Final Preliminary Geotechnical Report – Rev1

Proposed Development, 126 and 140 Bradford Street, Barrie, Ontario

Crown (Bradford) Developments Inc.

05 May 2023 (Revised 04 December 2023)

→ **The Power of Commitment**

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1. Introduction

Crown (Bradford) Developments Inc. (Crown) has retained GHD Limited (GHD) to provide geotechnical engineering services to support potential purchasing of the property and preliminary design for the proposed residential and retail development at 126 and 140 Bradford Street in Barrie, Ontario, collectively hereinafter referred to as the “Site”.

As per email received from Crown dated November 27, 2023, GHD understands that the proposed development concept plan contains one (1) mixed-use building with two residential towers, each forty-five (45) storeys in height and a shared six (6) storey podium with commercial at-grade. As a whole, the proposal will result in a total gross floor area (GFA) of approximately 51,383 m^2, comprised of 51,049 m^2 of residential GFA and 333 m2 of retail/commercial GFA.

It is noted that no underground structure, such as underground parking levels, is planned for the development. The Site is currently utilized for multiple commercial operations, and occupied by the following businesses: NAPA Autocare Centre, Barrie Foot Clinic and Get Moving Physio. The Site location is shown on **Figure 1.**

In November 2022, GHD advanced four boreholes and four monitoring wells on the Site for environmental purposes. The boreholes were spread across the Site and were not focused within the proposed building footprints.

The purpose of this geotechnical investigation was to obtain information on the subsurface soil and groundwater conditions within the proposed development footprints, by means of a limited number of boreholes and based on our interpretation of the data, to provide engineering recommendations on the geotechnical aspects of preliminary design of the project. It should be noted that the City of Barrie (the City) has a Drinking Water Protection Policy that applies to the proposed development at the Site. As instructed in a letter received from the City referenced as “Subsurface investigations extending into aquitards” dated February 13, 2023, the subsurface investigation was not to be extended beyond 3 m in Aquitard ‘C2’ and all works were to be terminated above 195 m amsl (above mean sea level) to ensure protection of the municipal aquifer.

The investigation and reporting were carried out in general accordance with the scope of work provided in our proposal dated February 10, 2023 and in accordance with the City’s instructions.

The factual data, interpretations and preliminary recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or Site location. This report should be read in conjunction with the Statement of Limitations appended to this report. The reader’s attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2. Investigation Procedures

2.1 Health & Safety Plan

Upon project initiation, a Site-specific Health, and Safety Plan (HASP) was prepared in accordance with the requirements of the Occupational Health and Safety Act for implementation during the field investigation program. The HASP presents the visually observed Site conditions to identify potential physical hazards to field personnel. Required personal protective equipment was also listed in the HASP. It was mandatory for all GHD personnel, involved in the field program, to read the HASP and have a copy of the HASP available at the Site during the investigative work. Health and Safety requirements in the HASP were implemented during the field investigation program and the HASP was maintained on the Site during all field activities.

2.2 Utility Clearances

All applicable utility companies (gas, hydro, Bell, Rogers, etc.) were contacted through Ontario One-Call prior to the commencement of the drilling program. In addition, a private utility locator (Premier Locates) was retained to demarcate the locations of the privately owned utilities within the area of the boreholes to ensure that the private utilities were not damaged during the investigation work. A GHD representative was present on Site during demarcation of private utilities and reviewed the site conditions and located boreholes and testing locations based on access restrictions. Based on the site visit, GHD confirmed that the boreholes could be advanced at the proposed locations.

2.3 Borehole Advancement Activities

The drilling program for this geotechnical investigation was carried out on February 21 and February 22, 2023, and consisted of advancing of two (2) boreholes (designated as Boreholes BH5-23 and BH6-23) to the depths of 28.0 mbgs (below ground surface). The depth of boreholes was determined by the City's Drinking Water Protection Policy. No monitoring wells were installed on Site during geotechnical investigation. Along with the boreholes, Cone Penetration Tests were also performed at five locations (designated as SCPT23-01 through SCPT23-04), to depths ranging from 14.4 m to 28.9 mbgs. The boreholes and CPT holes were advanced at the locations shown on the Borehole and CPT Location Plan (**Figure 1**).

The drilling work was carried out utilizing a CME 55 track-mounted drill rig and CME 75 truck mounted drill rig supplied and operated by the drilling subcontractor 3D Drilling Inc. under the full-time supervision of GHD technical representatives.

The boreholes were advanced using hollow stem augers (108 mm inside diameter) to the depth of approximately 3.0 mbgs. Thereafter, the drilling method was switched to wash boring with HQ casing for BH5-23 and mud rotary with a tricone bit for BH6-23 to the borehole termination depth, using bentonitic mud mixtures to balance hydrostatic uplift pressures. Representative, disturbed soil samples were obtained at 0.75 m to 1.5 m interval of depth using a 50 mm outer-diameter (O.D.) split-spoon sampler advanced by a 63.5 kg automatic hammer dropping approximately 760 mm in accordance with Standard Penetration Test (SPT) procedures described in ASTM D1586[1]. A detailed record of each borehole is presented in **Appendix A**.

The GHD technical representatives logged the material encountered in the boreholes and examined the samples as they were obtained. The recovered samples were sealed in clean, airtight containers and transferred to GHD's laboratory, where they were reviewed by a geotechnical engineer.

To measure the in situ undrained shear strength of the cohesive soil layer encountered in BH6-23, Vane Shear Tests (VST) were also conducted. The recorded undrained shear strengths are provided on the borehole logs in **Appendix A**.

Pressuremeter testing (PMT) was carried out by our subcontractor In-Depth Geotechnical Inc. using a TEXAM unit within borehole BH6-23 at three different depths during the drilling with the purpose of evaluating specific parameters related to shear strength and deformation properties of the encountered soils. Results of PMT are presented in **Appendix C.**

Groundwater level observations and measurements were made in the boreholes as drilling proceeded and upon completion of overburden drilling. However, it is noted that a mud mixture was utilized as needed to maintain basal and wall stability in all boreholes. Consequently, water level readings immediately following drilling completion are not considered representative of stabilized natural groundwater levels.

The boreholes were backfilled and sealed in accordance with Ontario Regulation 903, as amended, and in general accordance with the drilling and backfilling methodology outlined in the document "12493831-LRT-3-Barrie Deep Drilling Request – 126, 136 and 140 Bradford Street, Barrie, Ontario", submitted to the City on January 25, 2023. Excess soil cuttings were collected in drums and temporarily stored on Site prior to disposal off Site by GHD.

Piezocone Penetration Tests were conducted on February 27, 2023, by our subcontractor ConeTec, at five locations SCPT23-01, SCPT23-02, SCPT23-03A, SCPT23-03B, and SCPT23-04. Downhole Shear wave velocity

[1] ASTM D1586-18 - Standard Test Method for Standard Penetration Test and Split-Barrel Samplings of the soil, ASTM International, West Conshohocken, PA 2015

measurements at 1 m depth intervals and porewater pressure dissipations tests were conducted in conjunction with the CPTs. Only SCPT23-04 could be advanced to the planned termination depth of the tests. The remainder of the tests met early refusal (excessive rod flex or approaching the limiting thrust pressure of the rig) at depths ranging between approximately 15 m and 20 mbgs. The CPT results are presented in **Appendix D.**

The as-drilled borehole and CPT locations and ground surface elevations were obtained using a handheld GPS with a vertical and horizontal accuracy of ±2 cm. The locations given on the Borehole Records are positioned relative to UTM Coordinates (UTM-17T NAD83) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. A summary of the borehole locations, geographic coordinates, ground surface elevation, and drilled depths are presented in the table below.

