v. 1, n. 2, 2021

Journal of
Engineering
Research

CASE STUDY: USE OF CORRELATIONS BETWEEN SPT AND CPT DATA FROM GEOTECHNICAL INVESTIGATION TO UNDERSTAND SOIL PROPERTIES

Rafaela Baldi Fernandes
Departamento de Engenharia Civil - UERJ
Rio de janeiro, RJ, Brazil
ORCID: 0000-0001-6093-9114

Luis Fernando Sieira Castro
Pontifícia Universidade Católica do Rio de Janeiro (PUC-RJ)
Rio de janeiro, RJ, Brazil
ORCID: 0000-0001-6093-9114.

Pietro Salomão de Sá
Instituto de Geociências - USP
São Paulo, SP, Brazil
ORCID: 0000-0002-8152-5102

Abstract: This article present data from a subsurface exploration analysis is ascertaining the various soil profile components at test locations, providing laboratory test results for use by the design engineers in preparing and installation techniques, record groundwater levels at the time of the investigation and discuss the potential impact on the proposed construction. Usually the scope of the exploration and analysis included site geologic research and evaluation, subsurface exploration, field testing and sampling, laboratory testing, geotechnical engineering analysis and evaluation of the subsurface materials.
Keywords: CPT, Site Investigation, Soil parameter.

INTRODUCTION

The principal purpose of subsurface exploration analysis is ascertaining the various soil profile components at test locations, providing laboratory test results for use by the design engineers in preparing and installation techniques, record groundwater levels at the time of the investigation and discuss the potential impact on the proposed construction. Usually the scope of the exploration and analysis included site geologic research and evaluation, subsurface exploration, field testing and sampling, laboratory testing, geotechnical engineering analysis and evaluation of the subsurface materials.

Soil borings and standard penetration tests (SPTs) were conducted in general accordance with ASTM D-5783 (Standard Guide for Use of Direct Rotary Drilling with water based drilling fluid for Geoenvironmental Exploration and the installation of subsurface water quality monitoring devices) and ASTM D-1586 (Standard Test Method for Standard Penetration Test and Split Barrel Sampling of Soils). Relatively undisturbed Shelby tubes were collected on select fine-grained materials in general accordance with ASTM D-1587 (Standard Practice for Thin-Walled Tube Sampling of Fine-grained Soils for Geotechnical Purposes). Due to the small sample size of the standard split spoon or Shelby tube, samples may not accurately quantify the gravel content and/or diameter depict. Reported SPT N-Value of blows over inches indicates refusal at a specified number of blows over a specified distance. In addition, unconfined compressive strength (Qp) values were assessed with a pocket penetrometer within the fine-grained soils. The SPT resistance value N and Qp values are correlated with the engineering behavior of soil to develop earthwork recommendations.

Cone Penetration Testing (CPT) was performed by advancing a cone shaped probe into the ground with an ATV-mounted drill rig in general accordance with ASTM D-5778 (Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soil). The cone is equipped with electronic sensors that measure the tip resistance, side friction, and pore water pressure as the probe is advanced. Measurements are transmitted to a computer at the ground surface that compiles the data. The correlated subsurface soil properties including soil temperature, soil strength, compressibility, and soil classifications are generated based on the results.

Groundwater level observations were recorded during and at the completion of field operations prior to backfilling the borings. Seasonal variations, temperature, anthropogenic activities, seasonality, soil permeability, tidal influences, and precipitation will influence the actual and observed groundwater levels. Groundwater elevations derived from sources other than seasonally observed groundwater monitoring well may not be representative of true groundwater levels.

Soil borings were backfilled using a grout slurry. Low strength grout was mixed with the remaining drilling fluid and circulated through the borehole using the drilling auger. Sodium-base bentonite hole plug chips were installed near the surface and capped with asphalt or concrete patch.

The N_{60} values were calculated based on the SPT N-values obtained during our subsurface investigation. SPT N-values collected in the field were corrected for hammer efficiency, borehole diameter, sampler type and rod length. The correction factors used are included in the Table 1.

Sample Depth (feet)	Sample Depth (meters)	N_{60} Correction Factor
> 32	> 10	1.67
20-32	6-10	1.58
12-30	4-6	1.42
0-13	0-4	1.25

Table 1 – N_{60} correction factor

LABORATORY TESTING PROGRAM

For Physical/Textural Analysis, each sample was visually classified in general accordance with ASTM D-2488 (visual-manual procedure). In addition, representative samples of selected strata encountered were subjected to a laboratory testing program which included moisture content determinations (ASTM D-2216), particle size distribution (ASTM D-6913), Atterberg Limits (ASTM D-4318), Hydrometer Testing and washed gradation analyses (ASTM D-1140) in order to perform supplementary engineering soil classifications in general accordance with ASTM D-2487. The soil strata tested were classified by the Unified Soil Classification System (USCS).

Chemical Composition Testing use representative samples of selected strata encountered were subjected to a laboratory testing program which included electrical resistivity (ASTM G-187), pH in soil (ASTM D-4972), chloride in soil, carbonate in soil (ASTM D-4373), and water-soluble sulfate (ASTM D-516).

Electrical Resistivity Testing (ASTM G-187) was conducted on select samples. Electrical resistivity testing was performed per the ASTM with the utilization of distilled water. After testing at the as-received water content, the soil mass was removed from the testing box. Distilled water was added to make a moist mass that was reformed into the testing box and topped with additional distilled water when necessary to complete saturation. In general, the dry density of this saturated specimen was lower than that of the initial specimen tested at the as-received water content.

Thermal Resistivity Testing (ASTM D-5334) was conducted on select composite samples. Dimensions of samples were all nominally eight inches in height and 2.9 inches in diameter. The actual diameter and initial height of each specimen were measured as well as the initial water content, wet mass, and the final dry mass for the determination of densities. The densities report are the initial densities, as measured. There was generally some change in dry density as the samples dried that was similar to the expected changes a field sample starting at the same condition would experience as it dried. The samples were compacted using a level of effort based the condition indicated per the SPT N-value correlations, loose, medium dense, dense, etc. After compaction, the samples were wetted with tap water to essentially a saturated state and then slowly dried back taking conductivity and mass readings as they dried.

Unconsolidated – Undrained (U-U) Compression Testing: Representative undisturbed samples of selected fine-grained samples were subject to laboratory testing consisting of unconsolidated- undrained compression testing (ASTM D-2850).

CASE STUDY

Field exploration for the Project X investigation was conducted by means 13 soil borings (identified as borings BH) and 13 cone penetrometer tests (identified as CPT). The borings were drilled using mud rotary drilling techniques using a four-inch-diameter auger performed with a truck-mounted drill rig and CPT soundings using a 10 centimeter-squared probe performed with a truck-mounted rig and the specifications and calibrations for the equipment was included on the Drilling Equipment Specifications and Calibrations report. The soil borings and CPT soundings were completed in the presence of an engineer who performed field tests, recorded visual classifications, and collected samples of the various strata encountered. The test locations were located in the field by a professional surveyor. For this article we will show only two borings (BH/CPT-A and BH/CPT-B) and penetrometer tests, enough for the understanding of the methodology to combine the use of data. The Appendix A show the complete data from the boreholes presented in Figure 1 and in a resume.

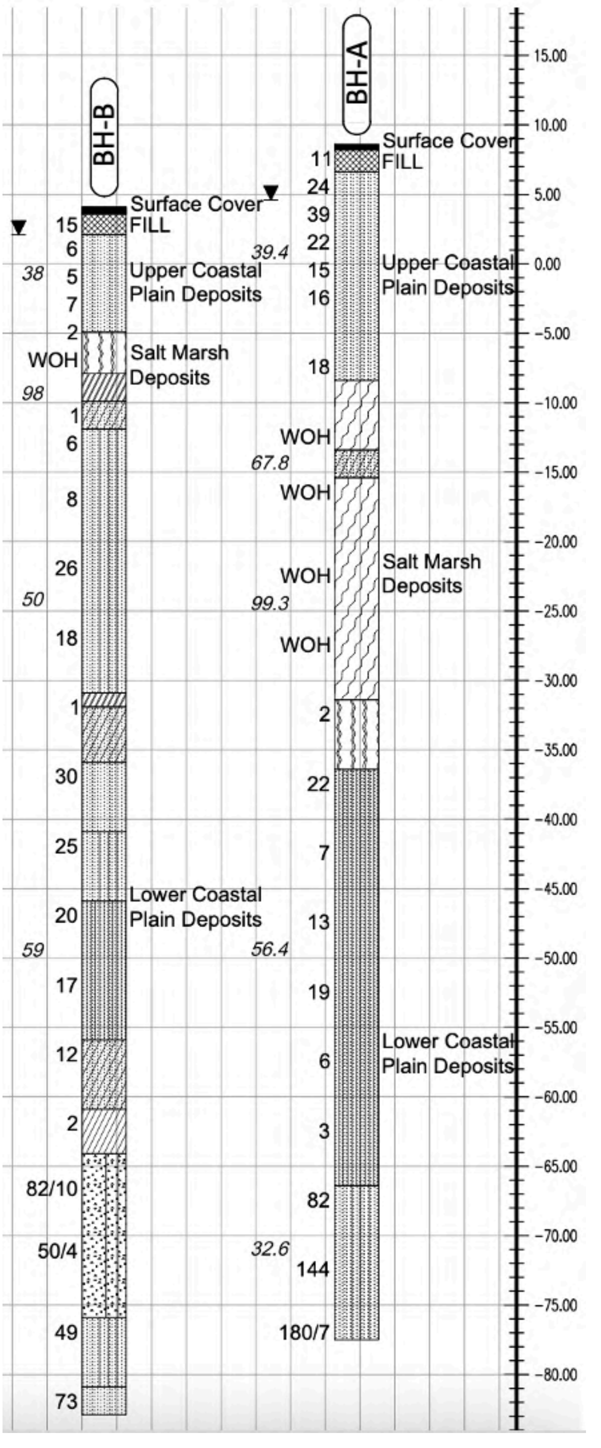


Figure 1 - Profiles

BH/CPT-A

Soil Boring

- Final Depth (feet/meter) = 86.1/26.2
- Reason for termination = Split Spoon Refusal

CPT Sounding

- Final Depth (feet/meter) = 73.3/22.3
- Reason for termination = Direct Push Refusal

BH/CPT-B

Soil Boring

- Final Depth (feet/meter) = 87.0/26.5
- Reason for termination = At Proposed Depth

CPT Sounding

- Final Depth (feet/meter) = 68.4/20.8
- Reason for termination = Direct Push Refusal

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125

At the time of our field investigation the surface cover was developed as two-lane roads with utilities. The existing surface cover included asphaltic concrete and sandy fill material in the shoulder of the roadway and bridge access road. For site drainage the surface runoff generally appears to follow existing site topography toward relatively lower lying.

Table 2 show a detailed summary of the laboratory tests conducted as part of this investigation for BH-A and BH-B. Figures 2 and 3 show the summary results from laboratory analysis and Figures 4 and 5 the summary of corrosion testing. The summary of thermal resistivity testing are at the Figures 6 and 7. The complete results of Sieve Analysis for BH-A is presented at Appendix B and in Appendix C for BH-B.

The Table 3 shows a summary of thermal resistivity analysis and in Appendix D the data from the laboratory tests for BH-A. The Appendix E show the Unconsolidated-Undrained compressive strength for BH-A and BH-B.

SUBSURFACE SOIL PROFILE

The soil borings were performed within pavement areas, shoulder of the roadway and bridge access road encountered approximately four to ten inches of asphaltic concrete at the surface in existing pavement areas, and approximately two to four feet of sandy fill material in the shoulder of the roadway and

Number	Moisture Content	Liquid and Plastic Limits	Sieve Analysis	Hydrometer	Organic Content	Thermal Resistivity	Electrical Resis tivity	Chemical Testing	Water Soluble Sulfate	Bulk and Dry Density	Unconsolidated Undrained Compression Testing
BH-A	20	4	7	4	3	5	3	3	3	7	1
BH-B	16	4	11	7	3	4	3	3	3	1	1

Table 2 – Laboratory Schedules Summary for BH-A and BH-B

BORING NO.	SAMPLE NO.	DEPTH	IDENTIFICATION TESTS															REMARKS / TEST ID
		(ft)	WATER CONTENT (%)	LIQUID LIMIT (-)	PLASTIC LIMIT (-)	PLAS. INDEX (-)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	HYDRO. % MINUS 2 μm (%)	ORGANIC CONTENT (bumoff) (%)	TOTAL UNIT WEIGHT (pcf)	DRY UNIT WEIGHT (pcf)	Type Test	PEAK DEVIATOR STRESS (ksf)	PEAK SHEAR STRENGTH (ksf)	REMOLDED SHEAR STRENGTH (ksf)	AXIAL STRAIN @ PEAK STRESS (%)	
BH-A	S-3	4-6	13.7				SP-SM	7										
	S-6	10-12	17.2															See corrosion summary
	S-(2-7)	2-17	14.8*				SP-SM				124.4*	108.3*						See thermal test
	S-8	20-22	43.4															
	ST-1	22-24									110.3							
	ST-1	22.1	42.1															
	ST-1	22.4	40.8*				OH				111.5*	79.1*						See thermal test
	ST-1	22.85	45.1															
	ST-1B	23.05	34.7	33	23	10	SC	38.2	12	2.1	115.7	85.9	UU@1.5	1.0			5.8	UU205c
	S-9	24-26	47.6															See corrosion summary
	S-10	30-32	99.8	141	61	80	OH	79	38	12.5								
	S-11	35-37	45.2															
	S-12	40-42	73.8	68	27	41	OH	75	30	4.1								
	S-(8-12)	20-42	50.8*				OH				103.4*	68.6*						See thermal test
	S-13	45-47	21.2	37	17	20	CL											
	S-(13-16)	45-62	27.5*				SM				116.9*	91.7*						See thermal test
	S-17	65-67	29.6				SM	33	9									
	S-18	70-72	23.4															See corrosion summary
	S-19	75-77	13.0				SP-SM	6										
	S-21	83-85	8.2				SP-SM	7										
	S-(17-21)	65-87	13.5*				SM				134.4*	118.4*						See thermal test

Figure 2 – Laboratory Testing Data Summary BH-A

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS WATER CONTENT (%)	LIQUID LIMIT (-)	PLASTIC LIMIT (-)	PLAS. INDEX (-)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	HYDRO. % MINUS 2 µm (%)	ORGANIC CONTENT (burnoff) (%)	TOTAL UNIT WEIGHT (pcf)	DRY UNIT WEIGHT (pcf)	Type Test	PEAK DEVIATOR STRESS (ksf)	AXIAL STRAIN @ PEAK STRESS (%)	REMARKS
BH-B	S-2 to S-5A	2-9	23.1				SP	2.3								See corrosion summary See thermal test
	S-5B	9-10	83.4	116	41	75	OH	81	33	7.1						
	S-6	10-12	56.8													See corrosion summary
	ST-1	12-14									105.0					
	ST-1	12.0	87.1													
	ST-1B	12.95														See thermal test
	ST-1	13.3	55.2													
	ST-1C	13.6	48.6	56	23	33	CH	75.9	25	2.2	107.0	72.0	UU@0.7	0.7	8.1	UU280d
	S-7	14-16	34.2				SC	22	9							
	S-8	16-18	25.9													See corrosion summary
	S-10	25-27	25.3				SP-SM	8								
	S-7 to S-11	14-32														See thermal test
	S-12A	35-36	49.3	69	25	44	CH	75	34	3.6						
	S-12B	36-37	20.2				SC	40	13							
	S-13	40-42	15.1													
	S-14 to S-17	45-62	30.2				SM	30	6							See thermal test
	S-18	65-67	29.6	31	19	12	CL	85	13							
	S-19	70-71.3	10.3				SW-SM	7								
	S-21	80-82	19.3				SP-SM	9								

Figure 3 – Laboratory Testing Data Summary BH-B

SAMPLE ID			RESISTIVITY TESTS					CHEMICAL TESTS					REMARKS
Boring No.	Sample No. *	Depth (ft)	Test Method (2)	As-Received Water Content (%)	As-Received Resistivity (kΩ - cm)	@ Minimum Resistivity Water Content (%)	@ Minimum Resistivity Resistivity (kΩ - cm)	pH (ASTM D4972) Distilled Water	pH (ASTM D4972) 0.01 M CaCl Solution	Leachable Chloride (1) ASTM D512 (ppm)	Leachable Sulfate (1) ASTM D516 (ppm)	Rapid Carbonate ASTM D4373 (%)	
BH-A	S-6	10-12	ASTM G57	16.9	3.2	25.4	2.4	8.0	7.4	63	82	0.0	
	S-9	24-26	ASTM G57*	43.3	0.0	59.3	0.0	5.8	5.5	2480	12400	0.0	
	S-18	70-72	ASTM G57	23.8	0.2	40.6	0.2	6.6	6.2	888	2220	0.0	

* Tests performed on sample fraction finer than #8 sieve

KEY: (1) Test results provided by Testing Engineers International

(2) ASTM G57*: Multi point test using G187 equipment to identify minimum resistivity

