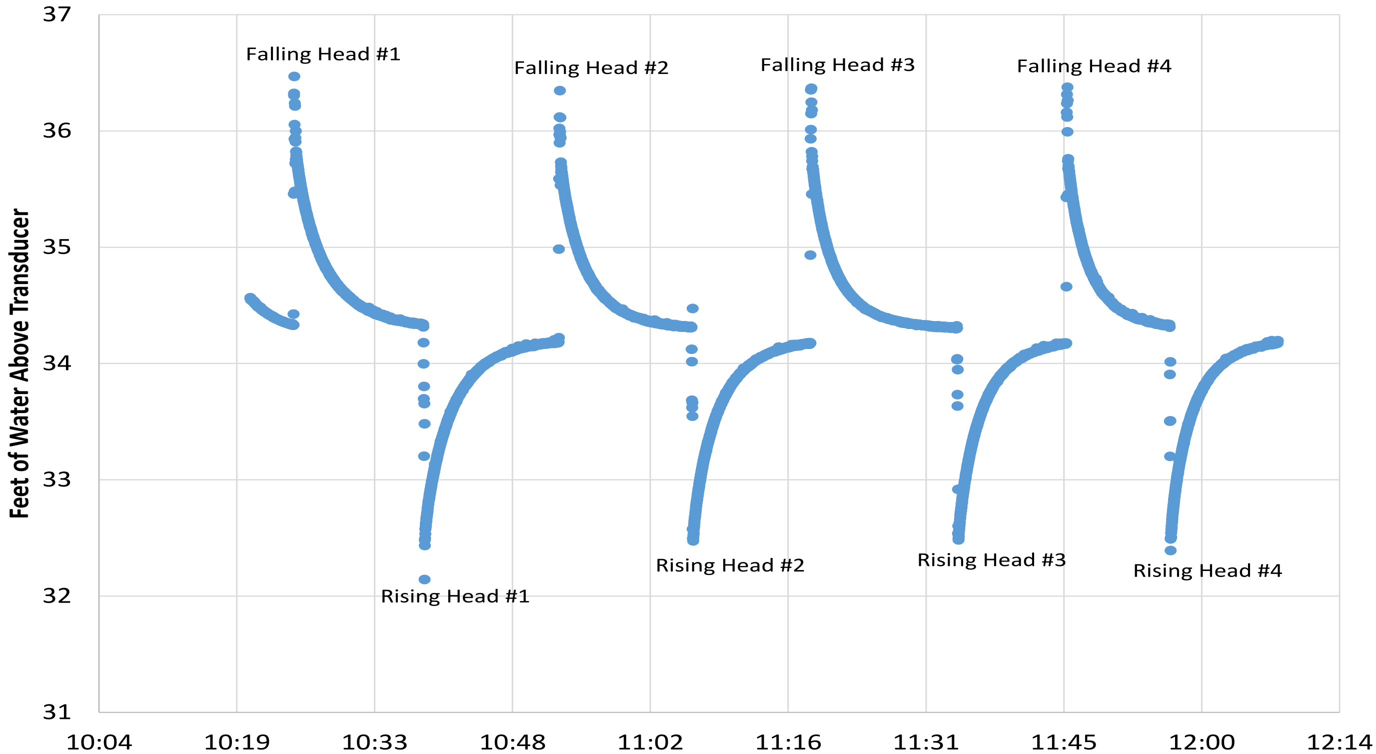
Well ID	**HC-1**	**HC-2**	**ABPB-M3**	**Terra-B1**
Boring Depth in Feet	41.5	41.5	26.5	31
Well Depth in Feet	40	39	25	17
Screen Interval Depth in Feet	20 to 40	29 to 39	NA	7 to 17
Depth to Sediment in Feet (1)	39.95	36.74	23.10	16.54
Depth to Water in Feet (1)	5.38	5.43	2.75	8.71
Saturated Thickness in Feet	35	31	20	8
Screened Interval Soil Description	ML - SM	SM	ML - SM	ML

Notes:
(1) Depth to sediment and depth to water was measured on November 17, 2014.
SM = Silty SAND
ML = Sandy SILT
NA = Data not available.