Table 2.1 Borehole and CPT Locations and Depths

Geotechnical Borehole and CPT ID	Northing (m)	Easting (m)	Ground Surface Elevation (mamsl*)	Termination Depth (mbgs)
BH5-23	4914988.3	604036.4	223.7	28.0
BH6-23	4914936.6	604037.2	223.6	28.0
SCPT23-01	4914988.7	604042.1	223.4	20.2
SCPT23-02	4914936.6	604042.5	223.6	20.3
SCPT23-03A	4915000.4	604009.9	224.6	14.4
SCPT23-03B	4915001.9	604009.9	224.6	14.6
SCPT23-04	4914952.9	604010.9	224.1	28.9

*Note: meters above mean sea level

2.4 Laboratory Testing

Prior to conducting geotechnical laboratory testing, the soil samples extracted from the boreholes were subjected to visual and tactile examination by an experienced GHD staff who confirmed the field descriptions and selected representative samples for detailed testing.

Geotechnical laboratory testing was conducted in accordance with the American Society for Testing and Materials (ASTM) and Canadian Council of Independent Laboratories (CCIL) applicable standards. Laboratory testing consisted of natural water content tests on thirty-six (36) select soil samples, grain size distribution analyses (sieve and hydrometer testing) on nine (9) select soil samples, and Atterberg Limits testing on eight (8) soil samples.

The results of water content, grain size distribution and Atterberg Limits tests are reported on the boreholes records presented in **Appendix A**. The associated laboratory test results are provided in **Appendix B**.

The soil testing program and soil classification conformed to the latest edition of the following standards:

– ASTM D6913-Standard Test Method for Particle Size Distribution (Gradation) of Soils using Sieve Analysis
– ASTM D7928-Standard Test Method for Particle Size Distribution (Gradation) of Fine-Grained Soils using the Sedimentation (Hydrometer) Analysis
– ASTM D4318-Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils
– ASTM D2216-Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass Scope
– ASTM D2487-Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

3. Site Geology and Subsurface Conditions

3.1 Regional Geology and Previous Investigations

The Site is located in a physiographic region of Simcoe Lowlands with sand plains as predominant physiographic landform as delineated in The Physiography of Southern Ontario (Chapman and Putman, 1984)[2]. Regional surficial geology mapping[3] of the area indicates that the Site is underlain by coarse-textured glaciolacustrine deposits. Based on bedrock geology mapping of the area, the bedrock consists of limestone, dolostone, shale, arkose, and sandstone of the Shadow Lake Formation, Simcoe Group (Upper Ordovician). Based on the Bedrock Topography Map[4], bedrock depth in the area is more than 100 mbgs.

3.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the geotechnical investigation and the results of the laboratory tests carried out on selected soil samples are presented on the borehole records provided in **Appendix A**. The results of the geotechnical laboratory testing are presented in **Appendix B**. The results of in-situ field tests, such as SPT "N" values are presented on the borehole records and summarized in this report are uncorrected. The recorded undrained shear strengths are provided on the borehole logs in **Appendix A**. The *Notes on Borehole and Test Pit Reports* are also included in **Appendix A** to assist in the interpretation of the borehole records. Results of Pressuremeter test and Cone Penetration Tests are presented in **Appendix C** and **Appendix D,** respectively.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling process and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

Generally, the subsurface conditions encountered at the Site were consistent with the regional geology. In summary, the subsurface conditions encountered within the boreholes below asphalt and granular base consisted of very loose to compact sand fill material, underlain primarily by non-cohesive soils characterized by increasing silt content with depth.

The soils consisted of very loose to very dense, sand, silty sand, sandy silt, sand and silt ranging from a depth of 2.3 m (Elevation 221.3) to 12.2 mbgs in Borehole BH5-23 (Elevation 211.5) and to 22.9 mbgs in Borehole BH6-23 (Elevation 200.8 m). A layer of cohesive silty clay was encountered in Borehole BH6-23 at depth from 22.9 mbgs to 25.9 mbgs (Elevation from 200.8 m to 197.7 m). A lower non-cohesive silt deposit was found in all boreholes underlying the upper sand and silt and silty clay deposits to the termination depth of 28.0 mbgs (Elevation 195.7 and 195.6 m).The Soil Behaviour Type (SBT) indicated by the CPTs generally agreed with the corresponding borehole stratigraphy.

Detailed descriptions of subsurface conditions are provided in the following sections of this report. The subsurface conditions are described in accordance with the Unified Soil Classification System and the Canadian Foundation Engineering Manual, (CFEM 2006).

3.2.1 Fill

Fill material comprising of sand was encountered immediately below asphalt and granular base in all boreholes at the Site. The fill extended to a depth of 2.3 mbgs (Elevations ranging from 221.4 m to 221.3 m).

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[2] Chapman, L.J. and Putnam, D.F., 1984. The Physiography of Southern Ontario. Ontario Geological Survey, Special Volume 2.
[3] Surficial Geology of Southern Ontario - Miscellaneous Release-Data 128-REV. Ontario Geological Survey, 2010.
[4] Bedrock Topography Barrie Area - Map P.3212. Mines and Minerals Division Ontario Geological Survey, 1993.

SPT 'N' values within the fill material ranged between 0 and 10 blows per 0.3 m of penetration, suggesting a very loose to compact relative density.

Grain size testing was carried out on one (1) representative sample of the fill material. The results are summarized in the table below and presented in **Appendix B** and on the borehole records in **Appendix A.**

Table 3.1 Grain size Distribution Analysis Test Results – Fill Material

Borehole Identification Number	Sample Number	Depth (mbgs)	Grain Size Distribution			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH6-23	SS3	1.5 – 2.1	5	88	7	0

Moisture contents measured on samples of the fill material ranged between 5% and 23%.

3.2.2 Native Deposits – Sand, Silty Sand/Sandy Silt, Silt and Sand, Silt

A native sand deposit was encountered immediately below the fill material in borehole BH5-23 at depth of 2.3 mbgs (Elevation 221.4 m) and extended to the depth of 4.6 mbgs (Elevation 219.1 m). SPT 'N' values within the sand deposit ranged between 1 blow to 4 blows per 0.3 m of penetration, suggesting a very loose relative density. Moisture contents measured on samples of the fill material ranged between 26% and 29%.

Silty sand, sandy silt, sand & silt deposits were encountered below the sand layer in borehole BH5-23 to a depth of 12.2 mbgs (Elevation 211.5 m), and below the fill material in borehole BH6-23 to a depth of 22.9 mbgs (Elevation 200.8). SPT 'N' values within the silty sand, sandy silt deposit ranged between 5 blows per 0.3 m of penetration to 50 blows per 0.11 m of penetration, suggesting loose to very dense relative density. Moisture contents measured on samples of these soils ranged between 18% and 30%.

A deposit of silt was encountered below the upper sand and silt deposit in borehole BH5-23, and below a silty clay deposit in BH6-23 at depths ranging from 12.2 mbgs to 25.9 mbgs (Elevations ranging from 211.5 m to 197.7 m) and extended to the termination depth of 28.0 mbgs (Elevations 195.7 m and 195.6 m). SPT 'N' values within the silt deposit ranged between 14 blows per 0.3 m of penetration and 50 blows per 0.13 m of penetration, suggesting compact to very dense relative density. Moisture contents measured on samples of the silt ranged between 14% and 25%.

Grain size testing was carried out on seven (7) select soil samples within these deposits and the results are provided in **Appendix B** and on the borehole records in **Appendix A**. The results are also summarized in the table below.