Figure 4 – Summary of Corrosion Testing BH-A

SAMPLE ID			RESISTIVITY TESTS					CHEMICAL TESTS					REMARKS
Boring No.	Sample No. *	Depth (ft)	Test Method (2)	As-Received Water Content (%)	As-Received Resistivity (kΩ - cm)	@ Minimum Resistivity Water Content (%)	@ Minimum Resistivity Resistivity (kΩ - cm)	pH (ASTM D4972) Distilled Water	pH (ASTM D4972) 0.01 M CaCl Solution	Leachable Chloride (1) AASHTO T290 (ppm)	Leachable Sulfate (1) AASHTO T291 (ppm)	Rapid Carbonate ASTM D4373 (%)	
BH-B	S-2 to S-5A	2-9	ASTM G57	21.5	4.0	28.5	3.5	8.3	7.9	18	<10	0.0	
	S-6	10-12	ASTM G57*	56.7	0.2	82.8	0.2	6.4	6.0	1710	3025	0.2	
	S-8	16-18	ASTM G57*	24.3	0.4	34.7	0.4	7.9	7.6	1340	<10	0.3	

* Tests performed on sample fraction finer than #8 sieve

KEY: (1) Test results provided by Testing Engineers International

(2) ASTM G57*: Multi point test using G187 equipment to identify minimum resistivity

Figure 5 – Summary of Corrosion Testing BH-B

SAMPLE ID			THERMAL RESISTIVITY TESTS							Notes / Remarks
SAMPLE LOCATION	SAMPLE NO.	DEPTH (ft)	Initial WATER CONTENT (%)	Initial DRY UNIT WEIGHT (pcf)	Initial TOTAL UNIT WEIGHT (pcf)	Initial THERMAL RESISTIVITY (°C-cm/W)	@ 5% Water Content THERMAL RESISTIVITY (°C-cm/W)	@ 3% Water Content THERMAL RESISTIVITY (°C-cm/W)	Dry THERMAL RESISTIVITY (°C-cm/W)	
BH-A	S-(2-7)	2-17	14.8	108	124	45	57	63	361	Compacted
	ST-1	22.4	40.8	79	111	75	137	234	302	
	S-(8-12)	20-42	50.8	69	103	93	233	282	288	Compacted
	S-(13-16)	45-62	27.5	92	117	58	1071	1167	1159	Compacted
	S-(17-21)	65-87	13.5	118	134	35	69	93	245	Compacted

Figure 6 – Summary of Thermal Resistivity Testing BH-A

SAMPLE ID			THERMAL RESISTIVITY TESTS								Notes / Remarks
			Initial				After Inundation	@ 5% Water Content	@ 3% Water Content	Dry	
SAMPLE LOCATION	SAMPLE NO.	DEPTH (ft)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	TOTAL UNIT WEIGHT (pcf)	THERMAL RESISTIVITY (°C-cm/W)	THERMAL RESISTIVITY (°C-cm/W)	THERMAL RESISTIVITY (°C-cm/W)	THERMAL RESISTIVITY (°C-cm/W)	THERMAL RESISTIVITY (°C-cm/W)	
BH-B	S (2 to 5A)	2-9	20.8	96	116	46	38				Compacted
	ST-1	12.95	69.8	57	97	98	98				Intact
	S-(7 to 11)	14-32	23.5	99	122	49	50				Compacted
	S-(14 to 17)	45-62	27.8	93	118	62	59				Compacted

Figure 7 – Summary of Thermal Resistivity Testing BH-B

Number	Depth (feet)	Depth (meters)	Thermal Resistivity at Saturation (°C-cm/w)
BH-A	2-17	0.6-5.2	39.4
BH-A	22.4	6.8	65.8
BH-A	20-42	6.1-12.8	99.3
BH-A	45-62	13.7-18.9	56.4
BH-A	65-87	19.8-26.5	32.6
BH-B	2-9	0.6-2.7	38
BH-B	12.95	3.9	98
BH-B	14-32	4.3-9.8	50
BH-B	45-62	13.7-18.9	59

Table 3 – Thermal Resistivity Summary Table for BH-A and BH-B

bridge access road. Beneath the surface cover, existing fill material was encountered that generally consisted of sand with variable amounts of silt and gravel. The existing fill material was encountered to depths ranging between approximately two feet and four feet below the ground surface; corresponding to elevations ranging between approximately elevation 25.7 feet and elevation 0.1 feet. Standard Penetration Test (SPT) N-values ranged between six blows per foot (bpf) and 41 bpf.

Beneath the existing fill material, natural coastal plain deposits that generally consist of upper coastal plain deposits, salt marsh deposits, and lower coastal plain deposits. A breakdown of the coastal plain deposits is detailed below. Beneath the existing fill material, upper coastal plain deposits were encountered that generally consisted of beach and nearshore marine sands consisting of coarse to fine sand (USCS: SP, SM, SP-SM, SW, and SC) with variable amounts of gravel, silt and clay. Interstratified layers of clay deposits were encountered that generally consisted of clay (USCS: CH, and CL) with variable amounts of sand, silt, and gravel. The upper coastal plain deposits were encountered to depths ranging between approximately eight feet and 17 feet below the ground surface; corresponding to elevations ranging between elevation 6.4 feet and elevation -11.6 feet. SPT N-values ranged between weight of hammer (WOH) and 150 bpf, and averaged approximately 18 bpf, generally indicating a medium dense condition. Unconfined compressive strength (Qp) values ranged between 0.25 tons per foot (tsf) and 2.50 tsf, and averaged approximately 1.13 tsf, generally indicating a stiff condition within fine grained soils.

Some borings presented a natural salt marsh deposit encountered interbedded within the upper sand deposits that generally consisted of organic clayey silt (USCS: OL, OH, and PT) with variable amounts of organic fibers, silty clay (USCS: CH) with variable amounts of organic fibers, and clayey sand (USCS: SC) with variable amounts of organic fibers. The natural salt marsh deposits were encountered to depths ranging between approximately 12 feet and 45 feet below the ground surface; corresponding to elevations ranging between elevation -7.9 feet and elevation 36.4 feet. SPT N-values ranged between weight of hammer (WOH) and one bpf. Unconfined compressive strength (Qp) values were approximately 0.25 tons per foot, and averaged approximately 0.25 tsf, generally indicating a very soft condition within fine grained soils.

Beneath the natural salt march deposits lower coastal plain deposits were encountered that generally consisted of Cape May Formation consisting of coarse to fine sand (USCS: SP-SM, SC, SM, and SP) with variable amounts of silt, clay, and gravel, silty clay (USCS: CH, and CL) with variable amounts of sand, and silt, and clayey silt (USCS: ML) with variable amounts of sand, and clay. The natural lower coastal plain deposits were encountered to the termination depths ranging between approximately 18.0 feet below the ground surface and 87 feet below ground surface; corresponding to elevations ranging between elevation -14.1 feet and elevation -83.8 feet. Except where refusal of the split spoon sampler was encountered, SPT N-values ranged between weight of hammer bpf and 180 bpf, and averaged approximately 35 bpf, generally indicating a dense condition. Unconfined compressive strength (Qp) values ranged between approximately 0.25 tons per foot and 0.50 tsf, and averaged approximately 0.30 tsf, generally indicating a soft condition within fine grained soils. The refusal of the split spoon sampler is likely due to the very dense conditions within the lower coastal plain deposits.

CONE PENETRATION TEST RESULTS BH/CPT-A

The Figure 8 shows the results of the tests obtained by BH/CPT-A. According to Schnaid et al (2009), a layer of soft clay is identified by low values of q_t combined with high values of poropressure, while a layer of sand is identified by high values of qt combined with poropressure close to hydrostatics. The results show high q_t resistance values up to 15.0 m deep, and below 40.0 m, combined with poropressures close to the hydrostatics, indicating the presence of sandy soils at these depths. It is also observed the presence of a layer with low tip resistance values and with generation of poropressures, between 16.0 m and 40, m in depth, indicating the presence of soft clay materials.

The classification of soils through the piezocone test is commonly done in the form of abacuses. This method uses the values measured in the piezocone tests (q_t or q_c, f_s and u_2), allowing the characterization of the type of soil. The classification of soils through the direct determination of their granulometric characteristics is not possible due to the absence of sample collection during the test. Consequently, the classification of soils by means of cone tests is done indirectly. The classification procedure is established based on standards of behavior and defined by the acronym SBT (Soil Behavior Type).

Robertson & Campanella (1983) present abacuses in which they relate R_f and q_c, and B_q and q_t (Figure 9). However, these classification procedures do not consider increase in the values of tip resistance and lateral friction with depth due to confinement stresses. Therefore, graphs expressed as a function of normalized parameters were included in order to correct

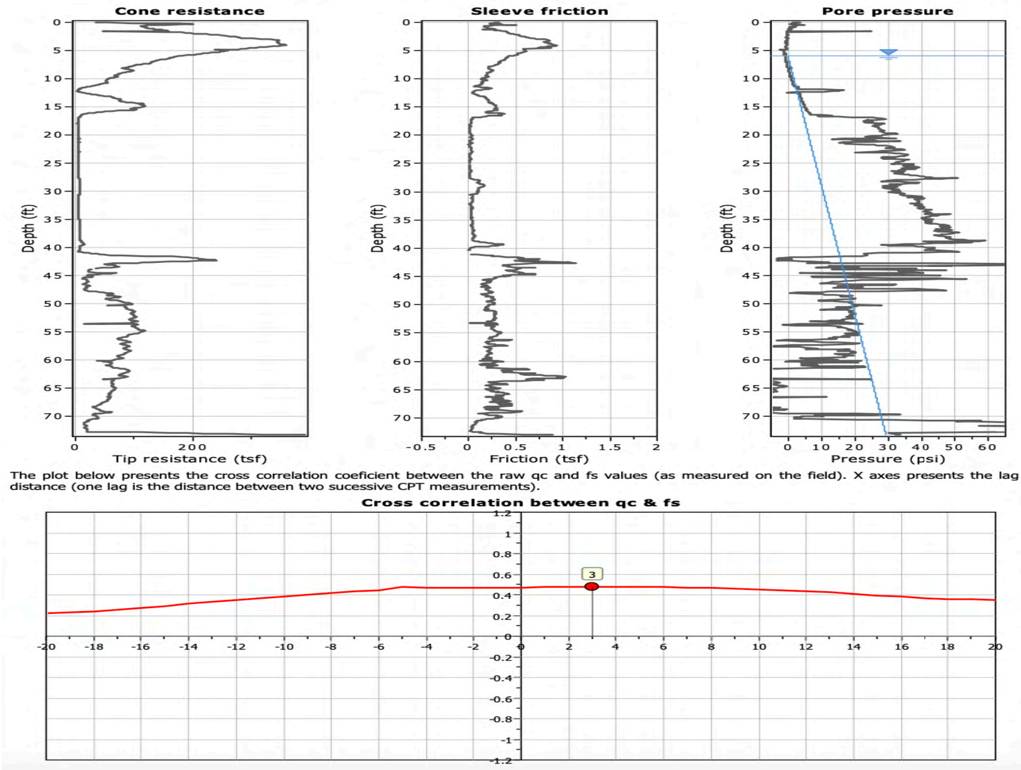


Figure 8. BH/CPT-A test results

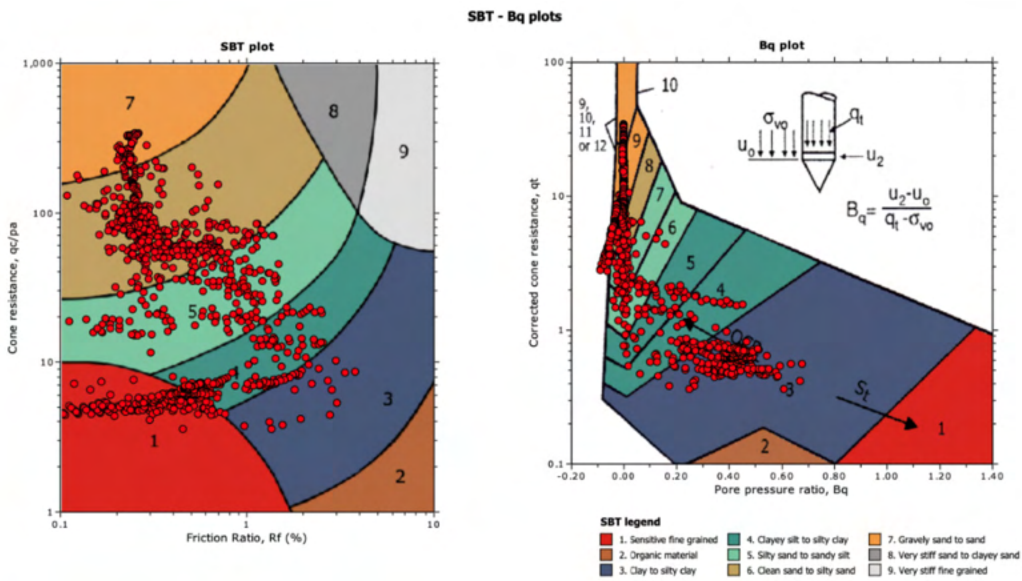


Figure 9. Classification system (Robertson & Campanella, 1983)

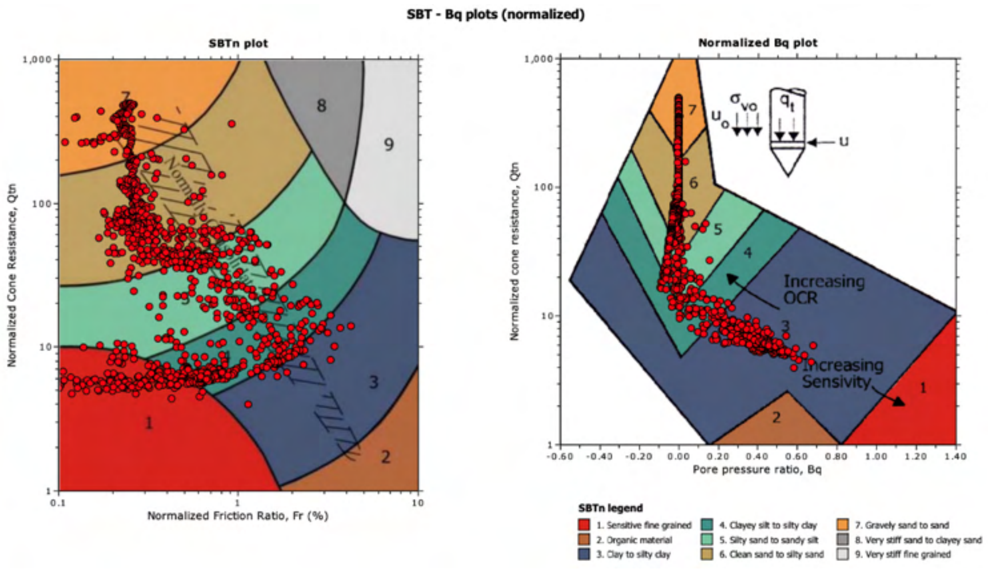


Figure 10. Classification system proposed by Robertson (1990)

these effects. Robertson (1990) proposes the inclusion of the poropressure parameter (B_q) and expands the method with results plotted in two abacuses, $Q_t \times F_r$ (%) and $Q_t \times B_q$, as can be seen in Figure 10.

The normalized tip resistance (Q_t) and the normalized friction ratio (F_r) are defined by:

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$$

$$F_r = \frac{f_s}{q_t - \sigma_{vo}}.100\%$$

Where:σ'_{vo} = effective vertical tension.

In this proposal, nine zones are classified, which have the purpose of identifying materials of different types of behavior, as shown in Table 4.

Zone	Types of Soil
1	Sensitive fine soil
2	Organic soil and peat moss
3	Clay - silty clay
4	Silty clay - clay silt
5	Sandy silt - silty sand
6	Clean sand- silty sand
7	Sands with boulders - sand
8	Sand - clean sand
9	Fine rigid sand

Table 4 - Soil classification by type of behavior

Figure 11 and Figure 12 show, respectively, the classification of soils according to the original proposal and the modified proposal.