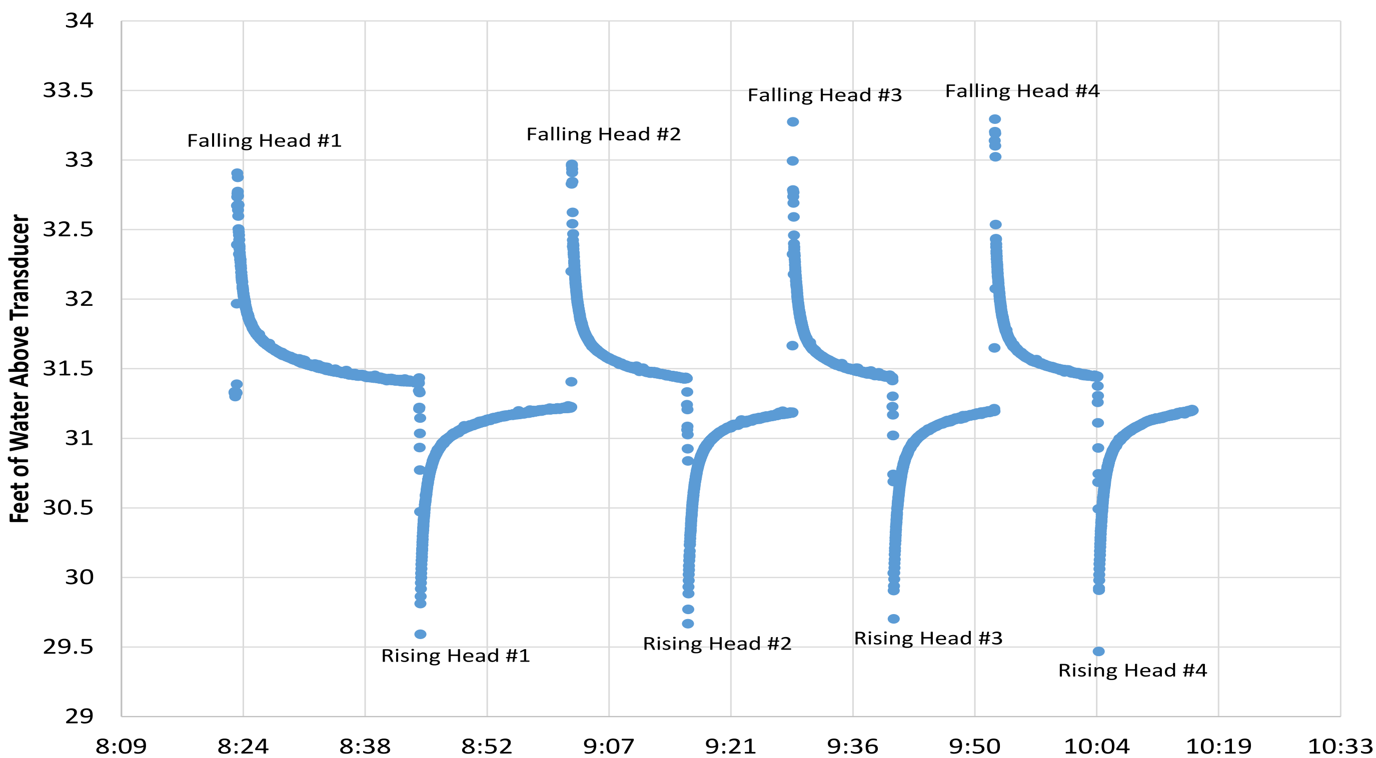
Table 2 - Summary of Slug Test Results

Well ID	Test Type	Test Number	Bouwer and Rice		
				K in ft/day	K in cm/sec
HC-1	Falling Head	Test 1		0.3	1.1E-04
	Rising Head	Test 1		0.4	1.4E-04
	Falling Head	Test 2		0.3	1.2E-04
	Rising Head	Test 2		0.4	1.5E-04
	Falling Head	Test 3		0.4	1.5E-04
	Rising Head	Test 3		0.4	1.5E-04
	Falling Head	Test 4		0.4	1.4E-04
	Rising Head	Test 4		0.4	1.5E-04
			Average	*0.4*	*1.4E-04*
HC-2	Falling Head	Test 1		2.4	8.4E-04
	Rising Head	Test 1		2.6	9.2E-04
	Falling Head	Test 2		2.1	7.5E-04
	Rising Head	Test 2		2.2	7.7E-04
	Falling Head	Test 3		2.6	9.3E-04
	Rising Head	Test 3		2.4	8.6E-04
	Falling Head	Test 4		1.9	6.6E-04
	Rising Head	Test 4		2.7	9.4E-04
			Average	*2.4*	*8.3E-04*
ABPB-M3	Falling Head	Test 1		1.8	6.3E-04
	Rising Head	Test 1		1.8	6.2E-04