Table 3.2 Grain size Distribution Analysis Test Results – Silty Sand/Sandy Silt, Sand & Silt, Silt

Borehole Identification Number	Sample Number	Depth (mbgs)	Grain Size Distribution			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH5-23	SS6	4.6 – 5.2	0	63	35	2
BH5-23	SS9	9.2 – 9.8	0	60	38	2
BH5-23	SS12	13.7 – 14.3	0	18	74	8
BH5-23	SS16	19.8 – 20.4	0	22	72	6
BH5-23	SS19	24.4 – 25.0	0	15	78	7

Borehole Identification Number	Sample Number	Depth (mbgs)	Grain Size Distribution			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH6-23	SS7	6.1 – 6.7	1	65	31	3
BH6-23	SS11	12.2 – 12.8	0	22	68	10

Atterberg limits testing was carried out on seven (7) representative samples within these deposits and the samples were observed to be non-plastic. The results of these tests are provided in **Appendix B.**

3.2.3 Silty Clay

A silty clay deposit was encountered underneath the sandy silt deposit in borehole BH6-23 at a depth of 22.9 mbgs (Elevation 200.8 m) and extended to a depth of 25.9 mbgs (Elevation 197.7 m). A single SPT ‘N’ value within the silty clay deposit was recorded as 7 blows per 0.3 m of penetration. However, conventional vane shear testing performed within this deposit indicated undisturbed undrained shear strength value greater than 150 kPa. Based on the Pressuremeter test results, the inferred undrained shear strength value for this deposit was measured to be as high as 160 kPa. The inferred silty clay encountered in SCPT23-04 at a similar depth interval also indicated undrained shear strength values of between 125 and 150 kPa. Based on the above information, this deposit is of very stiff consistency in general. Grain size testing was carried out on one (1) select soil samples within the silty clay deposit and the results are provided in **Appendix B** and on the borehole records in **Appendix A**. The results are also summarized in the table below.

Table 3.3 Grain size Distribution Analysis Test Results – Silty Clay

Borehole Identification Number	Sample Number	Depth (mbgs)	Grain Size Distribution			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH6-23	SS15	22.9 – 23.5	1	0	57	42

Moisture content measured on sample of the silty clay deposit was recorded as 29%.

Atterberg limits testing was carried out on one (1) representative sample within the silty clay deposit and the results are provided in **Appendix B** and on the borehole records in **Appendix A**. The results are also summarized in the table below.

Table 3.4 Atterberg Testing Results – Silty Clay

Borehole Identification Number	Sample Number	Depth (mbgs)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Classification
BH6-23	SS15	22.9 – 23.5	37	20	17	CI

3.3 Groundwater Conditions

Groundwater was encountered during drilling at a depth of 2.3 mbgs (Elevation 221.4 m) in BH5-23 and 1.6 mbgs (Elevation 222.0 m) in BH6-23.

The groundwater observations were obtained from the open boreholes upon completion of drilling each borehole. Groundwater at completion of drilling was at the surface, however, it is noted that mud rotary/wash boring methods were utilized and, therefore, the water level upon completion of drilling is not considered realistic. The results of the groundwater observations are summarized in the borehole logs in the **Appendix A**.

It should be noted that the groundwater level is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

Based on the CPT porewater pressure dissipation tests, the inferred depth of groundwater table generally ranged between 2.0 and 2.5 mbgs, with the exception of the SCPT23-03 location that indicated a groundwater depth ranging between 2.9 and 3.5 mbgs.

Based on the GHD Preliminary Hydrogeological Assessment Report dated February 10, 2023, the groundwater levels measured (dated January 23, 2023) in the monitoring wells MW1-23 through MW4-23 ranged from 2.2 to 3.1 mbgs.

4. Engineering Discussion and Recommendations

This section of the report provides preliminary geotechnical engineering design recommendations for the proposed development.

The following recommendations are based on the interpretation of the factual data obtained from the boreholes and other testing advanced during this subsurface investigation. It should be noted that the geotechnical investigation, discussion, and recommendations are intended for use by the designers only and should not be relied upon for any other purpose or by any other parties, including the construction or design-build contractor. Contractors bidding on or undertaking any work at the proposed development should examine the factual results of the assessment, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of this factual data as it affects their proposed construction techniques, equipment capabilities, costs, sequencing, scheduling, and the like. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Comments, techniques, or recommendations pertaining to construction should not be construed as instructions to the contractor.

4.1 Seismic Site Classification

GHD conducted a Multichannel Analysis of Surface Waves (MASW) testing as part of the geotechnical investigation for the proposed development. The results of the findings are presented in **Appendix E**. Based on the findings of the investigation, a weighted average shear wave velocity of 327 m/s was calculated for a depth of 30 mbgs.

Based on the Seismic Piezocone Penetration Test findings at locations SCPT23-01 through SCPT23-04, SCPT23-04 was chosen as the reference for calculating the weighted average shear wave velocity due to the deepest depth and inclusion of the relatively weaker ground conditions at the southern portion of the Site. The weighted average shear wave velocity at SCPT23-04 was calculated as 272 m/s. Based on the above information from SCPT and MASW results, this Site may be classified as Site Designation X_{272} in accordance with Table 4.1.8.4.A of the National Building Code of Canada (NBCC) 2020 Volume 1, or Site Class D in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012, based on the measured shear wave velocities.

4.2 Spectral Response Values and Seismic Performance Category

Based on the location of the Site, the reference Site Class X_{272} spectral acceleration values shown in the table below were obtained based on NBCC 2020 seismic Hazard Tool Hazard Tool from Seismic Canada.

Table 4.1 Peak Ground Acceleration (PGA), Velocity (PGV) Values and Design Spectral Acceleration (S) Values

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period) Site Class X_{272}	5% Exceedance in 50 years (975-year return period) Site Class X_{272}	2% Exceedance in 50 years (2,475-year return period) Site Class X_{272}
PGA (g)	0.042	0.065	0.107
PGV (m/s)	0.041	0.067	0.118
S (0.2) (g)	0.084	0.129	0.210
S (0.5) (g)	0.072	0.112	0.184
S (1.0) (g)	0.040	0.064	0.108
S (2.0) (g)	0.0184	0.0305	0.0531
S (5.0) (g)	0.00433	0.00769	0.01440
S (10.0) (g)	0.00149	0.00263	0.00485

4.3 Liquefaction Potential

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface deformations) and under undrained conditions generate excess pore water pressures. The excess pore water pressures can lead to sudden temporary losses in soil strength. Where existing static shear stresses are present, the loss of soil strength can lead to loss of bearing resistance, slope instability, lateral spreading of the ground, settlement of the ground and loss of lateral resistance.

The liquefaction susceptibility of the soils at the Site was evaluated in accordance with *Idriss & Boulanger, 2008*[5]*, Anderson et. Al., 2007*[6] *and Rocscience Settle3 software*[7] by evaluating the SPT $(N1)_{60cs}$ for the existing fill material and native non-cohesive soils at all boreholes and CPT locations. The liquefaction analysis was carried out using in-situ testing data collected at the borehole and CPT locations. The parameter, $(N1)_{60cs}$ is based on the SPT "N" value obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies and fines content. The design groundwater level was determined based on the highest measured groundwater level measured during drilling in both boreholes. The results of the liquefaction assessment indicate that the saturated non-cohesive deposits encountered at the Site are likely not liquefiable during the 2,475-year design earthquake.

4.4 Depth of Frost Penetration

The estimated depth of frost penetration at this Site is 1.6 m, based on Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). Any shallow foundations/spread footings within the Site must be founded at least 1.6 m below the lowest adjacent final grade to provide adequate protection against frost penetration. Where 1.6 m of soil cover is not possible, rigid polystyrene foam insulation maybe used as an alternative. For preliminary design purposes, 25 mm (1-inch) of rigid polystyrene foam insulation can be considered equivalent to 0.3 m of soil cover.