Schneider et al. (2008) proposed an abacus for soil classification based on standardized data of tip resistance (Q) and excess pore-pressure (Δu_2 / $\sigma' v_o$). These abacuses were developed using parametric studies of analytical solutions, field data and in the judgment of several discussions between them: soil type, penetration speed, drained, undrained, partially drained behavior, dilation and compression. According to Schneider et al. (2008) this classification proposal based

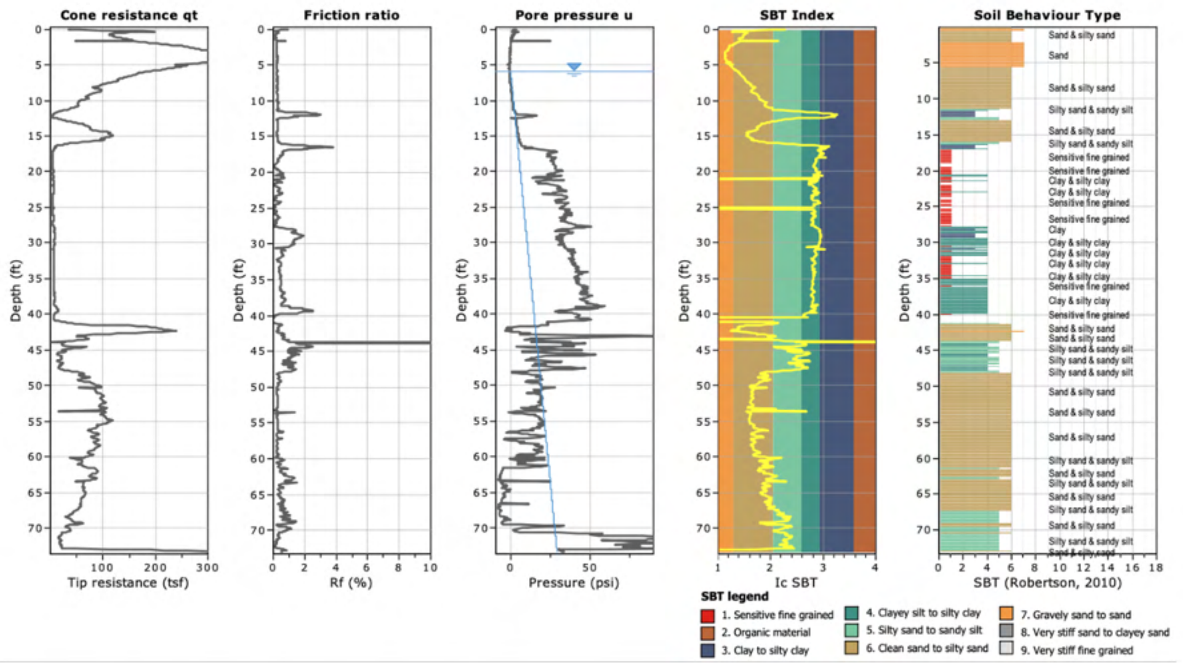


Figure 11. Soil classification based on R_f and q_c, and B_q and q_t (SBT).

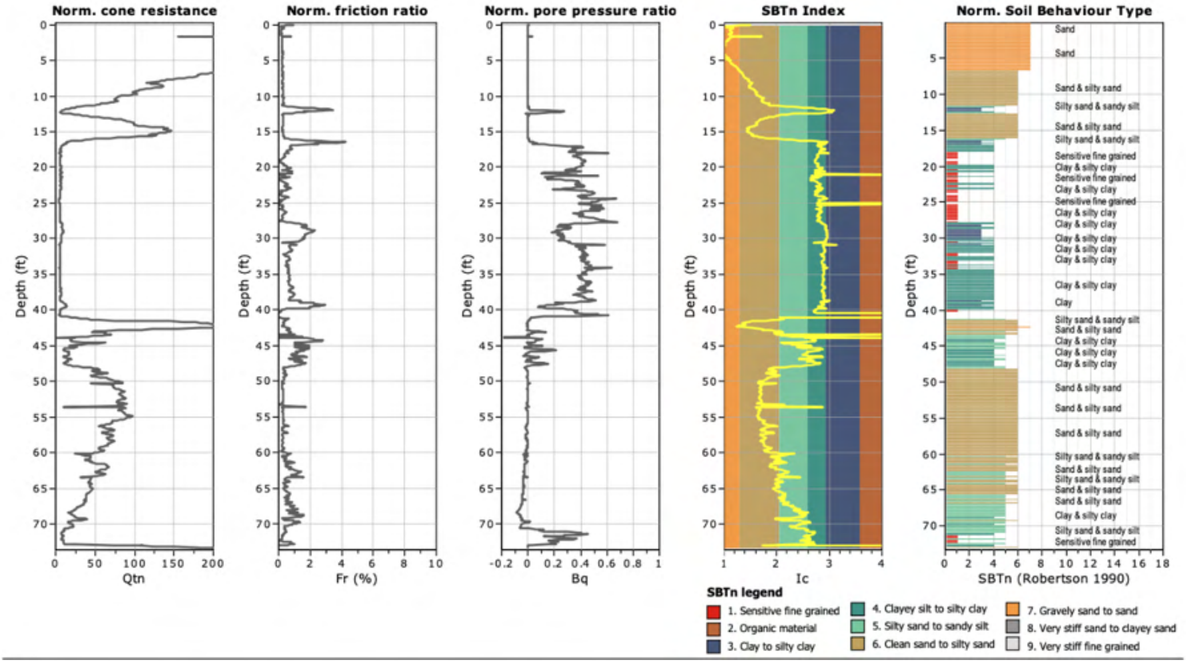


Figure 12. Classification of soils based on $Q_t \times F_r$ (%) and $Q_t \times B_q$ (SBTn).

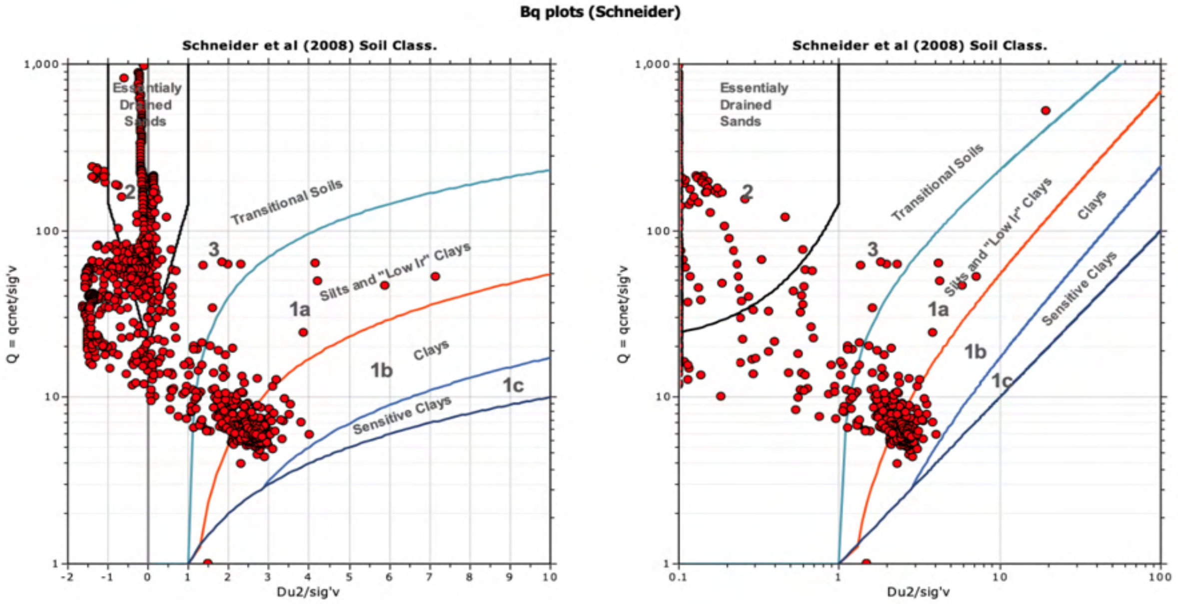


Figure 13. Classification proposal by Schneider et al. (2008)

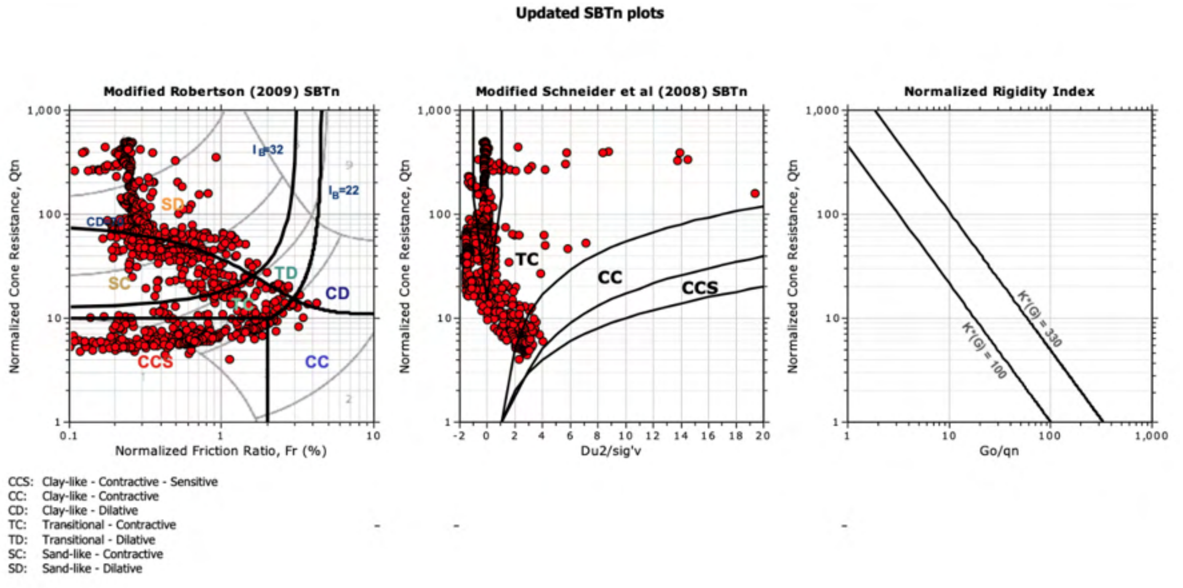


Figure 14. Classification according to Schneider et al. (2008)

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125

on the abacus Q-(Δu_2 / σ'vo) was developed mainly to help the separation between the drained, undrained and partially drained penetration.

The proposal by Schneider et al. (2008) is shown in Figure 13, which illustrates the soil classification abacuses, based on standardized piezocone parameters. Figure 14 shows the classification of soils according to the proposal by Schneider et al (2008).

Jefferies and Davies (1993) also defined the material classification index (I_c = material classification index) shown in the equation:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

The geotechnical parameters obtained differ according to the type of material present at the test in site. In clays, the usual correlations used in the interpretation of piezocone tests are: the estimate of undrained resistance (S_u), the history of stresses (OCR), the state of stresses (k_o), the undrained deformability modules (Eu) , oedometric (M) and shear to small deformations (G_0), and the density coefficients (c_h and c_v). In sands, the interpretation of the results provides an estimate of the shear strength parameters (Dr and ϕ'). Permeability parameters (k) can be estimated in a common way to different types of soils, as well as the parameters of stiffness to small deformations (E_0 and G_0), estimated in a non-destructive way through the additional measurement of the speed of the shear wave. (V_s).

Table 5 presents the methodology adopted to estimate the main geotechnical parameters from the results of piezocone tests. From the expressions presented in this table, the distribution of the different parameters is obtained along the test depth, as shown in Figures 15, 16 and 17.

Finally, the carrying capacity of the soils can be defined based on the methodology presented in Figure 18.

BH/CPT-B

The results of BH/CPT-B were interpreted based on the methodology presented in Item 2.2.1. As the figures 9 and 10, the Figures 19 and 20 also show respectively, the classification of soils (Table 4) according to system proposed by Robertson (1990).

The results shown in the Figure 21 and Figure 22 shows a predominance of "essentialy drained sand" with a decrease of the distribution to the other samples.

In the Figure 23 the results show high qt resistance values and medium poropressure values bellow 8.00 m and after 18.00 m until 42.00 meter indicates the presence of sand and silly sand with intercalated layers of silly sand and silty sand. After that occurs a decrease in the interval of 42.00 to 44.00 meters, after 44.00 m 58.00 m the results show high qt resistance and low poropressure values either indicating the presence of sand and silly sand.

The Figure 24 shows by another method almost the same classification of soils of figure 23, but in the gap of 8.00 to 14 m shows that the second method classified the soil with rougher grained classification and brought a rougher behavior to the soil classification.

The plots in the Figure 26 have shown the soil classification with the normalized cone resistance with parameters of Normalized Friction Ratio, du2/sig'v and Go/qn. The plots show a predominance of sand-like – dilative soil and transitional – contractive soil in the model of Robertson (2016). In the system of Schneider (2008) transitional contractive is where the predominance of plots are located. The soil have a significant amount of microstructure as the normalized rigidity test shows.

With all the tests realized for this sample, the load capacity of the soil was made and the results were shown in the Figure 30.

Parameter	Symbol	Expression	Unity
Unit Weight	y	$\gamma = \gamma_w . \left(0.27.\log(R_f) + 0.36.\log\left(\frac{q_t}{P_a}\right) + 1.236\right)$	kN/m^3
Permeability	k	$I_c<3.27$ and $I_c>1.00$ then $k=10^{0.952-3.04.Ic}$ $I_c\leq4.00$ and $I_c> 3.27$ then $k=10^{-4.52-1.37.Ic}$	m/s
Penetration Resistance	N_{SPT}	$N_{60} = \left(\frac{q_c}{P_a}\right).\frac{1}{10^{1.1268-0.2817.Ic}}$ $N_{1(60)} = Q_{tn} = \frac{1}{10^{1.1268-0.2817.Ic}}$	Blows/30 cm
Young's Modulus	Es	$(q_t - \sigma_v).0.15.10^{0.55.Ic+1.68}$ Applicable only to $I_c<I_{c\_cutoff}$	MPa
Relative Density	Dr	$100.\sqrt{\frac{Q_{tn}}{k_{DR}}}$ (Applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$	%
State Parameter	Ψ	$\Psi = 0.56-0.33.\log(Q_{tn,cs})$	
Peak drained friction angle	φ	$\varphi =17.60+11.\log(Q_{tn,cs})$ Applicable only to SBT_n: 5, 6, 7 and 8	(º)
1-D constrained modulus	M	If $I_c>2.20$ $a=14$ for $Q_{tn}>14$ $a= Q_{tn}$ for $Q_{tn}\leq14$ $M_{CPT}=a.(q_t-\sigma)$ If $I_c\leq2.20$ $M_{CPT}=(q_t-).0.0188 . 10^{0.55.Ic+1.68}$	MPa
Small strain shear Modulus	G_0	$G_{0=}.(q_t-\sigma). 0.0188 . 10^{0.55.Ic+1.68}$	MPa
Shear Wave Velocity	V_s	$V_s=(\frac{G_0}{\rho})^{0.50}$	m/s
Undrained peak shear strenght	S_u	$N_{kt}=10.50 +7.\log(F_r)$ or user defined $S_u=\frac{(qt-\sigma_v)}{Nkt}$ Applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c>I_{c\_cutoff}$	kPa
Remolded undrained shear strenght	$S_{u(rem)}$	$S_{u(rem)}=f_s$ Applicable only to SBT_n: 1,2, 3, 4 and 9 or or $I_c>I_{c\_cutoff}$	kPa
Overconsolidation Ratio	OCR	$k_{OCR}= \left[\frac{Q_{tn}^{0.20}}{0.25.(10.50+7.\log(F_r))}\right]^{1.25}$ or user defined $OCR= k_{OCR}.Q_{tn}$ (Applicable only to SBT_n: 1,2, 3, 4 and 9 or or $I_c>I_{c\_cutoff}$)	-
In situ Stress Ratio	K_0	$K_0=(1-\sin\varphi').OCR^{sinj'}$ (Applicable only to SBT_n: 1,2, 3, 4 and 9 or or $I_c>I_{c\_cutoff}$)	-
Soil Sentivity	S_t	$S_t=\frac{N_S}{F_r}$ (Applicable only to SBT_n: 1,2, 3, 4 and 9 or or $I_c>I_{c\_cutoff}$)	-
Effective Stress Friction Angle	φ'	$\varphi' = 29.5^o . B_q^{0.121}$.(0.256+0.336.B_q+logQ_t Applicable for $0.10<B_{q<}1.00$	(º)