	Falling Head	Test 2		1.8	6.5E-04
	Rising Head	Test 2		1.9	6.6E-04
	Falling Head	Test 3		1.6	5.7E-04
	Rising Head	Test 3		1.9	6.8E-04
	Falling Head	Test 4		1.9	6.7E-04
	Rising Head	Test 4		2.1	7.3E-04
			Average	*1.8*	*6.5E-04*
Terra-B1	Falling Head	Test 1		0.2	5.7E-05
	Rising Head	Test 1		0.5	1.8E-04
	Falling Head	Test 2		0.1	3.1E-05
	Rising Head	Test 2		0.3	1.2E-04
	Falling Head	Test 3		0.2	5.3E-05
	Rising Head	Test 3		0.3	1.1E-04
	Falling Head	Test 4		0.2	6.5E-05
	Rising Head	Test 4		0.3	1.0E-04
			Average	*0.3*	*9.0E-05*

HC-1 Hydrograph



HC-2 Hydrograph



Mercer Island Multi-Family Development
Mercer Island, Washington

HC-1 and HC-2 Hydrographs

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Figure
1

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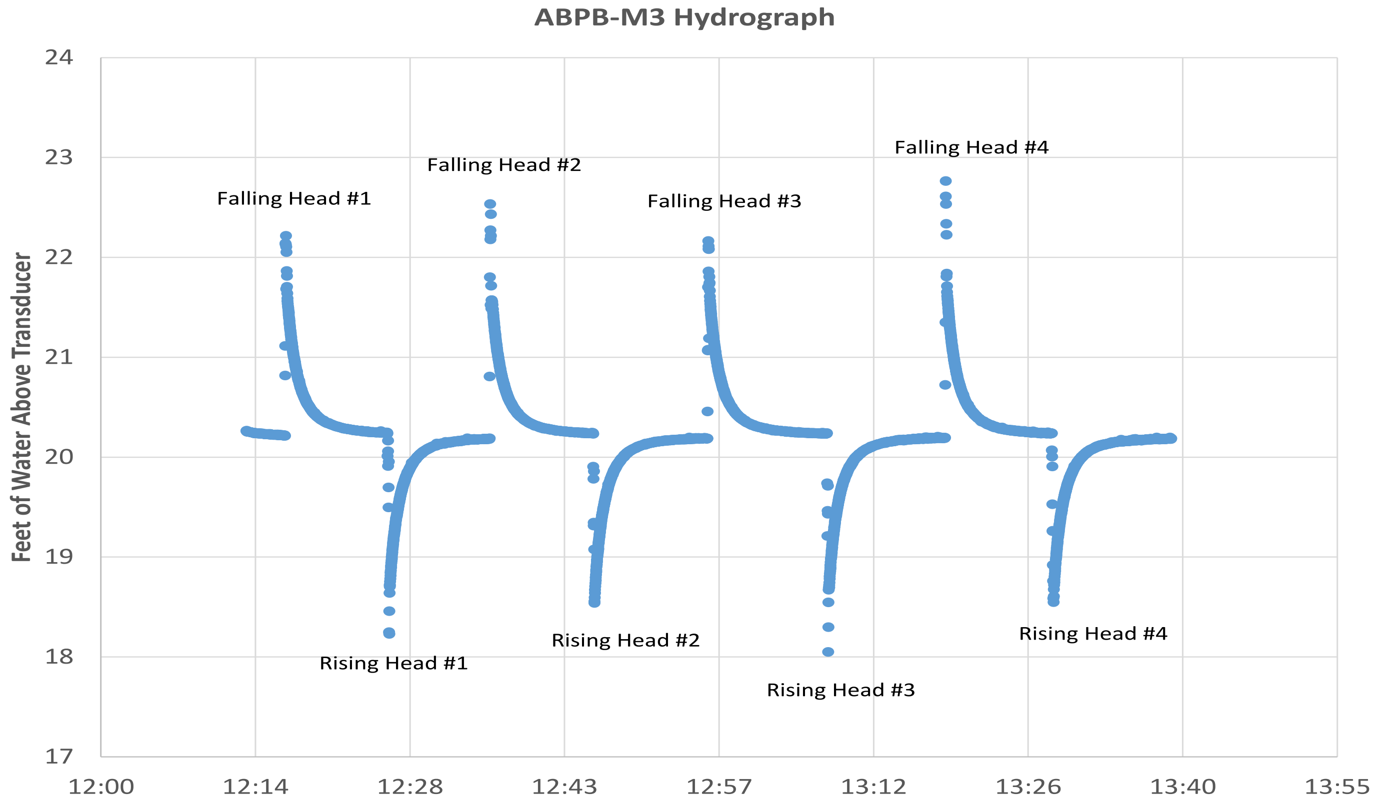