[5] Idriss, I.M. and Boulanger, R.W., 2008. Soil liquefaction during earthquakes. Earthquake Engineering Research Institute.
[6] Anderson, D. L., Byrne, P. M., DeVall, R. H., Naesgaard, E., & Wijewickreme, D. (2007). Task Force Report-Geotechnical design guidelines for buildings on liquefiable Sites in Greater Vancouver in accordance with NBCC 2005. Greater Vancouver Seismic Geotechnical Design Task Force, Vancouver, BC.
[7] Settle3D Liquefaction Theory Manual, Rocscience 2023.

4.5 Building Foundations

As per email received from Crown dated November 27, 2023, GHD understands that the proposed development concept plan contains one (1) mixed-use building with two residential towers, each forty-five (45) storeys in height and a shared six (6) storey podium with commercial at-grade. It is noted that no underground structure, such as underground parking levels, is planned for the development.

Based on the investigation results, the stratigraphy and soil strength vary across the Site. Additional geotechnical investigation will have to be performed prior to detail design to provide sufficient coverage across the proposed building footprints. The soils at the north end of the Site (BH5-23, SCPT23-02, SCPT23-03) generally appear to be stronger than those at the south end (BH6-23, SCPT23-01, SCPT23-04). The top approximately 14 to 15 m of the soil strata are in a very loose to compact state of relative density and therefore the geotechnical capacity of shallow foundations will be relatively low.

For foundation support of the proposed buildings, structural slab/raft supported by deep foundations such as caissons/drilled shafts terminating in the very dense native silt/sandy silt deposit is considered suitable. Another possible foundation alternative is large diameter helical shafts founded within the very dense native silt/sandy silt deposit, especially for the planned podium structure between the towers. The feasibility of this option and preliminary foundation capacity is to be checked with the specialist helical pile contractors. Given the urban setting of the project, the currently available geotechnical information and the expected relatively high column loads for high-rise structures, driven piles are not considered a feasible foundation option at this stage.

Temporary liners would be required during caisson installation to support the ground and groundwater within the fill material and the water-bearing non-cohesive deposits. The water level inside the casing is to be kept at least 1.5 m above the groundwater level to counterbalance the external hydrostatic pressure and prevent inflow of fine sands and silts. The concrete must be placed using tremie methods, in accordance with OPSS.MUNI 903 and immediately following cleaning and inspection of the base. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained a minimum of 1 m below the surface of the wet concrete during placement, in accordance with OPSS.MUNI 903 and to minimize “necking”.

It is expected that the liner would be installed and removed using a vibratory hammer. In this case, vibration monitoring is recommended during liner installation and removal. Depending on the length of the caissons and groundwater pressures, polymer slurry drilling fluid may be a preferred alternative for constructing the caissons.

If grade raises are planned across the Site, resulting ground settlements and potential down drag load on piles will have to evaluated.

4.5.1 Founding Elevations

Given the depth limitation imposed by the City, the boreholes were drilled to a maximum depth of 28.0 mbgs (Elevations of 195.7 m and 195.6 m). Ground conditions (soil type, strength and stiffness parameters) are unknown below this depth. Therefore, based on the existing data, the caissons should be founded at a maximum depth of about 22.0 mbgs, within the very dense native silt/sandy silt deposit. This results in an approximate caisson base elevation of 202 m.

4.5.2 Geotechnical Resistance

For preliminary/feasibility design purposes, the approximate Ultimate Limit State (ULS) and Serviceability Limit State (SLS) resistances have been estimated for 20 m long, 1.2 m and 1.6 m diameter caissons (starting beneath caisson caps founded at or below frost depth) and shown in the following tables. The geotechnical ultimate capacities were calculated based on contribution from both shaft/side resistance and end bearing. The end bearing component in the non-cohesive soils at the north end was appropriately reduced considering the influence of various factors such as pile displacement (taken as 5% of pile diameter) and construction procedures per Federal Highway Administration Drilled

Shaft design guidelines (2018)[8]. At the southern end, the end bearing resistance was conservatively estimated based on the properties of very stiff silty clay layer encountered below approximate elevation 201 m. The geotechnical capacities will have to confirmed with load tests/O-Cell tests, which will also allow the use of a higher geotechnical resistance factor in design.

It is to be noted that SLS resistance for a single caisson may not control design as pile group settlements will have to be checked separately when preliminary foundation arrangement is known.

Table 4.2 ***Factored*** *geotechnical resistances based on BH5-23 & SCPT23-02 – caissons at northern portion of Site*

Drilled shaft diameter (m)	Founding stratum	Unfactored ULS Resistance (kN)	Factored ULS Resistance, fULS (kN) with 0.4 resistance factor	SLS Reaction (kN), for 25 mm settlement
1.2	Very Dense Silt/Sandy Silt	7,750	3,100	Greater than factored ULS
1.6		11,850	4,740	Greater than factored ULS

Table 4.3 ***Factored*** *geotechnical resistances based on BH6-23 & SCPT23-01/SCPT23-04 – caissons at southern portion of Site*

Drilled shaft diameter (m)	Founding stratum	Unfactored ULS Resistance (kN)	Factored ULS Resistance, fULS (kN) with 0.4 resistance factor	SLS Reaction (kN), for 25 mm settlement
1.2	Very Dense Silt/Sandy Silt underlain by very stiff silty clay	4,300	1,720	Greater than factored ULS
1.6		6,360	2,540	Greater than factored ULS

The geotechnical resistances and settlement are dependent on the building location, foundation dimensions and founding elevations and must be reviewed at detailed design stage if any of the above building parameters differ significantly from those assumed at this stage.

If greater geotechnical capacities are required, the soil conditions will have to be verified through deeper boreholes potentially extending to the deep municipal aquifer, subject to the City's permission. The use of larger diameter caissons can also be considered for greater geotechnical capacities. The use of ground improvement (grouting or rammed aggregate piers) can be considered to improve the mechanical properties of the upper soils to provide additional bearing resistance under the raft/structural slab, to supplement the caissons.

4.5.3 Resistance to Lateral Loads

The design of caissons subjected to lateral loads should take into account such factors as the batter/inclination of the caisson (if any), the relative rigidity of the caisson to the surrounding soil, the fixity conditions at the head of the caisson (i.e., at the caisson cap level), the structural capacity of the caisson to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the caisson and group effects.

For a longer, more flexible caisson, the maximum yield moment of the caisson may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistance should be evaluated to establish the governing case.

Where the ground conditions are generally competent and the lateral loads on caissons are relatively small such that the maximum lateral caisson deflections will be relatively small, the resistance to lateral loading in front of a single caisson can be estimated using subgrade reaction theory in accordance with the Canadian Foundation Engineering Manual (CFEM 1992), as outlined below. However, the response of a caisson to lateral loads is highly non-linear and methods that assume linear behaviours (such as subgrade reaction theory) are most appropriate where the maximum caisson deflections are less than 1 per cent of the caisson diameter, where the loading is static (no cycling) and where

[8] Drilled Shafts: Construction Procedures and Design Methods, Publication No. FHWA-NHI 18-024, FHWA GEC 010, September 2018

the caisson material is linear (CFEM 1992). Where these conditions are not met, and where required for the structural engineering model, the non-linear lateral behaviour of the soils should be considered by the use of p-y curves. GHD can assist the Structural Engineer with the selection of and/or lateral resistance analysis using p-y curves, as required.

For preliminary design purposes, the factored serviceability geotechnical response of the soil in front of the caissons under lateral loading at this Site may be calculated using subgrade reaction theory suggested in Terzaghi (1955)[9] and the CFEM 1992, where the coefficient of horizontal subgrade reaction, k_h (kPa/m) is based on the equation given below.

For cohesive soils:

$$k_h = 67\frac{S_u}{B}$$

Where: S_u is the undrained shear strength of the soil (kPa)

B is the caisson diameter/width (m)

For non-cohesive soils:

$$k_h = \frac{\eta_h Z}{B}$$

Where: η_h is the constant of horizontal subgrade reaction (kPa/m),

Z is the depth (m) below ground surface,

B is the caisson diameter/width (m)

The following values of η_h (Terzaghi, 1955[9] and Reese, 1975[10]) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical caisson. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the nonlinear nature of the soil behaviour (such that k_h is a function of deflection).