Table 5. Determination of geotechnical parameters from CPTu tests

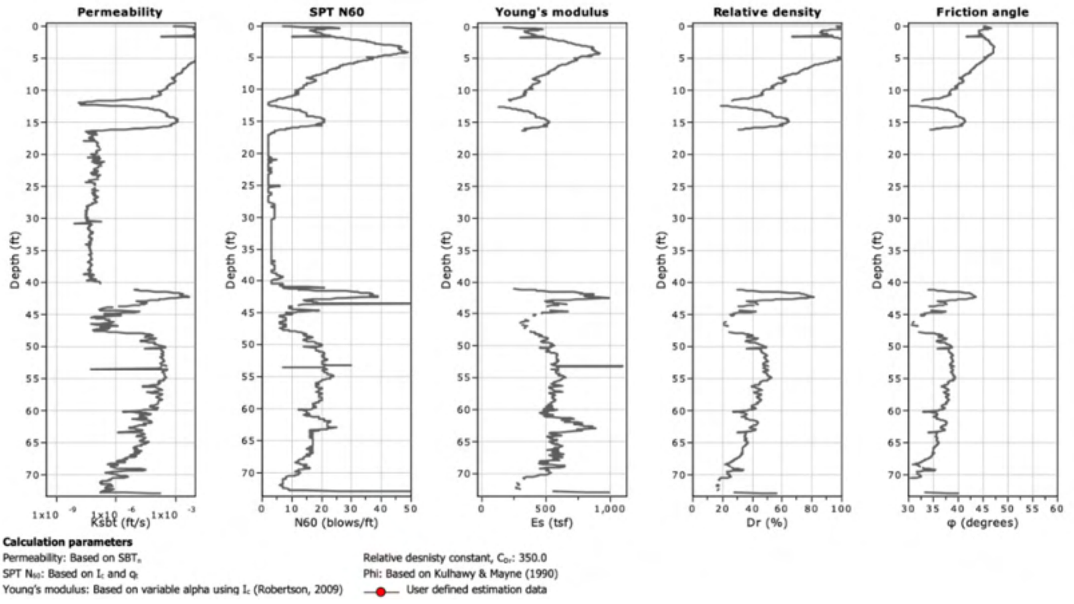


Figure 15. Distribution of k parameters. N60, E, Dr and ϕ with depth

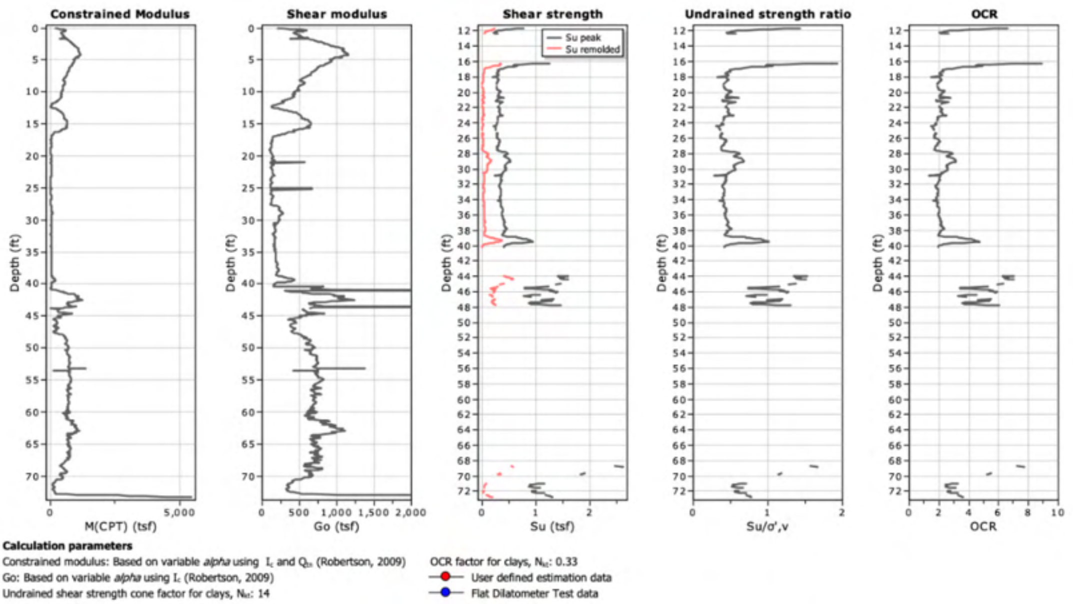


Figure 16. Distribution of parameters M, G_0, S_u and OCR with depth

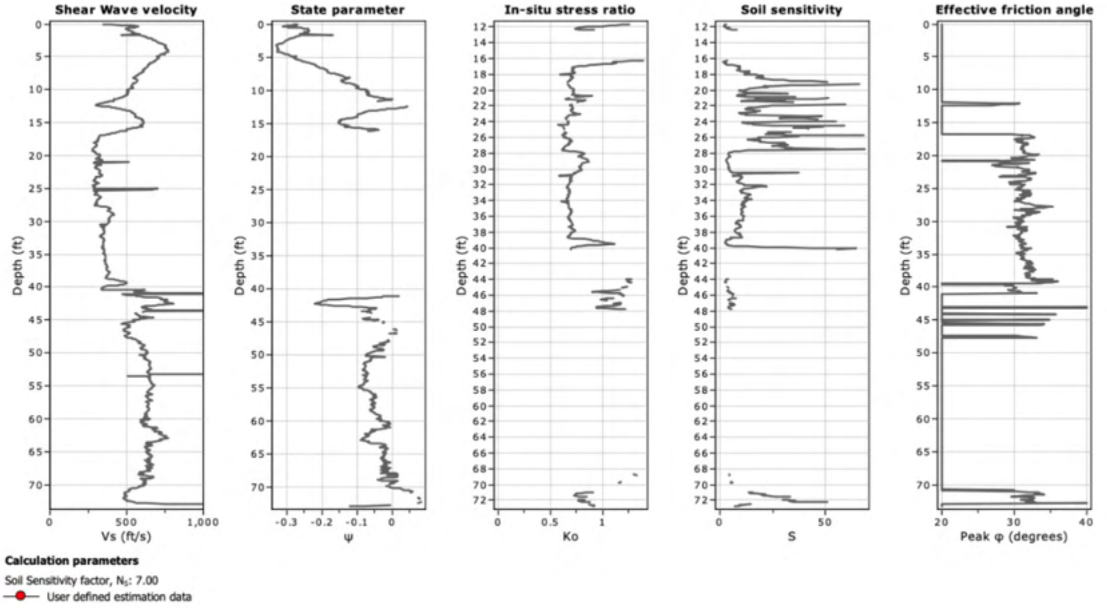
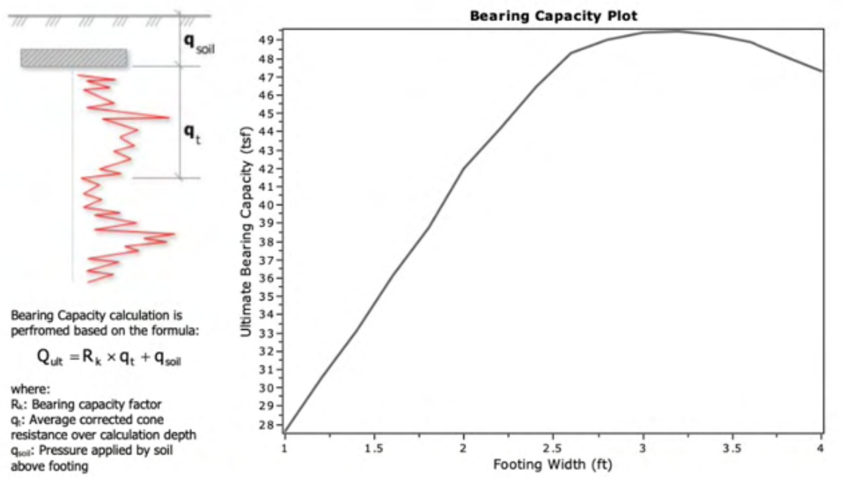


Figure 17. Distribution of parameters V_s, Ψ, k_o, S and peak ϕ with depth



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	1.00	0.50	2.00	138.07	0.20	0.03	27.64
2	1.20	0.50	2.30	152.47	0.20	0.03	30.52
3	1.40	0.50	2.60	165.33	0.20	0.03	33.10
4	1.60	0.50	2.90	180.35	0.20	0.03	36.10
5	1.80	0.50	3.20	193.81	0.20	0.03	38.79
6	2.00	0.50	3.50	209.86	0.20	0.03	42.00
7	2.20	0.50	3.80	220.49	0.20	0.03	44.13
8	2.40	0.50	4.10	232.16	0.20	0.03	46.46
9	2.60	0.50	4.40	241.28	0.20	0.03	48.29
10	2.80	0.50	4.70	245.12	0.20	0.03	49.05
11	3.00	0.50	5.00	246.82	0.20	0.03	49.39
12	3.20	0.50	5.30	247.26	0.20	0.03	49.48
13	3.40	0.50	5.60	246.30	0.20	0.03	49.29
14	3.60	0.50	5.90	244.33	0.20	0.03	48.90
15	3.80	0.50	6.20	240.50	0.20	0.03	48.13
16	4.00	0.50	6.50	236.25	0.20	0.03	47.28

Figure 18. Load capacity of soils

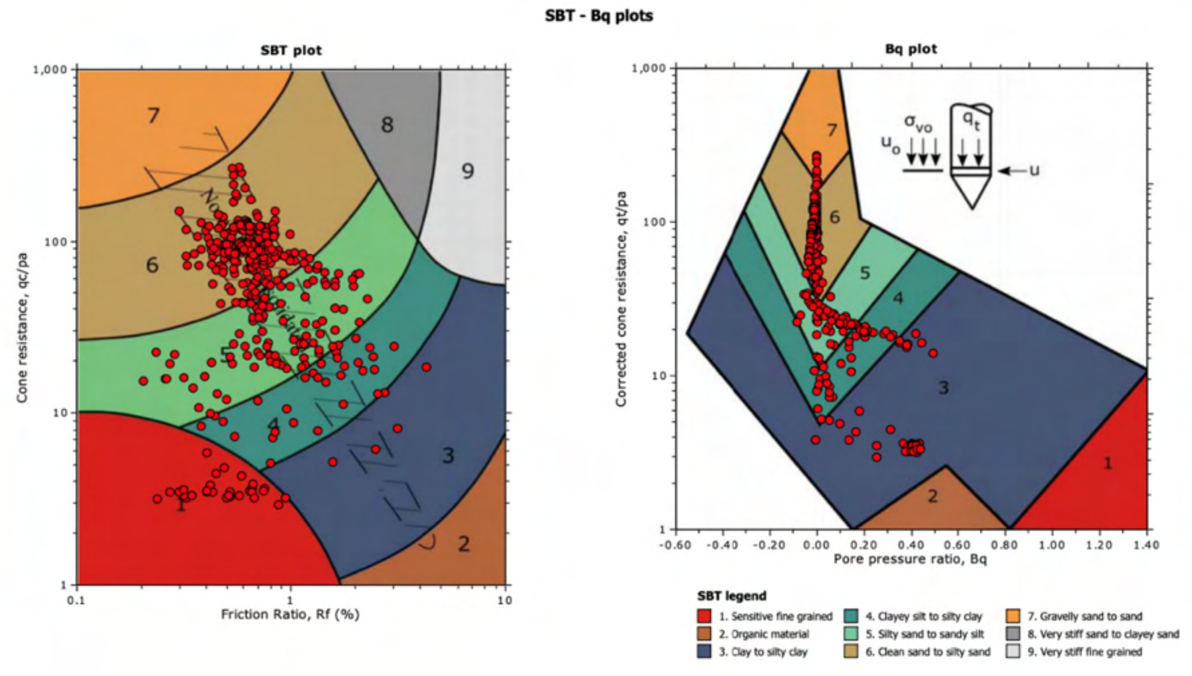


Figure 19: Classification system proposed by Robertson (1990)

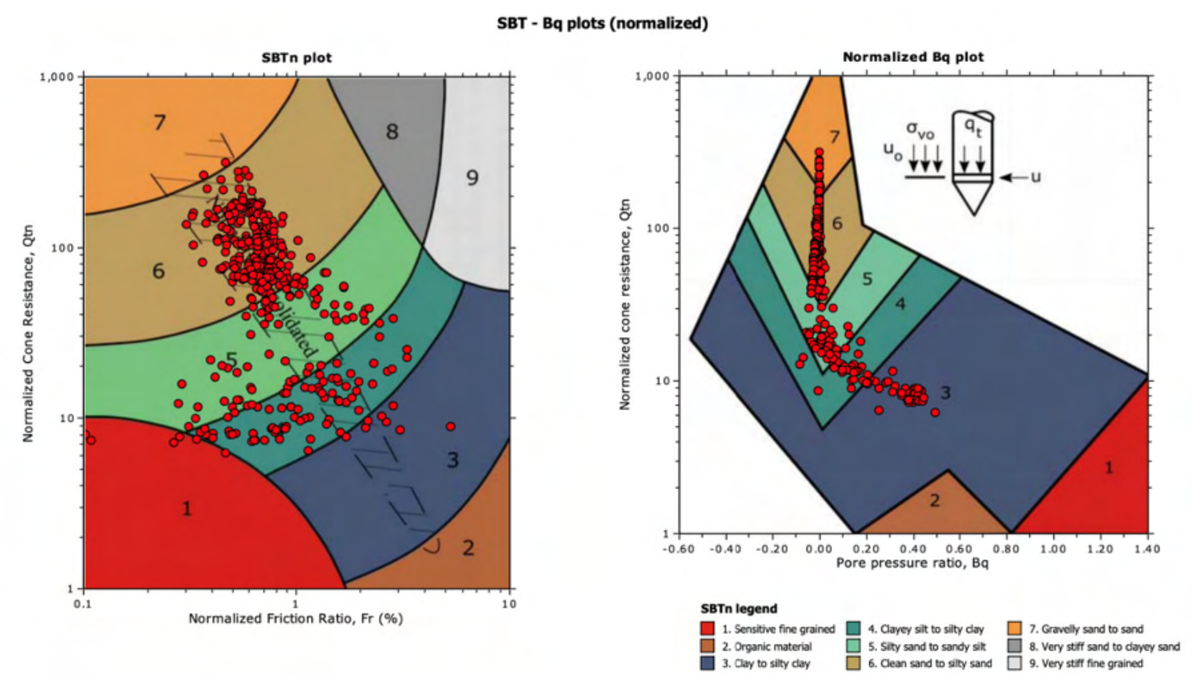


Figure 20: Classification system proposed by Robertson (1990)

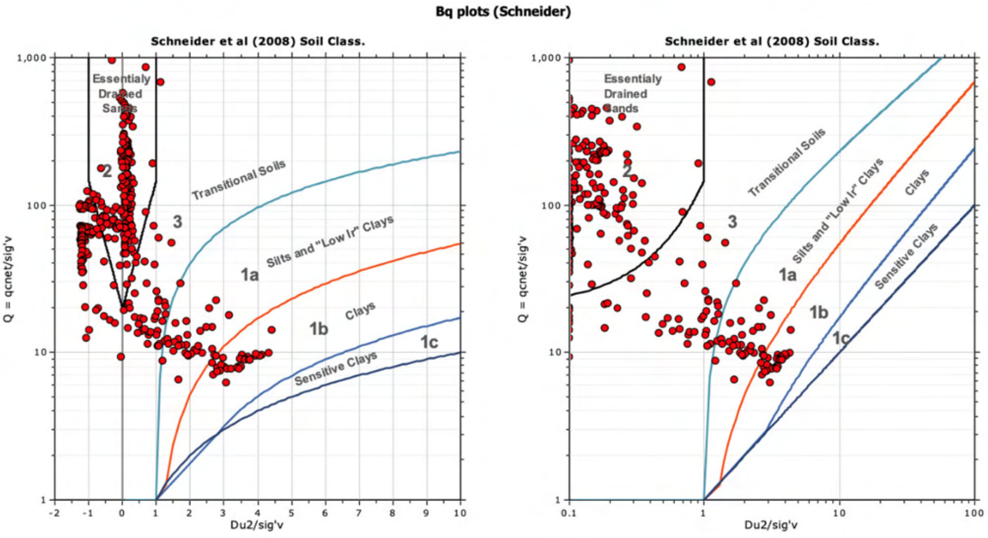


Figure 21: Classification proposal by Schneider et al. (2008)

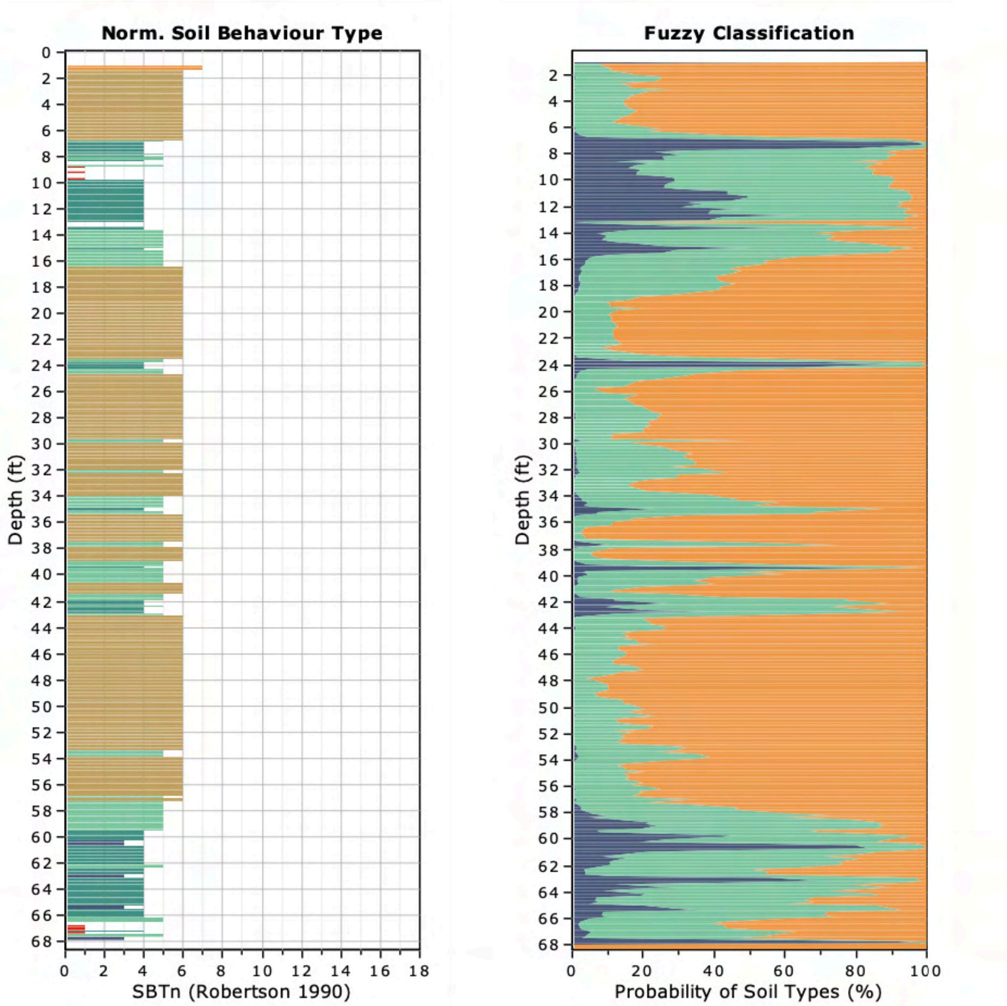


Figure 22: Classification proposal by Robertson 1990 and the aproach of the fuzzy classification.