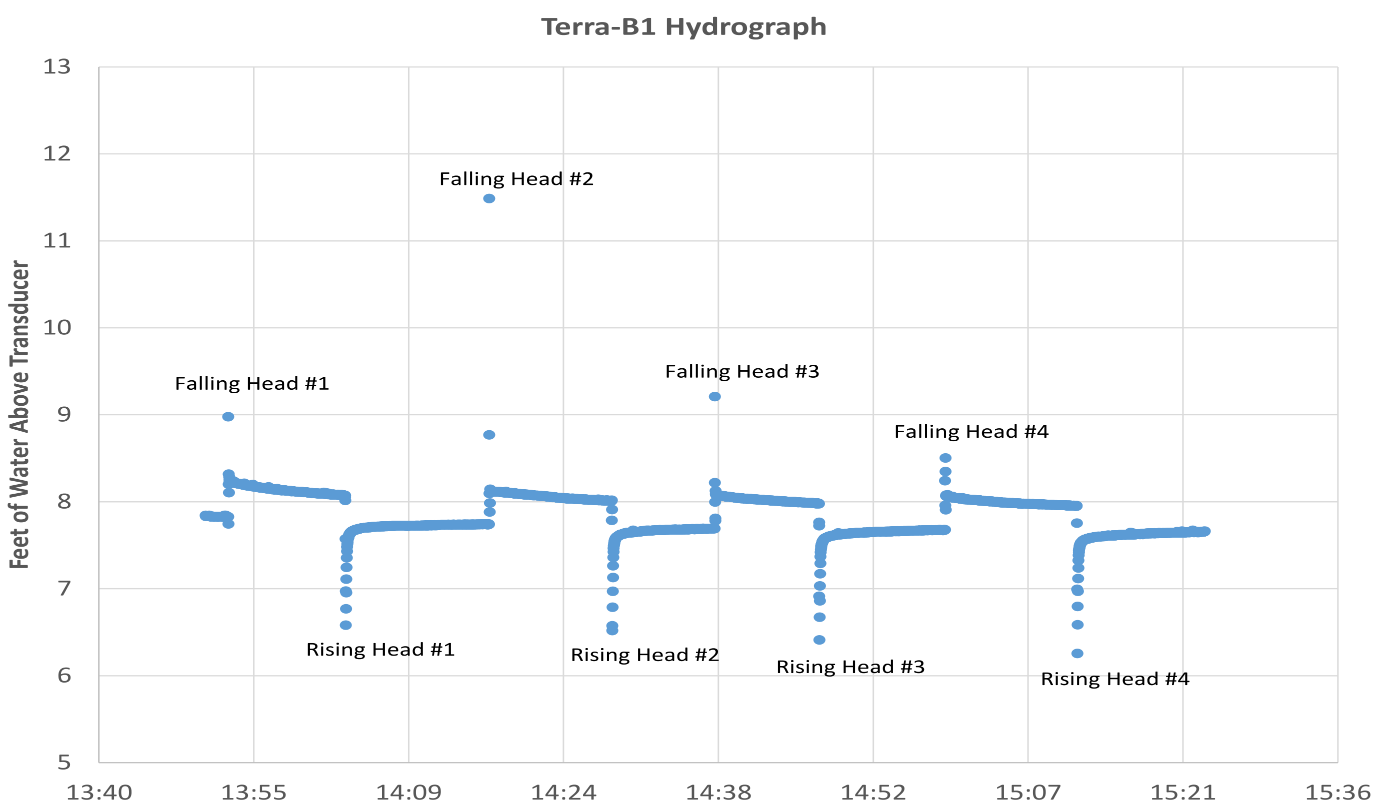

Mercer Island Multi-Family Development
Mercer Island, Washington

ABPB-M3 and Terra B-1 Hydrographs

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Figure
2

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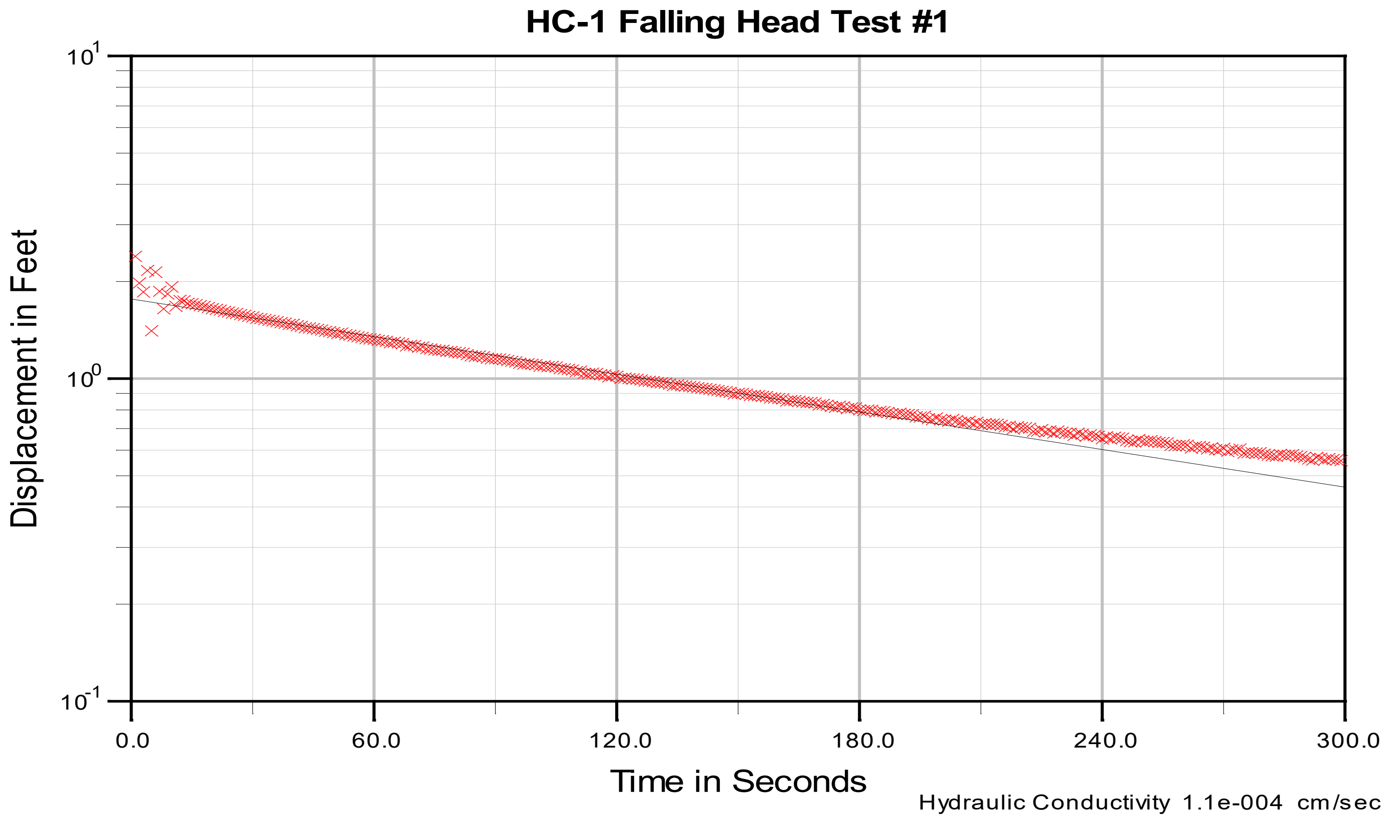