Table 4.4 Lateral design parameters – caissons

Soil Unit	Approximate Elevation (m)	η_h (kPa/m)	S_u (kPa)
Existing Non-Cohesive Fill Material	223.7 – 221.3	3,000	-
Native Non-Cohesive Deposits	221.3 – 195.6	10,000	-
Silty Clay Deposit	200.8 – 197.7	-	125

The upper zone of the soil (down to a depth below the caisson cap equal to 1.5 times B (after Broms, 1964[11,12], where B is the caisson diameter) should be neglected in the calculation of lateral resistance of the caisson to account for disturbance effects during installation.

Group action for lateral loading should be considered when the caisson spacing in the direction of the loading is less than 6 to 8 caisson diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a Reduction Factor, R, as follows:

[9] Terzaghi, K., 1955. Evaluation of coefficients of subgrade reaction. Geotechnique, 5(4), pp.297-326.
[10] Reese, L.C., Cox, W.R. and Koop, F.D., 1974, May. Analysis of laterally loaded piles in sand. In Offshore Technology Conference. OnePetro.
[11] Broms, B.B., 1964. Lateral resistance of piles in cohesive soils. Journal of the soil mechanics and foundations division, 90(2), pp.27-63.
[12] Broms, B.B., 1964. Lateral resistance of piles in cohesionless soils. Journal of the Soil Mechanics and Foundations Division, 90(3), pp.123-156.

Table 4.5 Subgrade Reaction Reduction Factor R for Group Action in Lateral Loading

Spacing in Direction of Loading (B = Pile Diameter)	Subgrade Reaction Reduction, R
8B	1.00
6B	0.70
4B	0.40
3B	0.25

4.6 Slab-on-grade Support

Considering the presence of very loose to compact fill material, the use of slab-on-grades carry the risk of long-term differential settlements and cracking. As such, ground floor slab for the buildings should be designed as structural slabs supported on deep foundations such as caissons.

4.7 Preliminary Pavement Design

The subsequent sections include preliminary pavement design recommendations for the construction of the driveway and surface parking lot for the proposed development.

4.7.1 Earth Fill Condition

The existing earth fill at the Site extended to an approximate depth of 2.3 mbgs at the borehole locations. Non-cohesive fill material generally consisting of sand trace gravel trace silt was encountered below the topsoil in boreholes BH5-23, BH6-23, and MW1-23 to MW4-23. The fill material was generally underlain by native sand and silty sand to sandy silt to the borehole termination depths.

SPT 'N' values within the non-cohesive fill material ranged from 0 to 8 blows per 300 mm penetration, indicating a very loose to loose state of compactness.

4.7.2 Frost Susceptibility of Subgrade Soils

The frost susceptibility of the subgrade soils is assessed using the Ministry of Transportation of Ontario's guidelines, which are based on the presence of silt sized particles in the 75µm to 5µm range. Based on the classification of the soils, it is assumed that the existing fill material underlying the topsoil and within the frost penetration depth is of Low to Susceptibility to Frost Heaving (LSFH).

4.7.3 Recommended Preliminary Pavement Structure

4.7.3.1 Driveway Pavement Structure

The following table summarizes the minimum pavement structures recommended for the design of the proposed driveways.

Table 4.6 Driveways - Flexible Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty Pavement Design (Driveways)
Surface Course Asphaltic Concrete HL3 (OPSS.MUNI 1150)	92% to 96.5% Maximum Relative Density (OPSS.MUNI 310)	40 mm
Base Course Asphaltic Concrete HL8 (OPSS.MUNI 1150)	92% to 96.5% Maximum Relative Density (OPSS.MUNI 310)	80 mm
Base Course: Granular A (OPSS.MUNI 1010) or 19 mm Crusher Run (OPSS.MUNI 1004)	100% SPMDD	150 mm
Subbase Course: Granular B Type I (OPSS.MUNI 1010)	98% SPMDD	650 mm (1)
Notes: (1) The recommended sub-base course thickness is intended to account for the weak sand fill layer and the presence of shallow groundwater. The additional sub-base thickness will provide additional support and drainage.		

4.7.3.2 Parking Lots Pavement Structure

The following table summarizes the minimum pavement structures recommended for the design of the proposed surface parking lots.

Table 4.7 Parking Lots - Flexible Pavement Structure

Pavement Layer	Compaction Requirements	Light Duty Pavement Design (Car Surface Parking Lot)
HL-3Surface Course Asphaltic Concrete HL3 (OPSS.MUNI 1150)	92% to 96.5% Maximum Relative Density (OPSS.MUNI 310)	40 mm
Base Course Asphaltic Concrete HL8 (OPSS.MUNI 1150)	92% to 96.5% Maximum Relative Density (OPSS.MUNI 310)	50 mm
Base Course: Granular A (OPSS.MUNI 1010) or 19 mm Crusher Run (OPSS.MUNI 1004)	100% SPMDD	150 mm
Subbase Course: Granular B Type I (OPSS.MUNI 1010)	98% SPMDD	650 mm (1)
Notes: (1) The recommended sub-base course thickness is intended to account for the weak sand fill layer and the presence of shallow groundwater. The additional sub-base thickness will provide additional support and drainage.		

It is recommended that geotechnical testing and inspections be carried out during construction operations to confirm construction is in accordance with the project specifications. Testing and inspections should include proof-rolling inspections on the subgrade prior to placing granular materials, compaction testing, monitoring of asphalt placement, etc.

The above pavement designs assume that the exposed subgrade has been adequately prepared. If localized soft areas are encountered, it may be necessary to sub-excavate and replace with additional granular fill. It is recommended that qualified geotechnical personnel be retained to complete an inspection of the subgrade and the placement of new granular during construction prior to placement of any hot-mix asphalt. Soft areas should be repaired by sub-excavating to a minimum depth of 300 mm and installing 300 mm Granular B Type I or III, compacted to 100% of the Specified Maximum Dry Density (SPMDD). If necessary, geogrid may be placed to strengthen soft soils.

The most severe loading conditions on paved areas and subgrades may occur during construction. Consequently, special provisions such as restricted drive lanes may be required, especially if construction is carried out during unfavourable weather conditions (i.e., cold weather or rain). If pavement construction occurs in wet, inclement weather, it may be necessary to provide additional subgrade support for construction traffic by increasing the thickness of the granular sub-base.

4.7.4 Transitions

Smooth transitions are required in all areas where new pavement structures meet existing facilities.

All longitudinal and transverse joints should meet the requirements of OPSS.MUNI 310. All longitudinal joints should be staggered between asphalt lifts. Staggering of the longitudinal joints should be made by putting the paving edge of the surface and the binder course at least 150 mm apart.

At the limits of paving, the existing pavement surface should be milled to the depth of the surface course layer, full width, to provide adequate thickness so the new asphalt material can be placed flush with the top of the existing pavement surface. The top lift of the new pavement surface should extend or "key into" a minimum of 5 m beyond the bottom lifts of the existing pavement structure. All milled surfaces should be cleaned thoroughly prior to the placement of a tack coat and new hot mix asphalt.

Transitions in between existing and new granular base and/or subbase where required should be completed at a minimum 10H:1V taper.

4.7.5 Subgrade Preparation

Fill materials were encountered in all the boreholes and extended down to depths ranging from approximately 1.5 to 2.3 mbgs. The existing fill materials may be considered for support of pavements for driveways and surface parking areas provided careful quality control and subgrade verification measures are employed during construction.