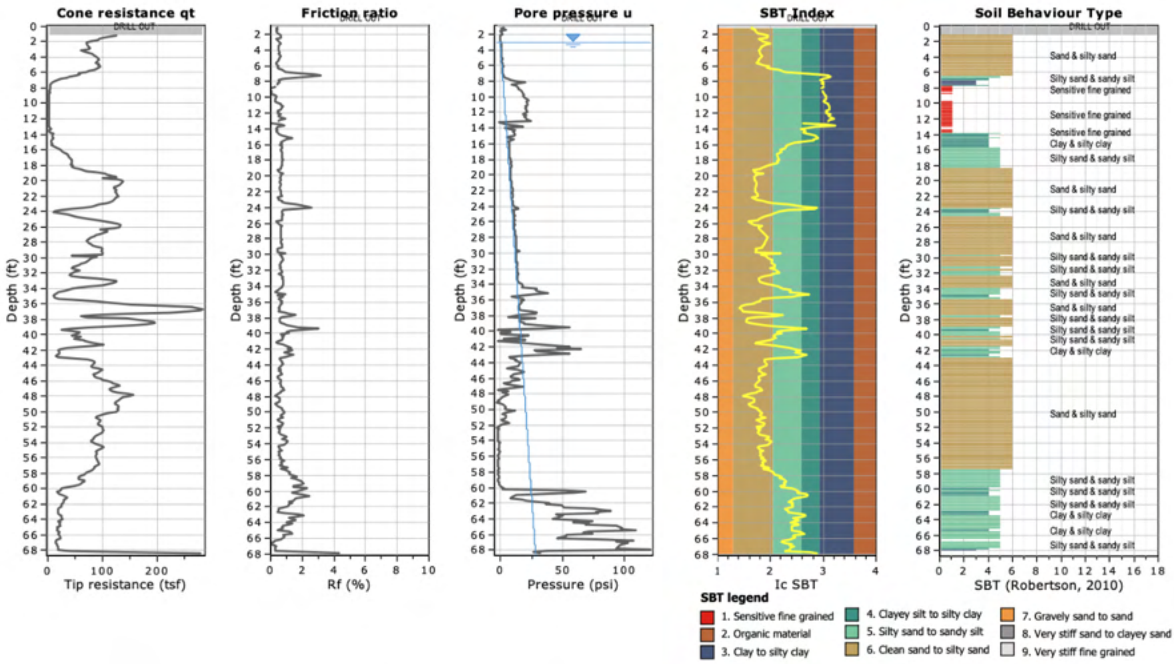


Figure 23. Soil classification based on R_f and q_c, and B_q and q_t (SBT).

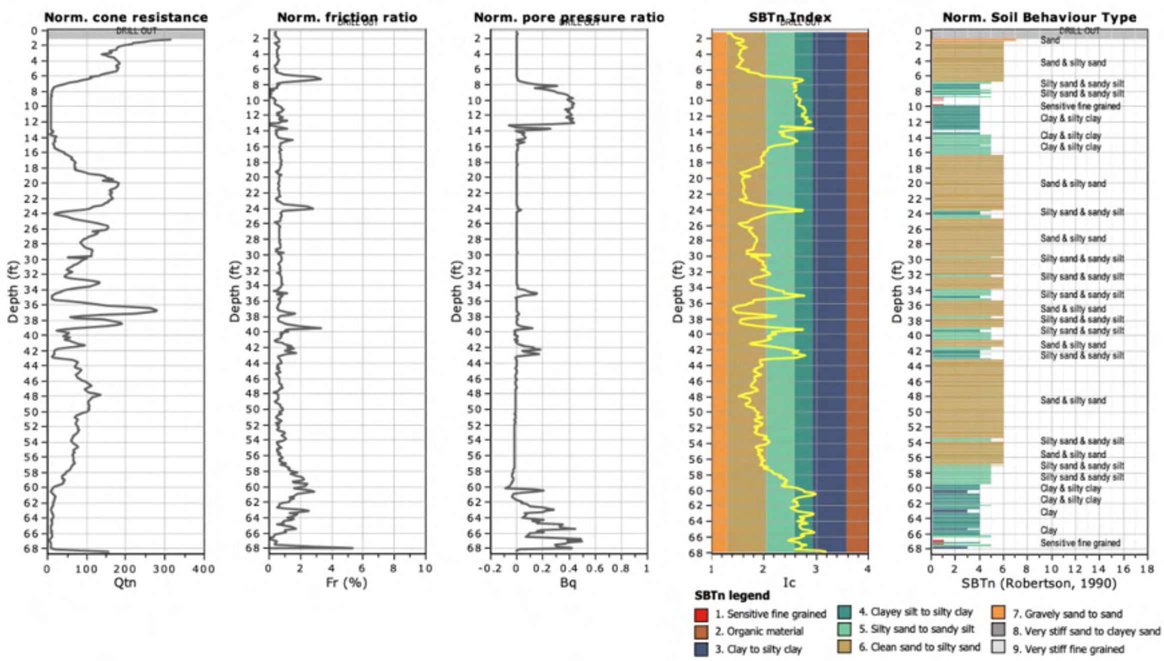


Figure 24. Classification of soils based on $Q_t \times F_r$ (%) and $Q_t \times B_q$ (SBTn)

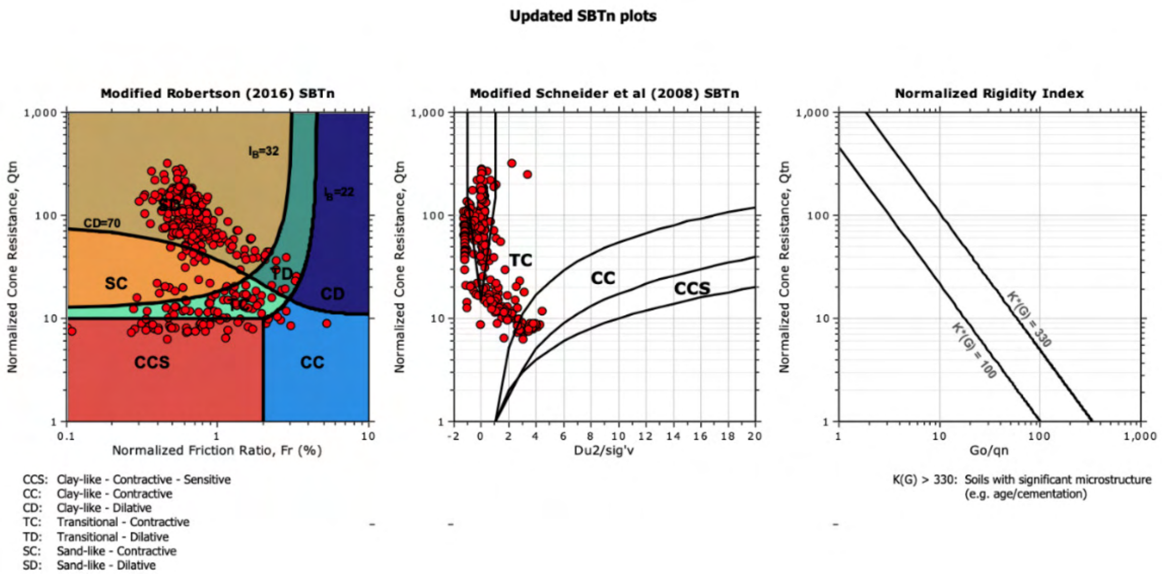


Figure 26: Modified classification system by Robertson (2016) and modified classification system by Schneider (2008).

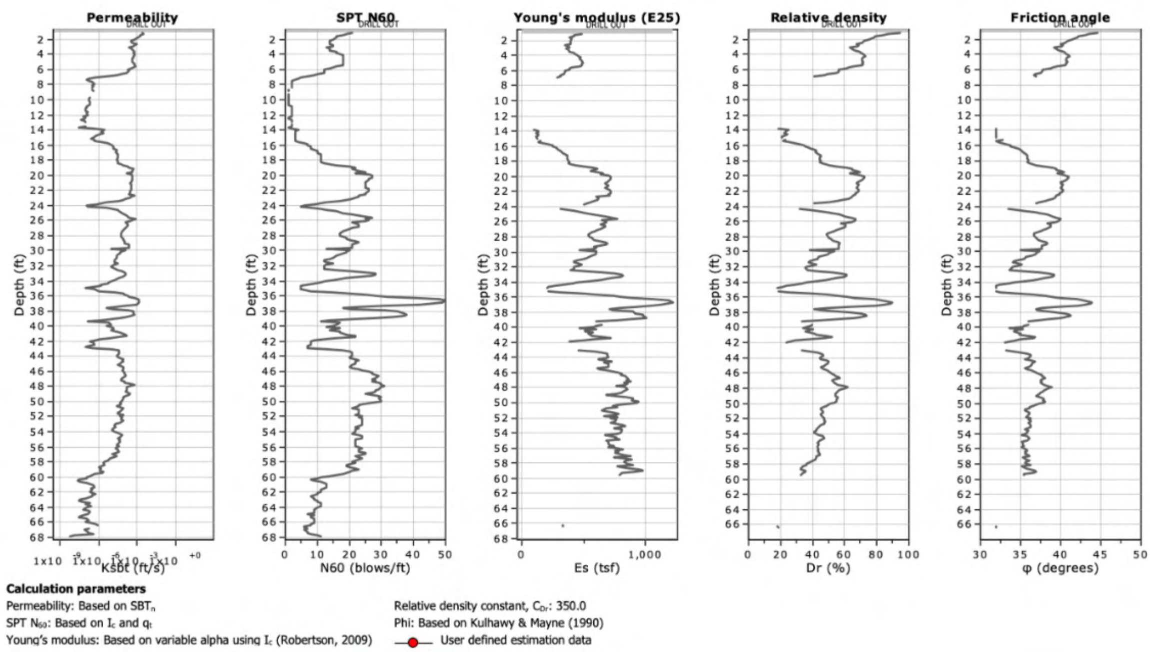


Figure 27: Permeability, SPT N60, Young´s modulus (E25), Relative density and Friction angle test.

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125

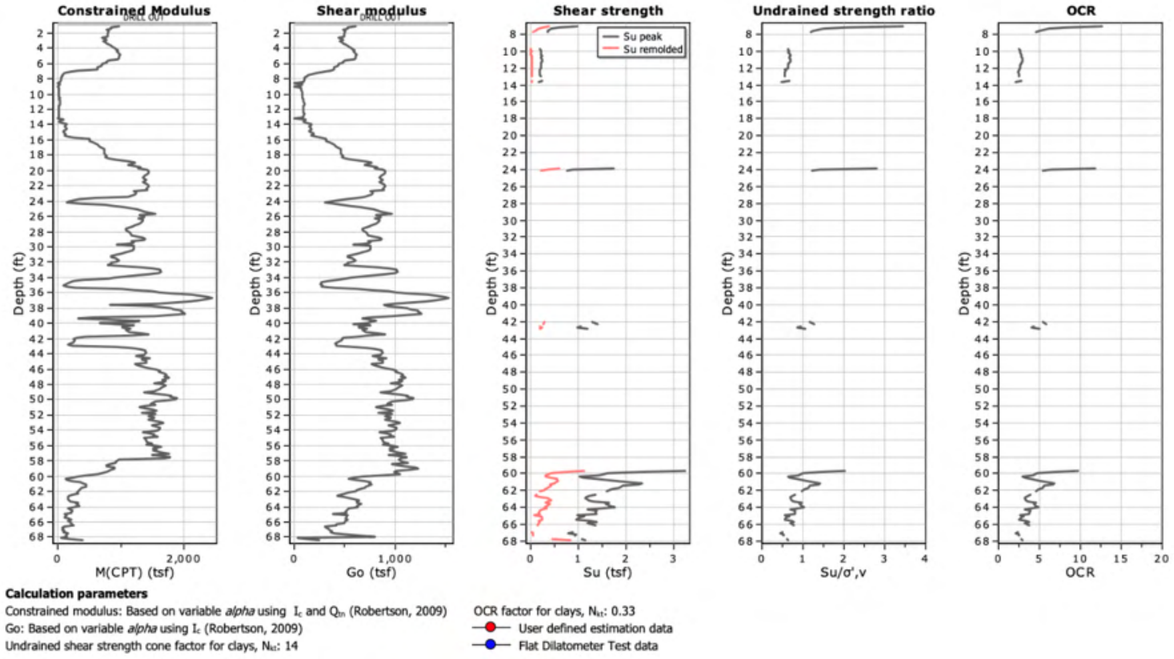


Figure 28: Constrained Modulus, Shear modulus, Shear strength, Undrained strength ratio and OCR Test.

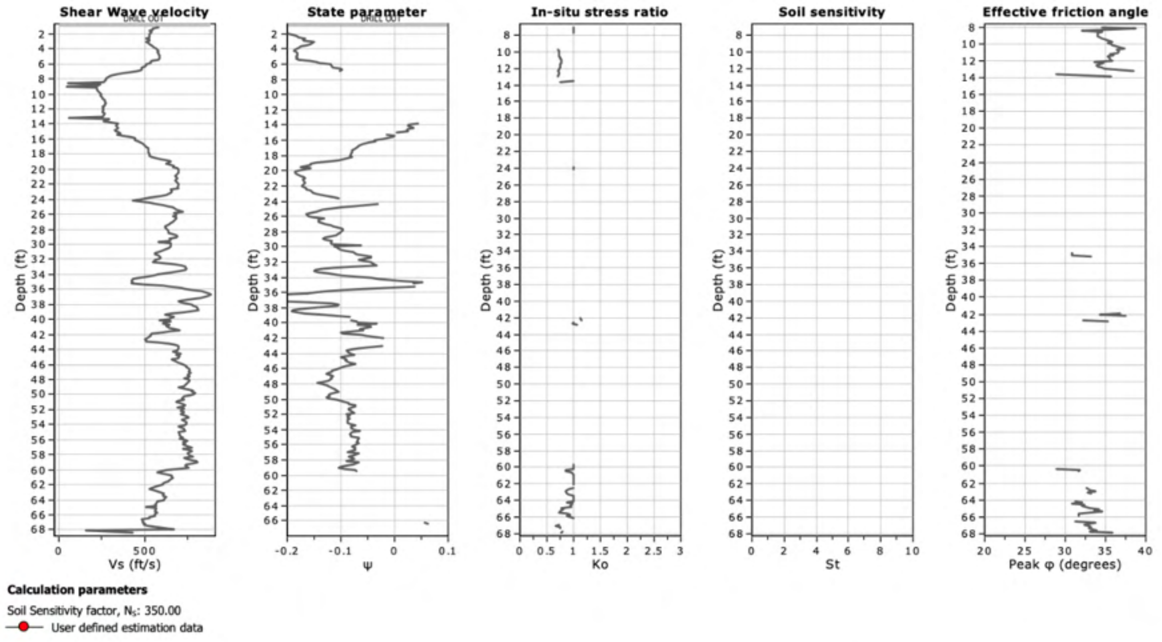
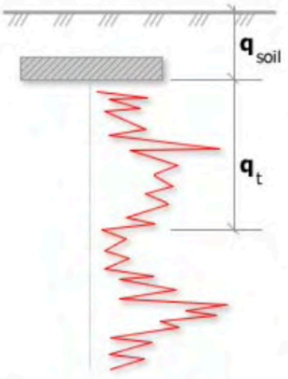


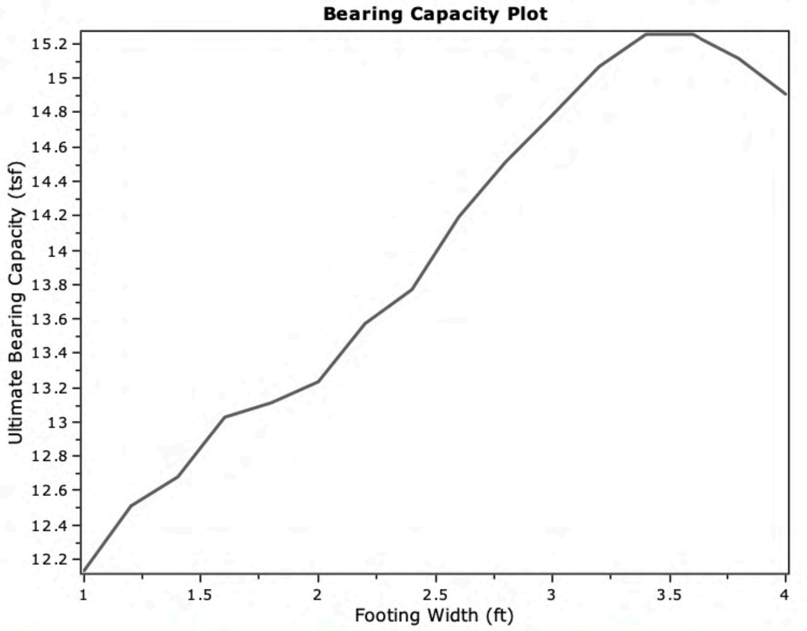
Figure 29: Shear Wave velocity, State parameter, in-situ stress ratio, Soil sensitivity and Effective friction angle test.