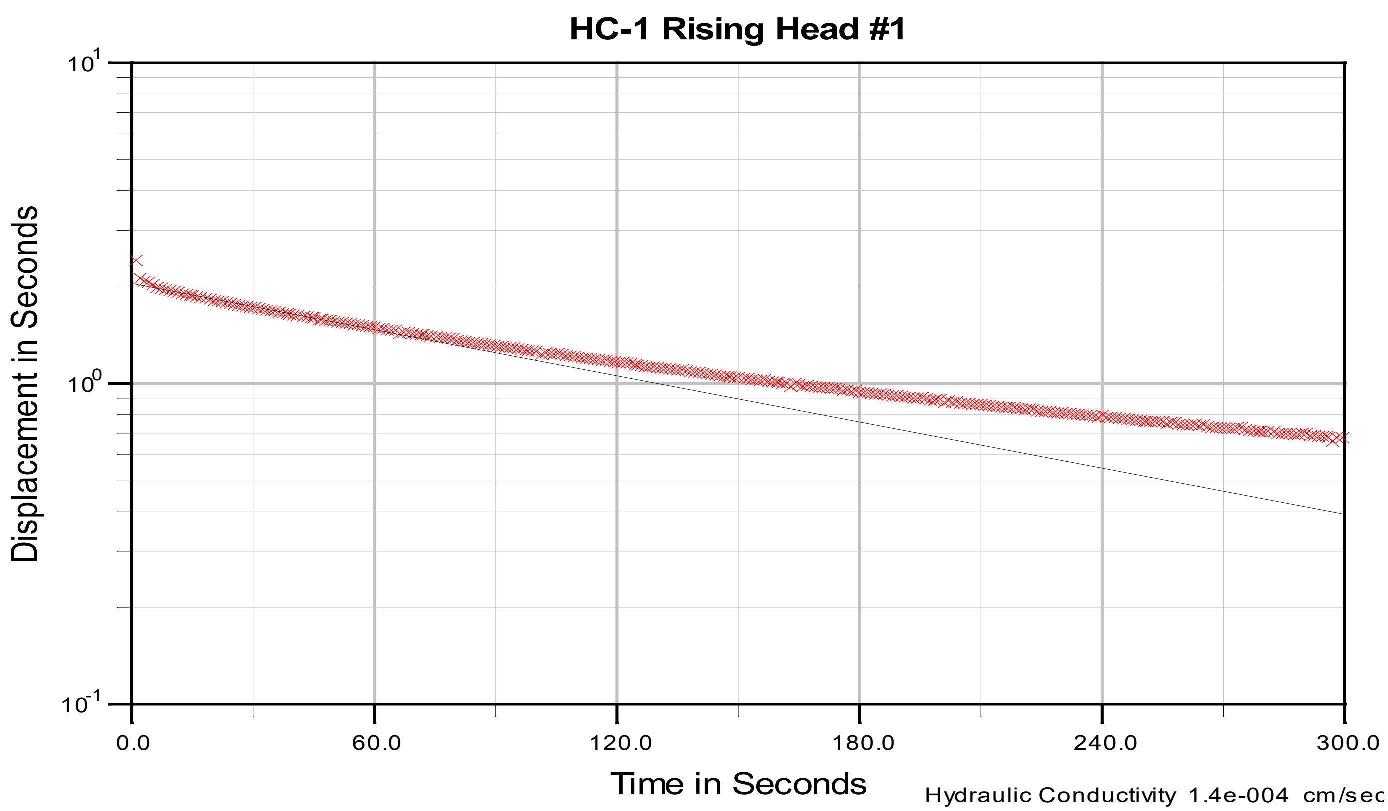

Note:
Bouwer and Rice method was used for the slug test analysis.

Mercer Island Multi-Family Development
Mercer Island, Washington

HC-1 Representative Slug Tests Results

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Figure 3

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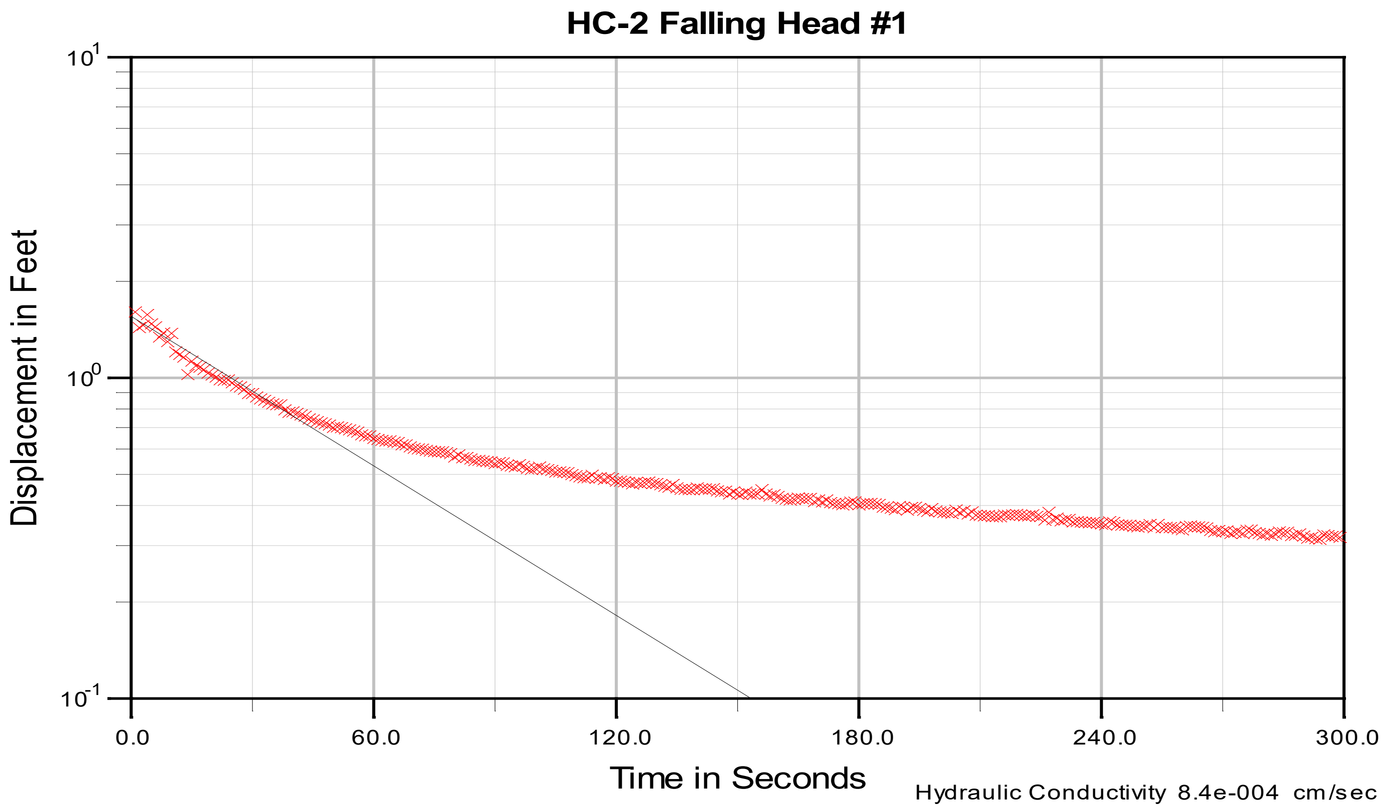