It is recommended that any subgrades comprising of existing fill be thoroughly proofrolled using a minimum 10-ton roller, inspected for obvious soft/loose areas and presence of deleterious materials. Should such areas be found, GHD can provide appropriate advice for subexcavation and replacement of the material and addressing localized weak areas at that time.

New fill may be required to replace unsuitable fill or to raise the grade. Any fill placed to increase or level the grade, must be compacted to a minimum 98 percent SPMDD in lifts not exceeding 200 mm. In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is also recommended. Geogrids can be used in the settlement sensitive areas for the granular layer stabilization. This will act as a composite due to interlock mechanism that develop between the aggregate and the stiff geogrid structure. This will also allow optimizing the overall pavement layer thickness provided in the following section.

The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of sub-base fills and restricted construction lanes may be required, especially if construction is carried out during wet weather conditions.

4.7.6 Drainage

Subsurface drainage (lateral and longitudinal) must be maintained in all areas of new construction.

The long-term performance of the proposed pavement structure for the driveways and surface car parking lot are highly dependent upon subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over-emphasized. It is recommended that the finished pavement surface be provided with a minimum cross fall of 2% and the subgrade should be sloped at 3% towards subdrains/stubdrains and catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

Subdrains should be installed along the perimeter of the driveway and parking areas. The invert of the subdrains should be at least 300 mm below the bottom of the subbase and should be sloped to drain to adjacent catch basins.

The subdrains should be installed in a 300 mm by 300 mm trench lined by suitable geotextile and consist of a 150 mm diameter perforated pipe wrapped in a suitable geotextile and surrounded with a minimum thickness of 50 mm of free draining sand such as clear stone or concrete sand.

Also, the pavement subgrade should be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward the edge of the pavement/subdrain and toward catch basins.

4.7.7 Frost Treatment

Frost treatment of the existing catch basins is strongly recommended to facilitate effective drainage of the storm structure as required, to intercept excess subsurface moisture and reduce the frost heaving actions.

The recommended frost treatment is to excavate 0.5 m length in all directions around catch basins and manhole structures to a minimum depth of 1.0 m. Subdrains (150 mm diameter perforated pipe) should be installed and wrapped around the structure. The excavation should be backfilled with Clear Stone and wrapped with geotextile before the placement of granular fill materials.

4.7.8 Maintenance Considerations

Systematic routine and preventative maintenance activities are recommended for all newly constructed, reconstructed, or rehabilitated pavements. Within 2 to 3 years of rehabilitation/reconstruction, crack routing and sealing will be needed. Patching localized areas with medium to high severity distresses (i.e., potholes, alligator cracking, etc.) will become critical as the pavements age. In several cases, regular, timely, and systematic maintenance may be able to prolong the life of an existing pavement for many years.

5. Construction Considerations

5.1 Site Preparation and Grading, Reuse of Existing Soils

At this time, the grading information is not known across the Site. Any topsoil, organic rich or deleterious material should be removed prior to site grading activities and such materials should not be used as backfill in settlement sensitive areas. The subgrade soils exposed after the removal of the unsuitable material should consist of approved earth fill or native soil. The subgrade should be visually inspected, compacted and proof rolled where required, using appropriate equipment compatible with the type of soil.

Where engineered fill is required to raise grades, suitable fill materials should be placed in thin layers (300 mm thick or less) and compacted to a minimum of 98 per cent SPMDD. In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is also recommended. Based on the characteristics encountered within the boreholes, the existing sand fill material on Site may be suitable for reuse as structural backfill or fill under settlement sensitive areas but will require further verification and testing prior to construction. The existing fill can be used as general fill in landscaped areas. If free-draining and non-frost susceptible materials are required as backfill against retaining walls, the use of fine-grained soils will not be appropriate and imported granular fills may be required, subject to further testing on the existing sand fill materials.

5.2 Site Servicing

Underground service lines can be founded on a prepared fill subgrade. The suitability of the soils to provide adequate support for buried services must be verified and confirmed on Site by qualified geotechnical personnel experienced in such work.

It is recommended that prior to commencing the construction of the Site servicing, consideration be given to the excavation of a series of trial test pit excavations along the alignment of the proposed sewers/watermains to determine more accurately the soil behavior and if any dewatering works are required.

The bedding for trenched services should consist of well graded materials meeting OPSS.MUNI requirements. The bedding and sand cover materials should be adequately compacted to provide support and protection to the service pipes. Provided the base area of the sewer pipes and watermains is free of all loose and deleterious materials, the pipe bedding should comply with the requirements of OPSD 802 series. Site servicing plans were not available at the time of this report and depending on the elevation of the utilities and subgrade material, additional bedding / base reinforcement may be required.

The bedding and cover material may consist of OPSS.MUNI 1010 Granular “A” material compacted to at least 95 per cent SPMDD. However, if some limited depths of standing water are present, High-Performance Bedding (HPB) and/or HL6 clear stone wrapped in geo-textile may be adopted as bedding material below the pipe to provide stabilization.

Backfilling of trenches can be accomplished by reusing the excavated soils or imported granular soil, provided the moisture content of the material is maintained within ±2 percent of optimum water content and the fill is free of topsoil, organics, and any deleterious materials. The fill placed in excavated trenches should be in loose lifts not exceeding 300 mm thick and compacted to not less than 95 per cent SPMDD. The top 1 m of the trench fil underlying settlement sensitive areas (such as road subgrades) should be compacted to at least 98 per cent SPMDD.

5.3 Open Cut Excavation

Excavations for the foundation slabs/pile caps and utility installation trenches are expected to extend through existing non-cohesive fill material and native non-cohesive deposits.

Where space permits, open cut excavations into these soil units must be carried out in accordance with the guidelines outlined in the current edition of the Occupational Health and Safety Act and Regulations (OHSA) for Construction Projects. All excavations should be carried out in the manner specified in Ontario Regulation 213/91 and the OHSA.

The existing non-cohesive fill material is classified as Type 4 soils. The existing loose to compact native non-cohesive soils are classified as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. The dense to very dense native non-cohesive soils below the water table are classified as Type 3 soils. Temporary excavations (i.e., those which are open for a relatively short time) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) in Type 3 soils and 3H:1V in Type 4 soils. If the excavation contains more than one type of soil, the excavation side slopes should be inclined based on the requirements for the soil with the highest number.

Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the depth of the open cut excavation. Excavation of the soils at this Site should be feasible using conventional excavation equipment.

5.4 Groundwater Control

Groundwater was encountered during drilling at a depth of 1.6 mbgs (Elevation 222.0 m) in BH6-23, and 2.3 mbgs (Elevation 221.4 m) in BH5-23. The inferred groundwater table depth from the CPT dissipation tests was in the range of 2 to 2.5 mbgs. Therefore, excavations for the utilities and foundations may extend below the groundwater level. Groundwater levels are typically high during snow melting and heavy rains in spring and summer. Therefore, depending on the time of construction and the depth of associated excavations, the groundwater level may rise and adversely affect construction.

Surface water and runoff should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the subgrade. Investigative test pits are recommended prior to construction for the contractor’s planning of groundwater control means and methods.

5.5 Temporary Excavation Support Systems/Shoring

Should the recommended excavation slopes be impractical in some areas of the Site, due to space limitations or risk of damaging existing structures or underground service utilities, engineered temporary shoring systems will be required. Temporary excavation support systems should be designed and constructed in accordance with OPSS.MUNI 539 (*Temporary Protection Systems*). The lateral movement of the protection system should meet Performance Level 2 as specified in OPSS.MUNI 539, provided that any utilities, if present can tolerate this magnitude of deformation. The selection and design of the temporary protection system will be the responsibility of the contractor.

It is anticipated that a driven interlocking sheet pile system may be constructable; Alternatively, a soldier pile and lagging system could be used at the Site.