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 (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	1.00	0.50	2.00	60.52	0.20	0.03	12.13
2	1.20	0.50	2.30	62.42	0.20	0.03	12.51
3	1.40	0.50	2.60	63.23	0.20	0.03	12.68
4	1.60	0.50	2.90	65.00	0.20	0.03	13.03
5	1.80	0.50	3.20	65.39	0.20	0.03	13.11
6	2.00	0.50	3.50	66.01	0.20	0.03	13.23
7	2.20	0.50	3.80	67.72	0.20	0.03	13.57
8	2.40	0.50	4.10	68.71	0.20	0.03	13.77
9	2.60	0.50	4.40	70.81	0.20	0.03	14.19
10	2.80	0.50	4.70	72.41	0.20	0.03	14.51
11	3.00	0.50	5.00	73.77	0.20	0.03	14.78
12	3.20	0.50	5.30	75.18	0.20	0.03	15.07
13	3.40	0.50	5.60	76.12	0.20	0.03	15.25
14	3.60	0.50	5.90	76.12	0.20	0.03	15.25
15	3.80	0.50	6.20	75.41	0.20	0.03	15.11
16	4.00	0.50	6.50	74.39	0.20	0.03	14.91

Figure 30: Load capacite of Soils

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125

SUMMARY OF CONE PENETRATION TESTING

The results of the CPT soundings generally correlated with the subsurface conditions encountered during our soil boring subsurface investigation. Beneath the existing fill material, silty sand with varying amounts of clay was encountered to depths ranging between approximately 9 feet and 87 feet below ground surface; underlain by silt and clay deposits to depths ranging between approximately 12 feet below ground surface and 45 feet below the ground surface. Beneath the silt and clay deposit, silty sand and clayey sand was encountered to refusal depths ranging between approximately 30 feet below the ground surface and 73 feet below the ground surface. The refusal of the CPT sounding is likely due to very dense conditions within the sandy deposits. The soil temperatures typically ranged between approximately 11.8 degrees Celsius (°C) and 29.2 °C. During CPT sounding preformed at location CPT-A (and others two CPT places) may not accurately depict subsurface temperatures due to hold time that may not have allowed for dissipation of temperature increase due to frictional resistance.

GROUNDWATER

Groundwater was encountered within the soil borings at depths ranging between approximately two feet and nine feet below the ground surface; corresponding to elevations ranging between 14.3 feet above mse and 0.1 feet below mse. Ground water was not encountered within soil borings at three locations, generally along the western and northern portions of the alignment. Groundwater is expected to fluctuate from these observed levels and this only represents the levels of groundwater encountered during the investigation.

CONCLUSIONS AND RECOMENDATIONS

The site investigations campaign is important to support the initial design and planning phase for all projects. Based in this data it's important consider demolition and surface cover stripping prior to the start of constructions ad all utilities should be identified and secured. If encountered, existing structural elements and/or debris should be removed from the area of proposed construction. The recommendations presented herein should be utilized by a qualified engineer in preparing the project plans and specifications. The engineer should consider these recommendations as minimum physical standards that may be superseded by local and regional building codes and structural considerations. These recommendations are relevant to the design phase and should not be substituted for construction specifications. The exploration and analysis of the foundation conditions reported herein are presented to form a reasonable basis for foundation design. The recommendations submitted for the proposed construction are based on the available soil information, loading information, and the preliminary design details furnished or assumed. Deviations from the noted subsurface conditions encountered during construction should be brought to the attention of the geotechnical engineer.

The possibility exists that conditions between borings may differ from those at specific boring locations, and conditions may not be as anticipated by the designers or contractors. In addition, the construction process may itself alter soil conditions. Therefore, a qualified geotechnical engineer or their representatives should observe and document the construction procedures used and the conditions encountered, as well as conduct testing and inspection, to ensure the design criteria are met or recommendations to address deviations are implemented.

Soils placed as structural fill material should consist of well graded sand or gravel with a maximum particle size of three inches in diameter and less than 15 percent of material passing the number 200 sieve. These materials should be free of objectionable debris (clay clumps, organic and/or deleterious material, etc.) and within moisture contents suitable for compaction. Alternative soil types with higher percentages of silt and clay may be considered, provided that the contractor is able to achieve proper compaction and maintain suitable subgrade once the material is placed. Any soil with higher percentages of silt and clay are extremely moisture sensitive and will only be suitable for reuse as structural fill material under ideal weather conditions. Materials wetted beyond the optimum moisture content; that contain oversized rock or debris; or with increased amounts of objectionable debris will not be suitable for reuse as structural fill material. The contractor should be responsible for importing structural fill material and/or processing on-site soils as required so that these materials are suitable for structural fill placement.

Based on this site investigations data the on-site soils expected to be encountered during construction include existing fill material and upper granular natural coastal plain deposits. Portions of the existing fill material (above the groundwater level) are preliminarily expected to be suitable for reuse as structural fill material, but special handling to remove objectionable debris and/or oversized particles should be anticipated. Granular portions of the natural coastal plain deposits (above the groundwater level) are preliminarily expected to be suitable for reuse as structural fill material, provided moisture contents are within tolerable limits for compaction. The organic deposits encountered during construction are not expected to be suitable for reuse as structural fill material. Reuse of these materials will be contingent upon further evaluation during construction.

About compaction and placement requirements, structural fill and backfill should be placed in maximum 12-inch loose lifts and compacted to 95 percent of the maximum dry density within a targeted two percent of the optimum moisture content as determined by ASTM D-1557 (Modified Proctor). Variations in moisture content may be acceptable, subject to qualified geotechnical engineer's approval, if the contractor is able to achieve the necessary compaction. It recommends using a minimum 20-ton smooth drum roller to compact subgrade soils within larger areas of fill placement (if required). Alternative compaction equipment, such as sheepsfoot rollers, excavator-mounted vibratory plate, and/or hand operated vibratory jumping jacks will likely be required within confined utility excavations. Fill material compacted with relatively light weight equipment may need to be placed in thinner loose lifts and an increased number of passes may be required to achieve proper compaction.

Before filling operations begin, representative samples of each proposed fill material (on-site and imported) should be collected. The samples should be tested to determine the maximum dry density (ASTM D-1557), optimum moisture content (ASTM D-1557), natural moisture content (ASTM D-2216), gradation (ASTM D-6913), and plasticity of the soil (ASTM D-4318). These tests are needed for quality control during compaction and also to determine if the fill material is acceptable. The placement of structural fill and backfill should be monitored by qualified geotechnical engineer or technician to ensure that the specified material and lift thicknesses are properly installed. A sufficient number of in-place density tests should be performed during fill placement to ensure that the specified compaction is achieved throughout the height of the fill or backfill.

The initial backfill at excavations that extend below the groundwater level (in conjunction with dewatering methods) may consist of nominally three-quarter- to one-inch crushed, washed aggregate placed to a minimum of one-foot above water levels before subsequent lifts of structural fill. Submerged fill should be separated from surrounding soils (below, adjacent, and above) with a woven geotextile, such as Mirafi 500X or equivalent to prevent migration of fines from surrounding soils into the aggregate.

The groundwater will be encountered at depths shallower than planned for excavations, for example, for proposed installation of foundations. A design of a groundwater control system is requerid and usually is responsibility of the contractor, but typically excavations extending less than two feet below the groundwater levels may be controlled by providing a sufficient number of sump pumps to draw down groundwater one foot below the bottom of the excavation. Deeper excavations or excavations that remain open for relatively long periods may require more extensive dewatering systems, such the agencies permitted dewatering wells or a well point system. Temporary watertight sheeting and shoring may be considered as an attempt to help control groundwater within excavations. Every effort must be made to maintain drainage of surface water runoff away from construction areas by grading and limiting the exposure of excavations to rainfall in order to mitigate exacerbation of the groundwater conditions during construction.

The coarse-grained materials are typically cohesionless will not support an open borehole. As such, drilling methods including injecting drilling fluid, pipe jacking, or reaming tools should be considered within the coarse- grained, cohesionless soil. The drilling method selected should be capable to drill through potential debris associated with the existing fill material/natural organic marine deposits and/or very dense/very stiff layers within the natural coastal plain deposits.

Deep trenching techniques may be considered for the proposed project. In general, deep trenching is performed by lowering a specific trenching cutting head "trencher" into the ground to a desired depth. The trencher makes a precise cut into the ground and the trench is backfilled immediately with desired fill material. As the trench is being advanced, the desired utility cable is fed into the trencher and laid at the bottom of the trench. Applications for deep trenching typically include the installation of utilities such as electricity, communications, oil, gas, water, and sewerage. Based on the subsurface conditions encountered, deep trenching is preliminarily expected to be feasible for this project. Special considerations for possible running sands during excavation should be evaluated. A specialty contractor should be consulted to provide the required installation of the cable by the use of deep trenching, if so desired.

The soils are most consistent with a Site Class E defined by the International Building Code. Potential liquefaction considerations should be evaluated by the project design team.

The existing fill material and upper granular soils encountered during the investigation are consistent with Type C Soil Conditions as defined by 29 CFR Part 1926 (OSHA), which requires a maximum unbraced excavation angle of 1.5:1 (horizontal: vertical). Actual conditions encountered during construction should be evaluated by a competent person (as define by OSHA) to ensure that safe temporary excavation methods and/or shoring and bracing requirements are implemented.

REFERENCES

ASTM D516-16. (2016) Standard Test Method for Sulfate Ion in Water, *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D0516-16.

ASTM D1140-17. (2017). Standard Test Methods for Determining the Amount of Material Finer than 75-μm (No. 200) Sieve in Soils by Washing. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D1140-17.

ASTM D1557-12. (2021). Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)). *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D1557-12R21.

ASTM D1586 / D1586M-18. (2018). Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D1586_D1586M-18.

ASTM D1587 / D1587M-15. (2015). Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes, *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D1587_D1587M-15.

ASTM D2216-19. (2019). Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass, *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D2216-19.

ASTM D2487-17e1. (2017). Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D2487-17E01.

ASTM D2488-17e1. (2017). Standard Practice for Description and Identification of Soils (Visual-Manual Procedures). *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D2488-17E01 .

ASTM D2850-15. (2015). Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D2850-15 .

ASTM D4318-17e1. (2017). Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D4318-17E01.

ASTM D4373-21. (2021). Standard Test Method for Rapid Determination of Carbonate Content of Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D4373-21.

ASTM D4972-19. (2019). Standard Test Methods for pH of Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D4972-19 .

ASTM D5334-14. (2014). Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D5334-14.

ASTM D5778-20. (2020). Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D5778-20.

ASTM D5783-17. (2017). Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D7928-17.

ASTM D6913/D6913M-17. (2017). Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/D6913_D6913M-17.

ASTM G187-18. (2018). Standard Test Method for Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method. *ASTM International, West Conshohocken, PA*. https://doi.org/10.1520/G0187-18 .

Jefferies, M.G., and Davies, M.P., (1993). Use of CPTU to estimate equivalent SPT N60. *Geotechnical Testing Journal,* ASTM, 16(4): 458-468. https://doi.org/10.1520/GTJ10286J .

Robertson, P.K. (1990). Soil classification using the cone penetration test. *Canadian Geotechnical Journal,* 27(1): 151–158. http://doi.org/10.1139/t90-014 .

Robertson, P.K. (2016) Cone Penetration Test (CPT)-Based Soil Behaviour Type (SBT) Classification System—An Update. *Canadian Geotechnical Journal,* 53, 1910-1927. https://doi.org/10.1139/cgj-2016-0044 .

Robertson, P.K., and Campanella, R.G., (1983a). Interpretation of cone penetration tests Part I (sand). *Canadian Geotechnical Journal*, 20(4): 718-733. https://doi.org/10.1139/t83-078 .

SCHNAID, F.; ODEBRECHT, E.; ROCHA, M. M.; BERNARDES, G. P. Prediction of soil properties from the concepts of energy transfer in dynamic penetration tests. J. Geotech. Geoenv. Eng., ASCE, v. 135, n. 8, p. 1092-1100, 2009.

Schneider, J.A., Randolph, M.F., Mayne, P.W., and Ramsey, N.R. (2008). Analysis of factors influencing soil classification using normalized piezocone tip resistance and pore pressure parameters. *Journal of Geotechnical and Geo environmental Engineering, ASCE,* 134(11): 1569–1586. https://doi.org/10.1061/(ASCE)1090-0241(2008)134:11(1569).

APPENDIX

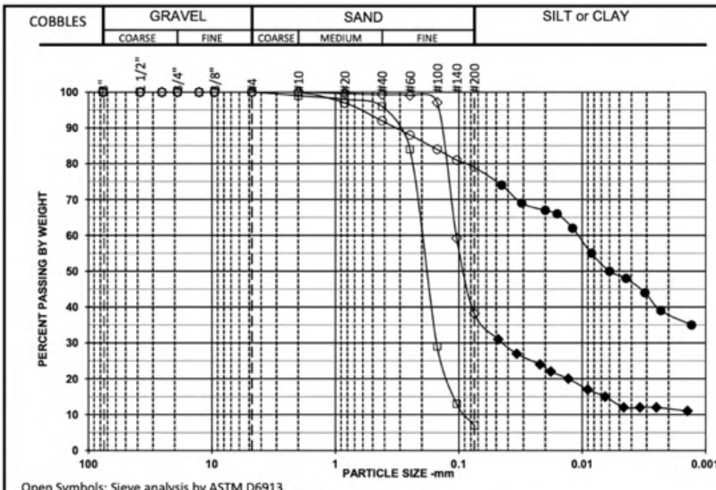
APPENDIX A – BOREHOLES (BH)

Borehole ID	Depth(ft)	Water Lever	SAMPLE		Rec (in)	RQD %	Blows per 6" or drill time (mm:ss)		N	N60	Remarks	Description of Materials (Classification)
			Number	Type								
BH-A	1		S-1	SS	6	–	–	4	11	14		1.5" asphaltic concrete, no apparent subbase material
	2						7	10				Brown to gray coarse to fine sand, trace fine gravel, trace silt, moist, medium dense (SP)
	3		S-2	SS	18	–	7	10	24	30		
	4						14	17				
	5		S-3	SS	18	–	10	18	39	49		Gray coarse to fine sand, trace silt, wet, dense (SP-SM)
	6						21	20				
	7		S-4	SS	16	–	7	10	22	28		
	8						12	14				
	9		S-5	SS	18	–	7	5	15	19		
	10						10	11				
	11		S-6	SS	18	–	6	7	16	20		
	12						9	7				
	13											
	14											
	15											
	16		S-7	SS	14	–	5	7	18	26		
	17						11	7				
	18											
	19										Organic material in cuttings @ 17ft	Gray organic silt, little coarse to fine sand, wet, very soft (OL)
	20											
	21		S-8	SS	29	–	WOH	WOH	WOH	WOH	Qp=0.25 tsf	
	22						WOH	WOH				
	23		ST-1	ST	12	–	–	–				Gray medium to fine sand, some clay, little peat, wet (SC)
	23.05		ST-1B									
	24		ST-1				–	–				
	25		S-9	SS	18	–	WOH	WOH	WOH	WOH	Qp=0.25 tsf	Gray organic silt, trace coarse to fine sand, wet, very soft (OL)
	26						WOH	WOH				
	27											
	28											
	29											
	30											
	31		S-10	SS	20	–	WOH	WOH	WOH	WOH	Qp=0.25 tsf	
	32						WOH	WOH				
	33											
	34											
	35											
	36		S-11	SS	20	–	WOH	WOH	WOH	WOH	Qp=0.25 tsf	
	37						WOH	WOH				
	38											
	39											
	40											
	41		S-12	SS	20	–	WOH	WOH	2	3	Qp=0.25 tsf	Gray organic clay, some coarse to fine sand, wet, very soft (OH)
	42						2	2				
	43											
	44											
	45											
	46		S-13	SS	16	–	6	6	22	37		Gray coarse to fine sand, and silt, wet, medium dense (SM)
	47						16	13				
	48											
	49											
	50											
	51		S-14	SS	14	–	4	3	7	12		Gray medium to fine sand, some silt, wet, loose (SM)
	52						4	9				
	53											
	54											
	55											
	56		S-15	SS	16	–	5	7	13	22		Gray medium to fine sand, little silt, wet, medium dense (SM)
	57						6	10				
	58											
	59											
	60											
	61		S-16	SS	16	–	5	8	19	32		As above, trace shell fragments (SM)
	62						11	12				
	63											
	64											
	65											
	66		S-17	SS	14	–	2	3	6	10		Gray medium to fine sand, some silt, wet, loose (SM)
	67						3	3				
	68											
	69											
	70											
	71		S-18	SS	16	–	WOH	WOH	3	5		Gray medium to fine sand, some silt, wet, very loose (SM)
	72						3	8				
	73											
	74											
	75											
	76		S-19	SS	14	–	19	40	82	137		Gray coarse to fine sand, trace fine gravel, trace silt, wet, very dense (SP-SM)
	77						42	65				
	78											
	79											
	80											
	81		S-20	SS	16	–	101	71	144	240		
	82						73	69				
	83											
	84											
	85											
	86		S-21	SS	12	–	50	130	180/7	300		Gray coarse to fine sand, little coarse to fine gravel, trace silt, wet, very dense (SP-SM)
	86,1						50/1	–				
												Boring BH-413A was terminated at approximately 86.1 feet below the ground surface.