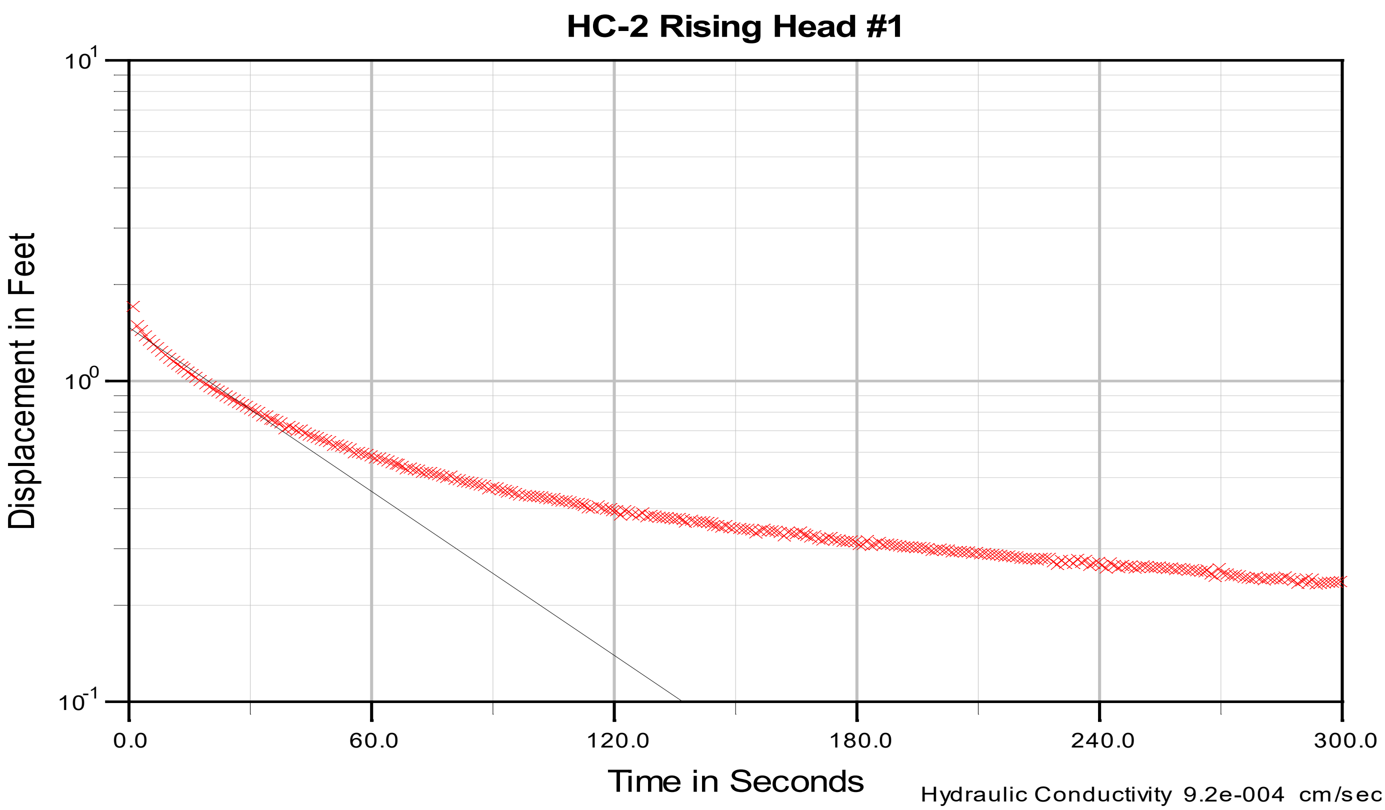

Note:
Bouwer and Rice method was used for the slug test analysis.