The following information is provided to the designers to aid in assessment of the approximate construction costs. Passive toe restraint to the soldier piles may be determined using conventional passive earth pressure distribution acting over an equivalent width equal to three times the soldier pile socket diameter provided that the soldier piles are separated by more than three times the socket diameter. Design of the temporary protection system should include an evaluation of basal stability and hydraulic uplift stability as defined in the CFEM (2006).

The total passive resistance below the base of the excavation (i.e., adjacent to the temporary protection system) may be calculated based on the values of K_p indicated below in Table 5.1. The earth pressure coefficients given in Table 5.1 assume that the ground surface behind the shoring system is horizontal. If the retained ground is sloping, the lateral earth pressure coefficients must be adjusted accordingly to account for the slope.

Table 5.1 Lateral Earth Pressure Coefficients – Static Condition

Soil Type	Bulk Density γ (kN/m³)	Angle of Internal Friction Φ (°)	Undrained Shear Strength S_u (kPa)	Lateral Earth Pressure Coefficients		
				K_a (active)	K_0 (at-rest)	K_p (passive)
Very Loose to Compact Non-Cohesive Fill Material	18	30	-	0.33	0.50	3.00
Compact to Very Dense Native sand and silt	19	33	-	0.29	0.46	3.39
Stiff to Very Stiff Silty Clay Deposit	19	30	125	0.33	0.50	3.00
Imported Fill Material (OPSS.MUNI 1010 Granular A)	22	35	-	0.27	0.43	3.69
Imported Fill Material (OPSS.MUNI 1010 Granular B Type II)	21	35	-	0.27	0.43	3.69

5.6 Vibration Monitoring

Residential/commercial buildings are present in the vicinity of the Site. A PPV threshold of 25 mm/s is generally considered applicable for buildings considering the typical range of frequencies of construction equipment. While it is expected that vibration levels will not reach these thresholds at the structures, it would be prudent to complete pre-and post-construction surveys and vibration monitoring at or near the structures, to defend against potential damage claims associated with vibration-inducing activities.

5.7 Construction Monitoring

The foundation installations and any engineered fill placement must be closely monitored and inspected by qualified personnel to ensure consistency with the design recommendations. The on-site review of the bearing conditions as the foundations are constructed is an integral part of geotechnical design.

Qualified geotechnical personnel should inspect and test all stages of the proposed development. Specifically, they should ensure that the materials and conditions comply with the geotechnical assessment report. In addition, qualified geotechnical personnel should provide material testing services prior to and during backfilling and grade raising operation. Should soil conditions be encountered that vary from those described in this report and any subsequent studies, our office should be informed immediately such that the proper measures are undertaken.

In addition to the above, it is also recommended to establish project specific monitoring, including monitoring of any shoring systems and adjacent structures, if applicable, during construction.

6. Limitations of the Investigation

This report is intended solely for Crown (Bradford) Developments Inc. and their designers and is prohibited for use by others without GHD's prior written consent. This report is considered GHD's professional work product and shall remain the sole property of GHD. As per client request and previous reports, no portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current Site use, ground surface elevation and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our preliminary subsurface investigation and resulting understanding of the project, as defined at the time of the study. Additional geotechnical investigations will be required prior to detailed design and we should subsequently be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, GHD is the geotechnical engineer of record. It is recommended that GHD be retained for supplementary geotechnical investigations and during construction of all foundations and during earthwork operations to confirm the conditions of the soils are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings of the geotechnical report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a Site and the comments included in this report are based on the results obtained at the test locations only. The subsurface conditions confirmed at the test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on Site (e.g., excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the Site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

7. Closure

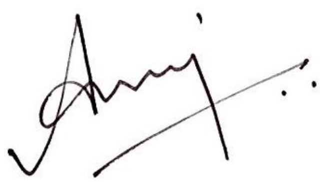
The fieldwork for this geotechnical investigation was completed by Brice Zanne, EIT and Matt Yee, EIT (junior engineer-in-training with GHD). This final report was prepared by Kateryna Pidriiko, M.Sc., P.Eng. and Anuj Choudhari, M.Sc., P.Eng., P.E. (intermediate geotechnical engineers with GHD).

Mrinmoy Kanungo, M.E.Sc., P.Eng., a Senior Geotechnical Engineer with GHD, conducted an independent review of the report. We trust that the above is satisfactory for your present requirements. Please contact us if you have any questions.

Respectfully submitted,

GHD

Kateryna Pidriiko, P.Eng., M.Sc.
Intermediate Geotechnical Engineer



Anuj Choudhari, P.Eng., P.E., M.Sc.
Intermediate Geotechnical Engineer

Mrinmoy Kanungo, M.E.Sc., P.Eng.
Senior Geotechnical Engineer

Figures

LEGEND

- BOREHOLE LOCATION
- CPT LOCATION
- MONITORING WELL LOCATION (GHD, NOVEMBER 2022)

0 5 10 15m

1:500

Coordinate System:
NAD 83 UTM ZONE 17
METRES

N

GHD

CROWN (BRADFORD) DEVELOPMENTS INC.
126 AND 140 BRADFORD STREET,
BARRIE, ONTARIO

BOREHOLE AND CPT LOCATION PLAN

Project No. 12593831
Date December 2023

FIGURE 1

Filename: \\ghdnet\ghd\CA\Mississauga\Projects\662\12593831\Digital_Design\ACAD\Figures\RPT001-Geotech\12593831-GHD-00-00-RPT-EN-D101_WA-001-Geotech.dwg
Plot Date: 04 December 2023 10:04 AM

Data Source: Image ©2022 Google (Imagery Date 7/24/2019).

Appendices

Appendix A

Borehole Records by GHD

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey soils is measured by the value of undrained shear strength (Cu).

Classification (Unified system)			
Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	>300 mm		

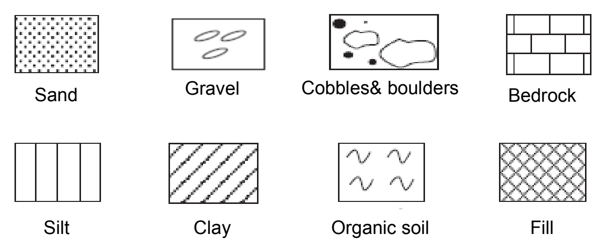
Terminology	
"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

Relative density of granular soils	**Standard penetration index "N" value** (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	**Undrained shear strength (Cu)** (P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation "RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND



Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
ST: Shelby tube
AG: Auger
SSE, GSE, AGE: Environmental sampling
PS: Piston sample (Osterberg)
RC: Rock core
GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
N_c: Dynamic cone penetration index
k: Permeability
R: Refusal to penetration
Cu: Undrained shear strength
ABS: Absorption (Packer test)
Pr: Pressure meter

LABORATORY TESTS:

I_p: Plasticity index
H: Hydrometer analysis
A: Atterberg limits
C: Consolidation
O.V.: Organic vapor
W_l: Liquid limit
GSA: Grain size analysis
w: Water content
CS: Swedish fall cone
Wp: Plastic limit
γ: Unit weight
CHEM: Chemical analysis

REFERENCE No.: 12593831

GHD

BOREHOLE No.: BH5-23
ELEVATION: 223.7 m

BOREHOLE REPORT

CLIENT: Crown (Bradford) Developments Inc.
PROJECT: Geotechnical Investigation
LOCATION: 126 and 140 Bradford Street, Barrie, Ontario
DESCRIBED BY: Matt Yee CHECKED BY: Anuj Choudhari
DATE (START): 21 February 2023 DATE (FINISH): 22 February 2023

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- ▼ - WATER LEVEL (MEASURED)
- ∇ - WATER LEVEL (OBSERVED)

NORTHING: 4914988.3 EASTING: 604036.4 DRILLING TYPE: CME 75 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Washbore