Borehole ID	Depth(ft)	Water Lever	SAMPLE Number	SAMPLE Type	Rec (in)	RQD %	Blows per 6" or drill time (mm:ss)		N	N60	Remarks	Description of Materials (Classification)
BH - B	0,5		S-1	SS	6	–	6	7	15			6" Asphaltic Concrete
	2						8	–				Dark gray medium to fine sand, little coarse to fine gravel, little silt, moist (FILL)
	3		S-2	SS	12	–	4	3	6			
	4						3	2				Tan medium to fine sand, trace silt, wet, loose (SP)
	5		S-3	SS	18	–	2	2	5			
	6						3	3				
	7		S-4	SS	20	–	4	3	7			
	8						4	3				Dark gray medium to fine sand, trace silt, wet, loose (SP)
	9		S-5 A/B	SS	18	–	2	1	2			
	10						1	1			Qp<0.25 tsf	
	11		S-6	SS	8	–	WOH	WOH	WOH			Dark gray organic clay, little medium to fine sand, wet, very soft (OH)
	12						WOH	WOH			Qp<0.25 tsf	
	13		ST-1	ST	12	–	–	–				Gray silty clay, some medium to fine sand, little organic fibers, wet (CH)
	14						–	–				
	15		S-7	SS	18	–	2	WOH	1			Dark gray fine sand, some clay, wet, very loose (SC)
	16						1	WOH				
	17		S-8	SS	12	–	WOR	3	6			
	18						3	2				
	19											
	20											
	21		S-9	SS	8	–	2	4	8			Dark gray medium to fine sand, trace silt, trace shells, wet, loose (SPSM)
	22						4	5				
	23											
	24											
	25											
	26		S-10	SS	14	–	7	11	26			
	27						15	16				
	28											
	29											
	30											Dark gray fine sand, trace silt, wet, medium dense (SP-SM)
	31		S-11	SS	12	–	6	8	18			
	32						10	12				
	33											
	34											
	35										Qp<0.25 tsf	
	36		S-12 A/B	SS	14	–	WOR	WOR	1		Qp=0.5 tsf	Dark gray silty clay, some medium to fine sand, trace shells, wet, very soft (CH)
	37						1	2				
	38											Dark gray medium to fine sand, and silty clay, wet, very loose (SC)
	39											
	40											
	41		S-13	SS	12	–	15	15	30			
	42						15	14				
	43											Gray medium to fine sand, trace silt, wet, dense (SP)
	44											
	45											
	46		S-14	SS	16	–	8	11	25			
	47						14	13				
	48											Dark gray medium to fine sand, trace silt, wet, medium dense (SPSM)
	49											
	50											
	51		S-15	SS	18	–	6	9	20			
	52						11	13				
	53											Dark gray medium to fine sand, little silt, trace shells, wet, medium dense (SM)
	54											
	55											
	56		S-16	SS	18	–	5	7	17			
	57						10	12				
	58											Dark gray fine sand, some silt, wet, medium dense (SM)
	59											
	60											
	61		S-17	SS	12	–	3	4	12			
	62						8	10				
	63											Dark gray fine sand, and silty clay, wet, medium dense (SC)
	64											
	65											
	66		S-18	SS	2	–	WOH	WOH	2		Qp=0.5 tsf	
	67						2	8				Dark gray silty clay, little medium to fine sand, wet, soft (CL)
	68		ST-2	ST	0	–	–	–				Shelby tube ST-2, No Recovery
	69						–	–				
	70											
	71		S-19	SS	6	–	19	32	82/10		Hard tube resistance 68.2 to 69 feet, possible stratum change	
	72						50/4	–				
	73											
	74											Gray coarse to fine sand, trace fine gravel, trace silt, wet, very dense (SW-SM)
	75		S-20	SS	3	–	50/4	–	50/4			
	76											
	77											
	78											
	79											
	80											
	81		S-21	SS	8	–	14	22	49			
	82						27	31				
	83											Brown medium to fine sand, trace silt, wet, dense (SP-SM)
	84											
	85											
	86		S-22	SS	8	–	22	33	73			Tan medium to fine sand, trace silt, wet, very dense (SP)
	87						40	48				

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125

APPENDIX B – SIEVE ANALYSIS BH-A

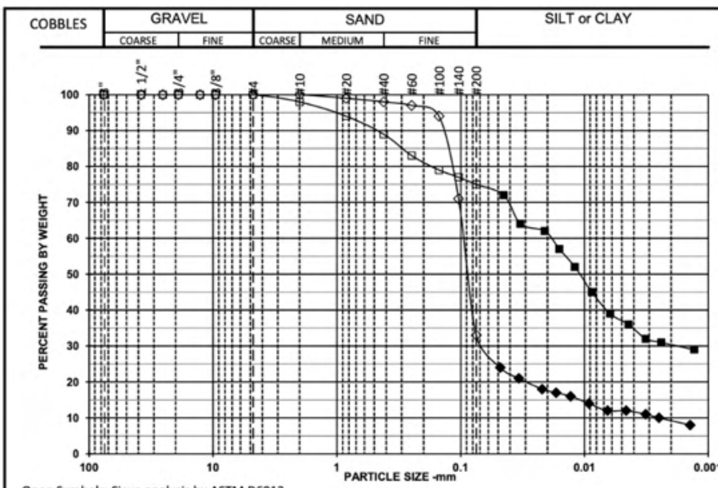


Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	13.7				SP-SM		Grayish brown, Poorly graded sand with silt	08/21/20
◇	34.7	33	23	10	SC		Gray, Clayey sand with Peat	07/24/20
○	99.8	141	61	80	OH		Gray, Organic silt with sand, Insufficient sample size	08/21/20

Symbol	□	◇	○
Boring			
Sample	S-3	ST-1B	S-10
Depth	4-6	23.05	30-32
% +3"	0	0	0
% Gravel	0	0	0
% SAND	93	61.8	21
%C SAND	1	0	0
%M SAND	3	0.7	8
%F SAND	89	61.1	13
% FINES	7	38.2	79
D_{100} (mm)	12.7	4.75	4.75
D_{60} (mm)	0.199	0.105	0.011
D_{30} (mm)	0.15	0.044	
D_{10} (mm)	0.088		
Cc	1.3		
Cu	2.3		
Sieve Size/ID #	**Percent Finer Data**		
6"	100	100	100
4"	100	100	100
3"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	100	100
3/8"	100	100	100
#4	100	100	100
#10	99	100	100
#20	98	99.6	97
#40	96	99.3	92
#60	84	99.1	88
#100	29	97.1	84
#140	13	59.2	81
#200	7	38.2	79
5μ m		13	49
2μ m		12	38
1μ m		10	33

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928



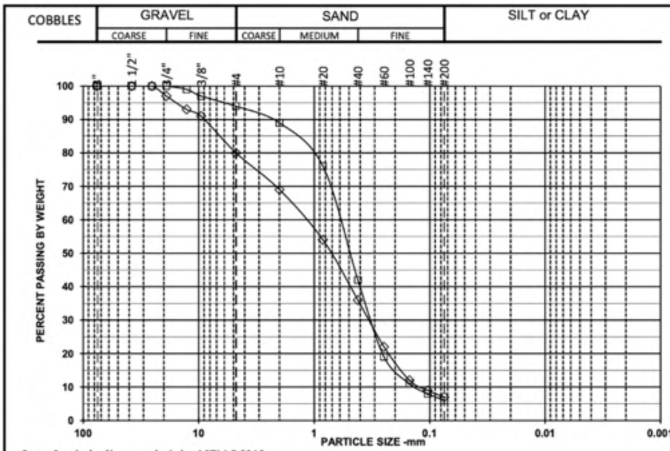
Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	73.8	68	27	41	OH		Dark gray, Organic clay with sand, Insufficient sample size	08/21/20
◇	29.6				SM		Gray, Silty sand	08/21/20
○								

Symbol	□	◇	○
Boring			
Sample	S-12	S-17	
Depth	40-42	65-67	
% +3"	0	0	
% Gravel	0	0	
% SAND	25	67	
%C SAND	2	0	
%M SAND	9	2	
%F SAND	14	65	
% FINES	75	33	
D_{100} (mm)	9.53	4.75	
D_{60} (mm)	0.019	0.095	
D_{30} (mm)	0.002	0.065	
D_{10} (mm)		0.003	
Cc		17.8	
Cu		37.9	
Sieve Size/ID #	**Percent Finer Data**		
6"	100	100	
4"	100	100	
3"	100	100	
1 1/2"	100	100	
1"	100	100	
3/4"	100	100	
1/2"	100	100	
3/8"	100	100	
#4	100	100	
#10	98	100	
#20	94	99	
#40	89	98	
#60	83	97	
#100	79	94	
#140	77	71	
#200	75	33	
5μ m	37	12	
2μ m	30	9	
1μ m	28	7	

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125



Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

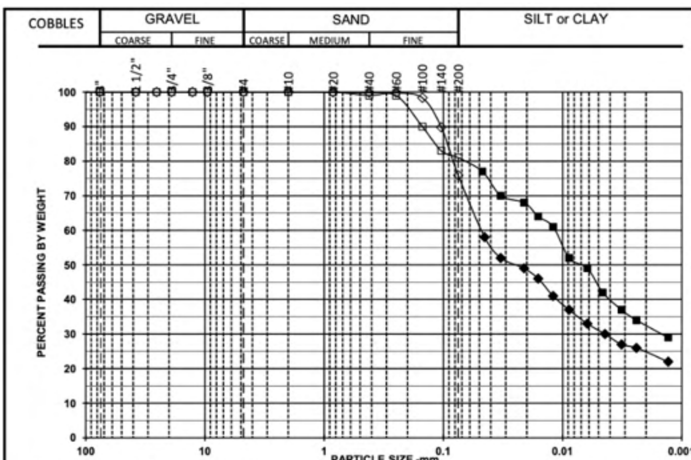
SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	13.0				SP-SM		Grayish brown, Poorly graded sand with silt	08/21/20
◇	8.2				SP-SM		Brown, Poorly graded sand with silt and gravel	08/21/20
○								

Symbol	□	◇	○
Boring			
Sample	S-19	S-21	
Depth	75-77	83-85	
% +3"	0	0	
% Gravel	6	20	
% SAND	88	73	
%C SAND	5	11	
%M SAND	47	33	
%F SAND	36	29	
% FINES	6	7	
D_{100} (mm)	19.1	25.4	
D_{60} (mm)	0.606	1.190	
D_{30} (mm)	0.32	0.34	
D_{10} (mm)	0.13	0.12	
Cc	1.3	0.8	
Cu	4.7	9.9	

Sieve Size/ID #	Percent Finer Data		
6"	100	100	
4"	100	100	
3"	100	100	
1 1/2"	100	100	
1"	100	100	
3/4"	100	97	
1/2"	99	93	
3/8"	97	91	
#4	94	80	
#10	89	69	
#20	76	54	
#40	42	36	
#60	19	22	
#100	11	12	
#140	8	9	
#200	6	7	
5µ m			
2µ m			
1µ m			

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928

APPENDIX C – SIEVE ANALYSIS BH-B



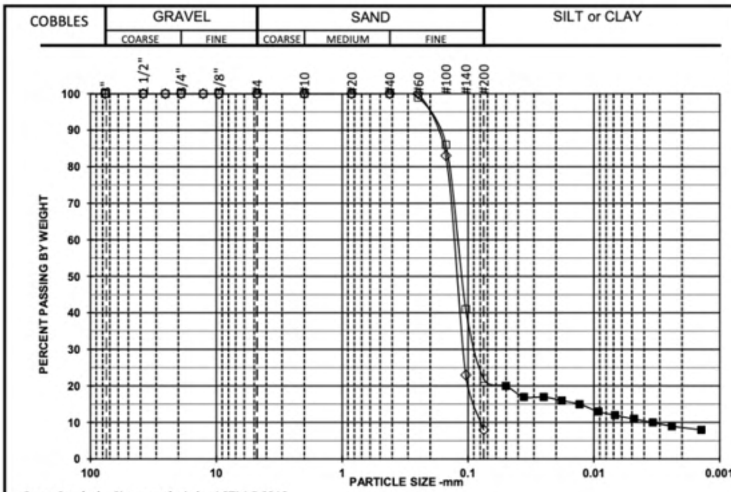
Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	83.4	116	41	75	OH		Gray, Organic clay with sand, Insufficient sample size	10/06/20
◇	48.6	56	23	33	CH		Gray, Fat clay with sand organic mat'l noted	10/06/20
○								

Symbol	□	◇	○
Sample	S-5B	ST-1C	
Depth	9-10	13.6	
% +3"	0	0	
% Gravel	0	0	
% SAND	19	24.1	
%C SAND	0	0	
%M SAND	1	0.2	
%F SAND	18	23.9	
% FINES	81	75.9	
D_{100} (mm)	4.75	2	
D_{60} (mm)	0.012	0.048	
D_{30} (mm)	0.002	0.004	
D_{10} (mm)			
Cc			
Cu			

Sieve Size/ID #	Percent Finer Data		
6"	100	100	
4"	100	100	
3"	100	100	
1 1/2"	100	100	
1"	100	100	
3/4"	100	100	
1/2"	100	100	
3/8"	100	100	
#4	100	100	
#10	100	100	
#20	100	99.9	
#40	99	99.8	
#60	99	99.8	
#100	90	98.3	
#140	83	89.8	
#200	81	75.9	
5µ m	44	31	
2µ m	33	25	
1µ m	27	20	

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928



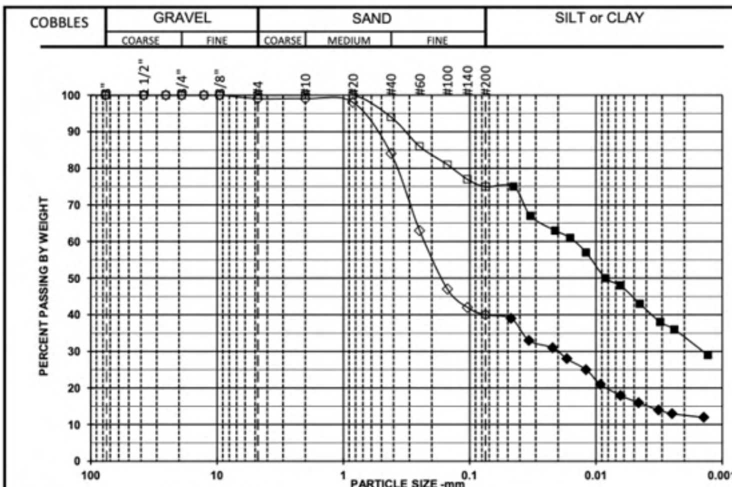
Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	34.2				SC		Gray, Clayey sand organics present	10/07/20
◇	25.3				SP-SM		Gray, Poorly graded sand with silt	10/07/20
○								

Symbol	□	◇	○
Sample	S-7	S-10	
Depth	14-16	25-27	
% +3"	0	0	
% Gravel	0	0	
% SAND	78	92	
%C SAND	0	0	
%M SAND	0	0	
%F SAND	78	92	
% FINES	22	8	
D_{100} (mm)	2	2	
D_{60} (mm)	0.121	0.13	
D_{30} (mm)	0.086	0.11	
D_{10} (mm)	0.003	0.078	
Cc	18	1.2	
Cu	35.6	1.7	

Sieve Size/ID #	Percent Finer Data	
6"	100	100
4"	100	100
3"	100	100
1 1/2"	100	100
1"	100	100
3/4"	100	100
1/2"	100	100
3/8"	100	100
#4	100	100
#10	100	100
#20	100	100
#40	100	100
#60	99	100
#100	86	83
#140	41	23
#200	22	8
5µm	11	
2µm	9	
1µm	7	