Mercer Island Multi-Family Development
Mercer Island, Washington

HC-2 Representative Slug Tests Results

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Figure
4

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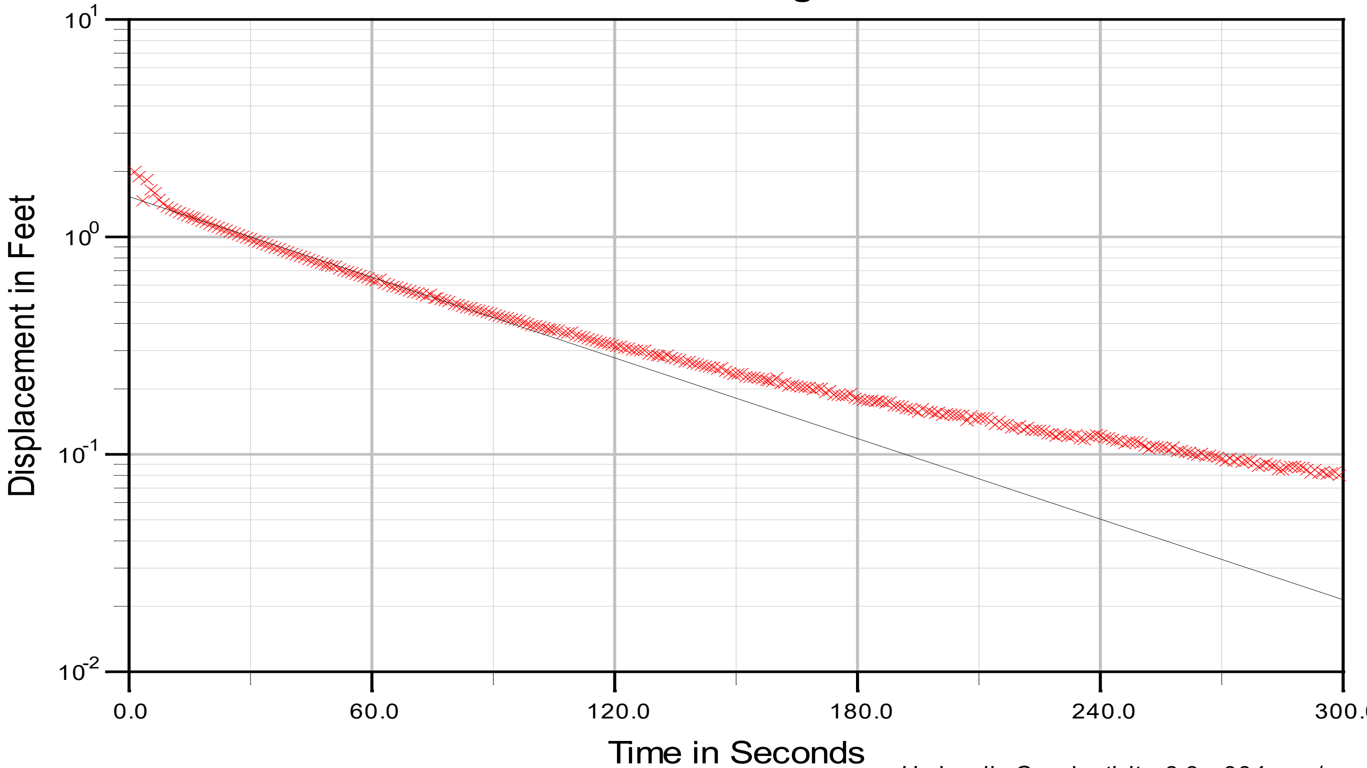
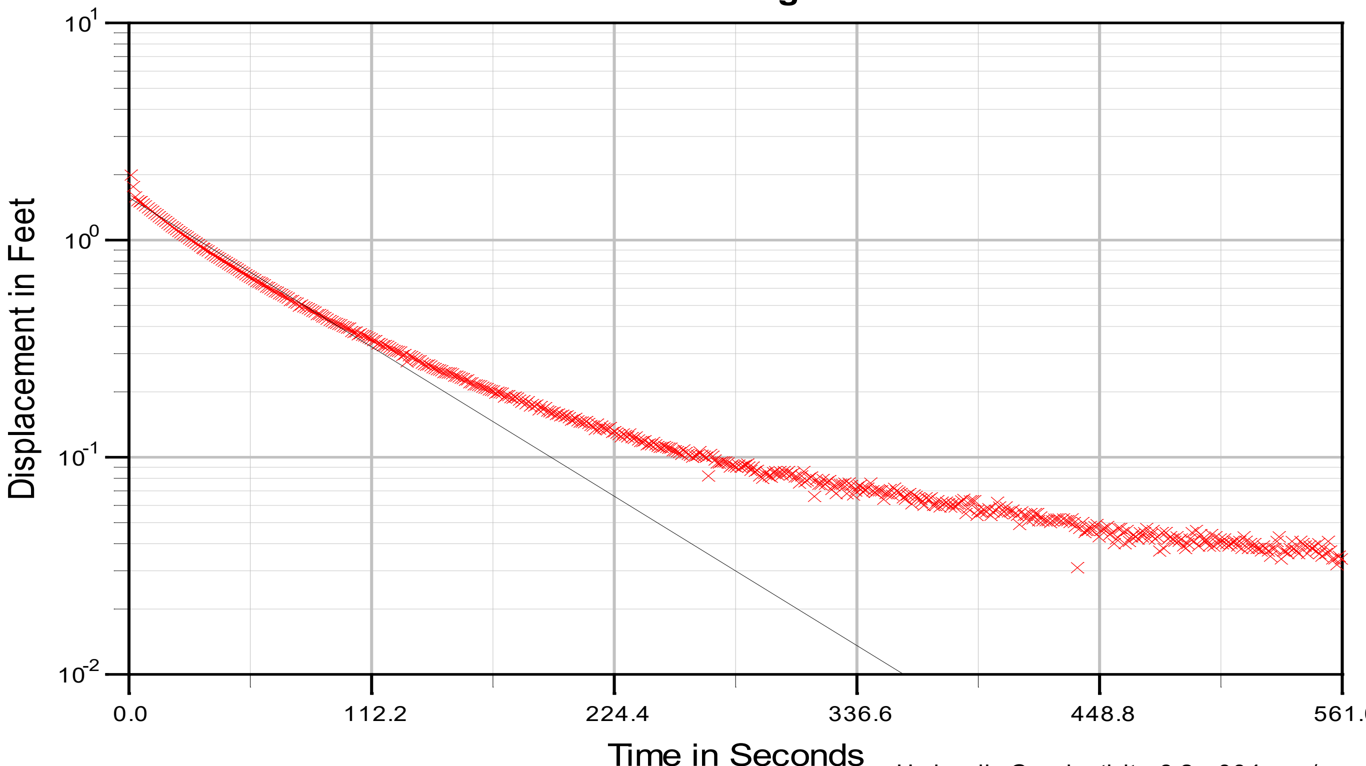
ABPB-M3 - Falling Head #1



Hydraulic Conductivity 6.3e-004 cm/sec

ABPB-M3 Rising Head #1



Hydraulic Conductivity 6.2e-004 cm/sec

Note:
Bouwer and Rice method was used for the slug test analysis.

Mercer Island Multi-Family Development
Mercer Island, Washington

ABPB-M3 Representative Slug Tests Results

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Figure
5

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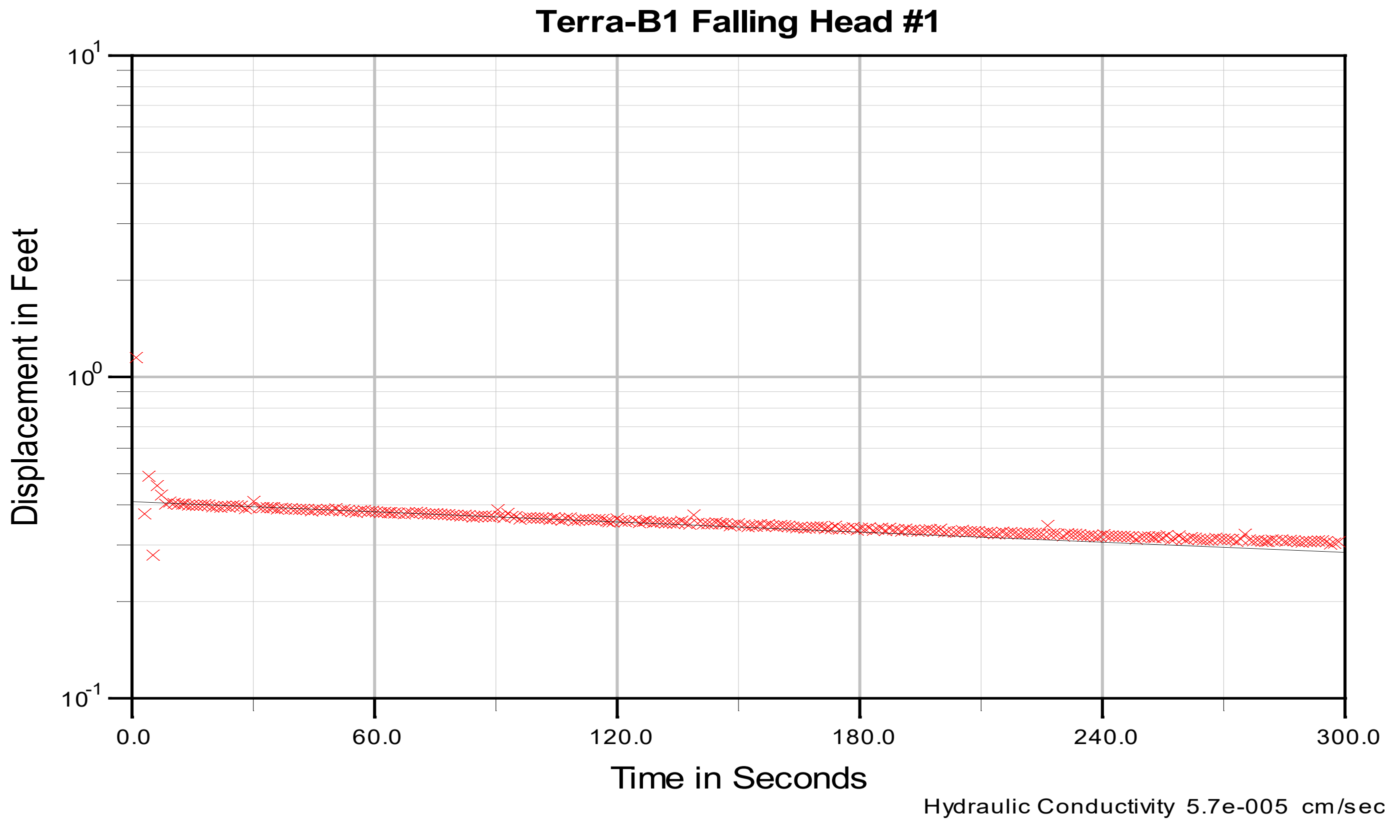

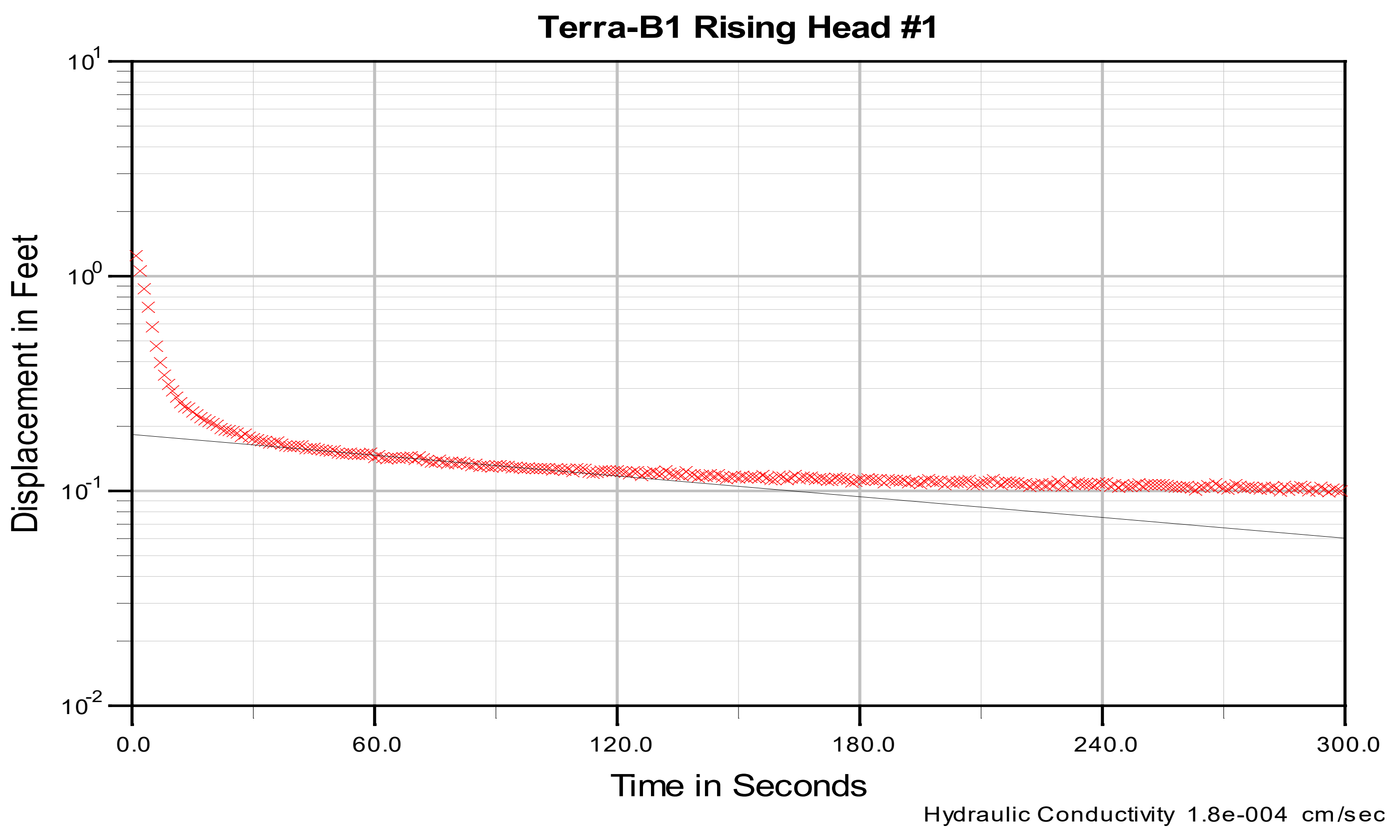

Note:
Bouwer and Rice method was used for the slug test analysis.

Mercer Island Multi-Family Development
Mercer Island, Washington

Terra-B1 Representative Slug Tests Results

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Figure
6

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