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928



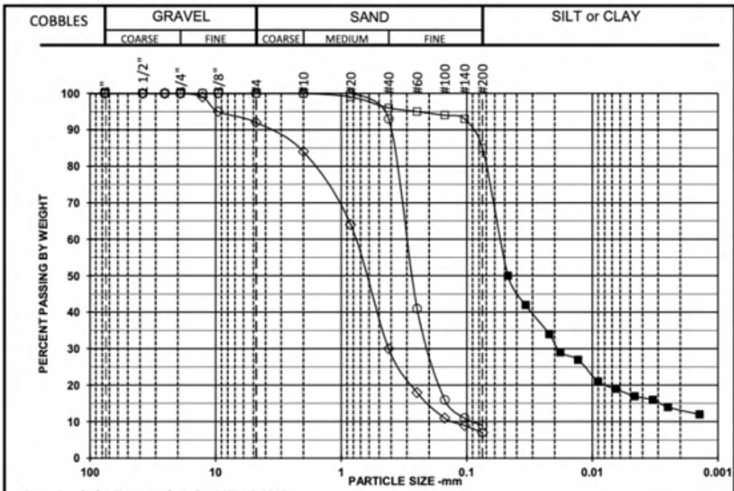
Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	49.3	69	25	44	CH		Gray, Fat clay with sand shell fragments noted	10/06/20
◇	20.2				SC		Gray, Clayey sand	10/06/20
○								

Symbol	□	◇	○
Sample	S-12A	S-12B	
Depth	35-36	36-37	
% +3"	0	0	
% Gravel	0	1	
% SAND	25	59	
%C SAND	0	0	
%M SAND	6	15	
%F SAND	19	44	
% FINES	75	40	
D_{100} (mm)	4.75	9.53	
D_{60} (mm)	0.015	0.226	
D_{30} (mm)	0.001	0.02	
D_{10} (mm)			
Cc			
Cu			

Sieve Size/ID #	Percent Finer Data	
6"	100	100
4"	100	100
3"	100	100
1 1/2"	100	100
1"	100	100
3/4"	100	100
1/2"	100	100
3/8"	100	100
#4	100	99
#10	100	99
#20	100	98
#40	94	84
#60	86	63
#100	81	47
#140	77	42
#200	75	40
5µm	44	17
2µm	34	13
1µm	26	11

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928



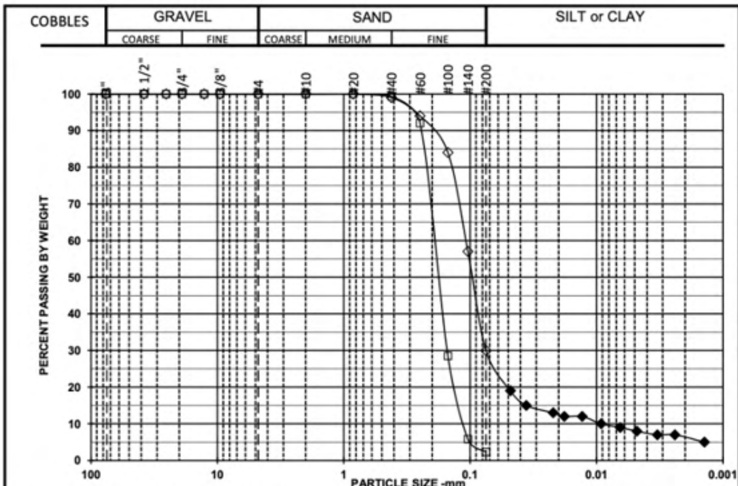
Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□	29.6	31	19	12	CL		Gray, Lean clay with sand	10/06/20
◇	10.3				SW-SM		Gray, Well-graded sand with silt	10/06/20
○	19.3				SP-SM		Brown, Poorly graded sand with silt	10/06/20

Symbol	□	◇	○
			3B
Sample	S-18	S-19	S-21
Depth	65-67	70-71.3	80-82
% +3"	0	0	0
% Gravel	0	8	0
% SAND	15	85	91
%C SAND	0	8	0
%M SAND	4	54	7
%F SAND	11	23	84
% FINES	85	7	9
D_{100} (mm)	4.75	19.1	4.75
D_{60} (mm)	0.054	0.775	0.301
D_{30} (mm)	0.019	0.42	0.2
D_{10} (mm)		0.12	0.088
Cc		1.9	1.5
Cu		6.5	3.4

Sieve Size/ID #	Percent Finer Data		
6"	100	100	100
4"	100	100	100
3"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	99	100
3/8"	100	95	100
#4	100	92	100
#10	100	84	100
#20	99	64	100
#40	96	30	93
#60	95	18	41
#100	94	11	16
#140	93	9	11
#200	85	7	9
5μ m	17		
2μ m	13		
1μ m	11		

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928



Open Symbols: Sieve analysis by ASTM D6913
Filled symbols: Hydrometer analysis by ASTM D7928 corrected for complete sample

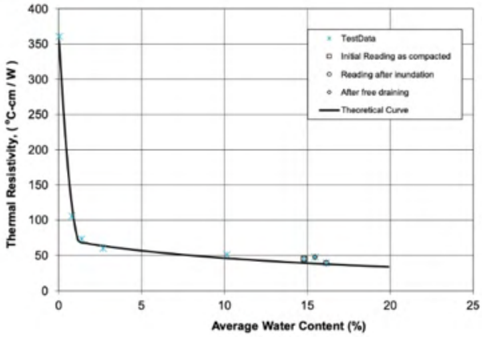
SYMBOL	w (%)	LL	PL	PI	USCS	AASHTO	USCS DESCRIPTION AND REMARKS	DATE
□					SP		Dark gray, Poorly graded sand	10/13/20
◇	30.2				SM		Gray, Silty sand shell fragments noted	10/06/20
○								

Symbol	□	◇	○
Sample	S-2 to S-5A	S-14 to S-17	
Depth	2-9	45-62	
% +3"	0	0	
% Gravel	0	0	
% SAND	97.7	70	
%C SAND	0	0	
%M SAND	0.7	1	
%F SAND	97	69	
% FINES	2.3	30	
D_{100} (mm)	2	4.75	
D_{60} (mm)	0.193	0.108	
D_{30} (mm)	0.15	0.075	
D_{10} (mm)	0.11	0.009	
Cc	1.1	5.7	
Cu	1.8	11.7	

Sieve Size/ID #	Percent Finer Data		
6"	100	100	
4"	100	100	
3"	100	100	
1 1/2"	100	100	
1"	100	100	
3/4"	100	100	
1/2"	100	100	
3/8"	100	100	
#4	100	100	
#10	100	100	
#20	100	100	
#40	99.3	99	
#60	92	94	
#100	28.5	84	
#140	5.8	57	
#200	2.3	30	
5μ m		8	
2μ m		6	
1μ m		4	

PARTICLE SIZE DISTRIBUTION
ASTM D6913 & ASTM D7928

APPENDIX D – THERMAL CONDUCTIVITY TESTING FOR BH-A



Test Summary

Sample Description: SP-SM, light gray fine sand with silt
Sample Type: compacted medium dense

	Water Content (%)	Dry Density (pcf)	Total Density (pcf)
As-Compacted:	14.8	108.3	124.4

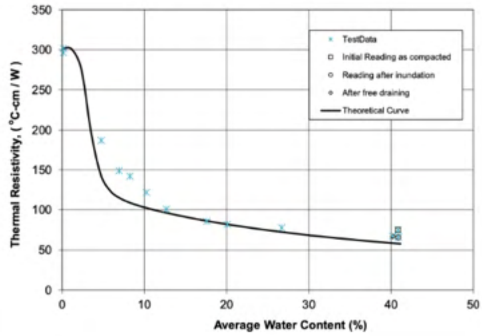
Note: Test densities may vary from initial densities due to water content variation.

Thermal Resistivity: 45.3 °C-cm/W Initial
Thermal Resistivity: 39.4 °C-cm/W after inundation
Thermal Resistivity: 47.6 °C-cm/W after free draining
Thermal Resistivity: 56.9 °C-cm/W @ 5% water content
Thermal Resistivity: 62.5 °C-cm/W @ 3% water content
Thermal Resistivity: 360.6 °C-cm/W ~dry

Reviewed By: GET

SUMMARY OF THERMAL CONDUCTIVITY TESTING ASTM D5334

Sample: S-(2-7)
Depth: 2-17 ft.



Test Summary

Sample Description: OH, dark gray organic silt
Sample Type: Intact tube sample

	Water Content (%)	Dry Density (pcf)	Total Density (pcf)
As-Compacted:	40.8	79.1	111.5

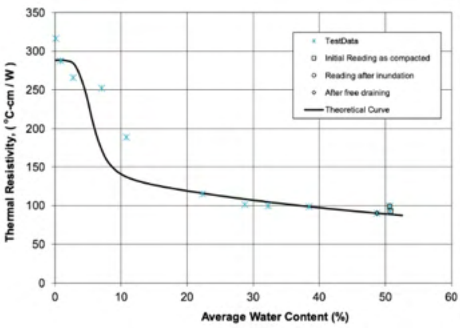
Note: Test densities may vary from initial densities due to water content variation.

Thermal Resistivity: 75.1 °C-cm/W Initial
Thermal Resistivity: 65.8 °C-cm/W after inundation
Thermal Resistivity: 67.1 °C-cm/W after free draining
Thermal Resistivity: 136.6 °C-cm/W @ 5% water content
Thermal Resistivity: 233.7 °C-cm/W @ 3% water content
Thermal Resistivity: 301.6 °C-cm/W ~dry

Reviewed By: GET

SUMMARY OF THERMAL CONDUCTIVITY TESTING ASTM D5334

Sample: ST-1
Depth: 22.4 ft.



Test Summary

Sample Description: OH, dark brown organic silt, trace fine gravel, trace fine to medium sand
Sample Type: compacted very soft

	Water Content (%)	Dry Density (pcf)	Total Density (pcf)
As-Compacted:	50.8	68.6	103.4

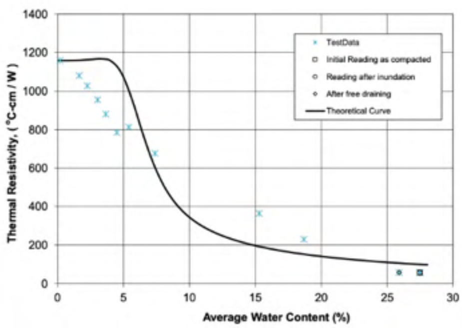
Note: Test densities may vary from initial densities due to water content variation.

Thermal Resistivity: 92.5 °C-cm/W Initial
Thermal Resistivity: 99.3 °C-cm/W after inundation
Thermal Resistivity: 90.2 °C-cm/W after free draining
Thermal Resistivity: 233.3 °C-cm/W @ 5% water content
Thermal Resistivity: 281.9 °C-cm/W @ 3% water content
Thermal Resistivity: 288.1 °C-cm/W ~dry

Reviewed By: GET

SUMMARY OF THERMAL CONDUCTIVITY TESTING ASTM D5334

Sample: S-(8-12)
Depth: 20-42 ft.



Test Summary

Sample Description: SM, dark gray silty fine sand, trace fine gravel
Sample Type: compacted medium dense

	Water Content (%)	Dry Density (pcf)	Total Density (pcf)
As-Compacted:	27.5	91.7	116.9

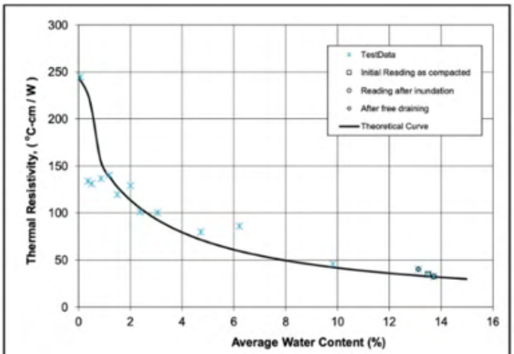
Note: Test densities may vary from initial densities due to water content variation.

Thermal Resistivity: 58.1 °C-cm/W Initial
Thermal Resistivity: 56.4 °C-cm/W after inundation
Thermal Resistivity: 57.7 °C-cm/W after free draining
Thermal Resistivity: 1071.1 °C-cm/W @ 5% water content
Thermal Resistivity: 1167.0 °C-cm/W @ 3% water content
Thermal Resistivity: 1158.6 °C-cm/W ~dry

Reviewed By: GET

SUMMARY OF THERMAL CONDUCTIVITY TESTING ASTM D5334

Sample: S-(13-16)
Depth: 45-62 ft.



Test Summary

Sample Description: SM, gray silty fine sand, trace fine gravel
Sample Type: compacted medium dense

	Water Content (%)	Dry Density (pcf)	Total Density (pcf)
As-Compacted:	13.5	118.4	134.4

Note: Test densities may vary from initial densities due to water content variation.

Thermal Resistivity:	34.8	°C-cm/W	Initial
Thermal Resistivity:	32.6	°C-cm/W	after inundation
Thermal Resistivity:	40.3	°C-cm/W	after free draining
Thermal Resistivity:	68.9	°C-cm/W	@ 5% water content
Thermal Resistivity:	93.3	°C-cm/W	@ 3% water content
Thermal Resistivity:	244.7	°C-cm/W	~dry

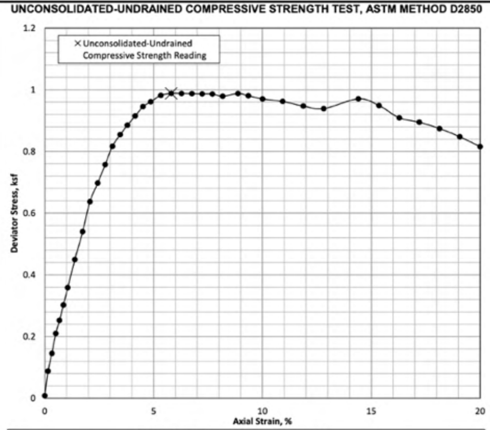
Reviewed By: GET

SUMMARY OF THERMAL CONDUCTIVITY TESTING ASTM D5334

Sample: S-(17-21)
Depth: 65-87 ft.

APPENDIX E – UNCONSOLIDATED-UNDRAINED COMPRESSIVE STRENGHT

UNCONSOLIDATED-UNDRAINED COMPRESSIVE STRENGTH TEST, ASTM METHOD D2850



Specimen and Material Property Information

Sample Type: Intact tube sample

Description and/or Classification: SC, Gray, Clayey sand with peat

Cell Pressure (ksf)	Water (1) Content (%)	Wet Unit Weight (pcf)	Dry Unit (1) Weight (pcf)	Void Ratio (-)	Saturation (2) (%)	Length (inch)	Diameter (inch)	L/D (-)	LL/ PL (-)	PI (-)	Specific (2) Gravity (-)
0 (Initial)	34.7	115.7	85.9	0.93	98.8	6.008	2.845	2.1	33	10	2.66
1.5	34.7	115.9	86.1	0.93	99.2	6.004	2.843	2.1	23		

Failure Summary

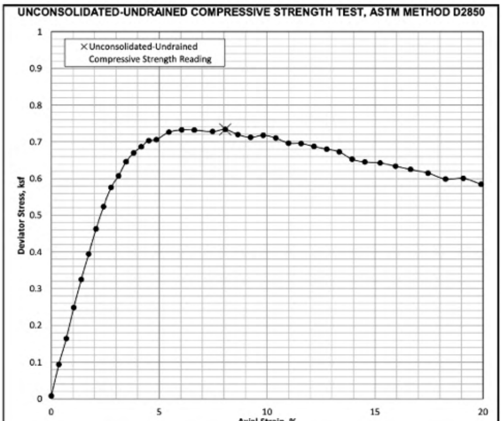
U-U Compressive Strength (ksf)	U-U Shear Strength, s_u (ksf)	Strain to Peak (%)	Strain Rate (%/min)
0.99	0.495	5.8	0.71

FAILURE SKETCH

Remarks and Notes:
(1) Water Content determined after shear from partial specimen.
(2) Assumed specific gravity

UNCONSOLIDATED-UNDRAINED COMPRESSION TEST
BH-A **Sample: ST-1**
Section: B Depth: 23.05 ft.

UNCONSOLIDATED-UNDRAINED COMPRESSIVE STRENGTH TEST, ASTM METHOD D2850



Specimen and Material Property Information

Sample Type: Intact tube sample

Description and/or Classification: CH, Gray, Fat Clay. Trace sand and plant matter noted.

Cell Pressure (ksf)	Water (1) Content (%)	Wet Unit Weight (pcf)	Dry Unit (1) Weight (pcf)	Void Ratio (-)	Saturation (2) (%)	Length (inch)	Diameter (inch)	L/D (-)	LL/ PL (-)	PI (-)	Specific (2) Gravity (-)
0 (Initial)	48.6	107.0	72.0	1.28	99.7	5.982	2.851	2.1	56	33	2.63
0.74	48.6	107.1	72.1	1.28	99.9	5.980	2.850	2.1	23		

Failure Summary

U-U Compressive Strength (ksf)	U-U Shear Strength, s_u (ksf)	Strain to Peak (%)	Strain Rate (%/min)
0.73	0.365	8.1	0.72

FAILURE SKETCH

Remarks and Notes:
(1) Water Content determined after shear from partial specimen.
(2) Assumed specific gravity

UNCONSOLIDATED-UNDRAINED COMPRESSION TEST
BH-B **B Sample: ST-1**
Section: C Depth: 13.6 ft.

Journal of Engineering Research ISSN 2764-1317 DOI 10.22533/at.ed.3172109125