Plot legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △ Number refer to Sensitivity; ○ Water content (%); Atterberg limits (%) (W_p, W_l); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm)

COMMENTS: PIEZOMETER/ STANDPIPE INSTALLATION

Depth (m)	Elevation (m)	Description of Soil
	223.7	GROUND SURFACE
0.1	223.7	ASPHALT (51 mm)
0.1	223.6	GRAVEL (51 mm)
		FILL: SAND, trace silt, trace gravel, brown, moist, compact to very loose
2.3	221.4	NATIVE: SAND, trace silt, grey, wet, very loose (hollow stem auger to 3.1 m bgs before switching to washbore)
4.6	219.1	SAND and SILT, trace clay, brown to grey, wet, compact to loose

Sample Type and Number	Gravel/Sand/Silt/Clay (%)	Moisture Content (%)	Recovery/TCR (%)	Blows per/15cm/RQD(%)	'N' Value SCR(%)
SS1		7	83.3	4-5-5-5	10
SS2		5	75	1-1-1-1	2
SS3		10	75	0-0-0-1	0
SS4		29	66.7	0-0-1-1	1
SS5		26	83.3	2-2-2-3	4
SS6	0-63-35-2	23	100	5-5-8-8	13
SS7		--	66.7	6-8-10-11	18
SS8		20	75	4-4-4-4	8
SS9	0-60-38-2 Non-Plastic	21	83.3	4-5-5-5	10

Water level observed: 2/21/2023 (approx. 2.4 m)

REFERENCE No.: 12593831

BOREHOLE REPORT

Page 2 of 3

BOREHOLE No.: BH5-23
ELEVATION: 223.7 m

CLIENT: Crown (Bradford) Developments Inc.
PROJECT: Geotechnical Investigation
LOCATION: 126 and 140 Bradford Street, Barrie, Ontario
DESCRIBED BY: Matt Yee CHECKED BY: Anuj Choudhari
DATE (START): 21 February 2023 DATE (FINISH): 22 February 2023

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- ▼ - WATER LEVEL (MEASURED)
- ▽ - WATER LEVEL (OBSERVED)

NORTHING: 4914988.3 EASTING: 604036.4 DRILLING TYPE: CME 75 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Washbore

Plot legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △ Number refer to Sensitivity; ○ Water content (%); W_p W_l Atterberg limits (%); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm). Scale: 10 20 30 40 50 60 70 80 90

COMMENTS: PIEZOMETER/STANDPIPE INSTALLATION

Depth (m)	Elevation (m)	Description of Soil	Type and Number	Comments (Gravel-Sand-Silt-Clay %)	Unit Weight (Dry) (KN/m3)	Moisture Content (%)	Recovery/TCR (%)	Blows per/15cm/RQD(%)	'N' Value SCR(%)
	223.7	GROUND SURFACE							
		SAND and SILT, trace clay, brown to grey, wet, compact to loose							
11.0			SS10			23	83.3	6-5-6-7	11
12.2	211.5	SILT, some sand to sandy, trace clay, grey, wet, compact to very dense	SS11			25	100	8-7-7-12	14
14.0			SS12	0-18-74-8 Non-Plastic		23	100	5-6-8-14	14
			SS13			17	91.7	10-16-17-17	33
			SS14			16	83.3	14-22-22-25	44
		contains sand seams	SS15			15	75	12-21-40-44	61

Depth scale: Feet 33–65; Metres 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0

REFERENCE No.: 12593831

GHD

BOREHOLE No.: BH5-23

ELEVATION: 223.7 m

BOREHOLE REPORT

CLIENT: Crown (Bradford) Developments Inc.

PROJECT: Geotechnical Investigation

LOCATION: 126 and 140 Bradford Street, Barrie, Ontario

DESCRIBED BY: Matt Yee   CHECKED BY: Anuj Choudhari

DATE (START): 21 February 2023   DATE (FINISH): 22 February 2023

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- ▼ - WATER LEVEL (MEASURED)
- ▽ - WATER LEVEL (OBSERVED)

NORTHING: 4914988.3 EASTING: 604036.4 DRILLING TYPE: CME 75 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Washbore

Chart legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △³ Number refer to Sensitivity; ○ Water content (%); W_p W_l Atterberg limits (%); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm)

COMMENTS: PIEZOMETER/ STANDPIPE INSTALLATION

Depth (Feet)	Depth (Metres)	Elevation (m)	Description of Soil	Samples: Type and Number	Lab Testing: Comments / Gravel Sand Silt Clay (%)	Unit Weight (Dry) (KN/m3)	Moisture Content (%)	Recovery/ TCR(%)	Blows per/15cm/ RQD(%)	'N' Value SCR(%)
		223.7	GROUND SURFACE							
66			SILT, some sand to sandy, trace clay, grey, wet, compact to very dense	SS16	0-22-72-6 Non-Plastic		14	66.7	21-33-40-41	73
71				SS17			16	83.3	17-21-25-23	46
76				SS18			16	91.7	16-20-21-22	41
81				SS19	0-15-78-7 Non-Plastic		16	83.3	7-13-25-18	38
85.5				SS20			15	100	35-50/127 mm	50/127 mm
91				SS21	Non-Plastic		18	100	10-10-12-12	22
92	28.0	195.7	END OF BOREHOLE							

NOTES:

- End of Borehole at 28.0 m bgs.
- Groundwater was encountered at 2.3 m bgs during drilling.
- bgs denotes 'below ground surface'.

Depth scale: 66–98 feet; 21.0–29.0 metres. Chart scale: 10 20 30 40 50 60 70 80 90

REFERENCE No.: 12593831

BOREHOLE REPORT

BOREHOLE No.: BH6-23

ELEVATION: 223.6 m

CLIENT: Crown (Bradford) Developments Inc.

PROJECT: Geotechnical Investigation

LOCATION: 126 and 140 Bradford Street, Barrie, Ontario

DESCRIBED BY: Brice Zanne   CHECKED BY: Anuj Choudhari

DATE (START): 21 February 2023   DATE (FINISH): 22 February 2023

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- ▼ - WATER LEVEL (MEASURED)
- ▽ - WATER LEVEL (OBSERVED)

NORTHING: 4914936.6 EASTING: 604037.2 DRILLING TYPE: CME 55 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Mud Rotary

Graph legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △ Number refer to Sensitivity; ○ Water content (%); W_p–W_l Atterberg limits (%); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm). Scale: 10 20 30 40 50 60 70 80 90

COMMENTS: PIEZOMETER/ STANDPIPE INSTALLATION

Depth (Metres)	Elevation (m)	Description of Soil	Sample Type and Number	Gravel-Sand-Silt-Clay (%)	Unit Weight (Dry) (KN/m3)	Moisture Content (%)	Recovery/TCR (%)	Blows per/15cm/ RQD(%)	'N' Value SCR(%)
0	223.6	GROUND SURFACE							
0.1	223.5	ASPHALT (102 mm)							
		FILL: SAND, trace gravel, trace silt, trace clay, brown to black, moist, loose	SS1			15	80	7-3-5-2	8
			SS2			15	12.5	4-4-2-4	6
			SS3	5-88-7-0		23	58.3	2-3-5-6	8
2.3	221.3	NATIVE: SILTY SAND, trace clay, trace gravel, brown, wet, loose	SS4			30	60	3-4-4-5	8
		(hollow stem auger to 3.1 mbgs before switching to mud rotary)	SS5			22	100	1-2-4-6	6
			SS6			22	50	6-4-4-4	8
			SS7	1-65-31-3		23	41.7	0-5-5-5	10
7.6	216.0	SANDY SILT, trace clay, grey, wet, loose to very dense	SS8			25	58.3	0-4-4-3	8
			SS9			--	50	2-3-2-2	5

Water level (observed) 2/21/2023 at approx. 5 ft (1.5 m).

REFERENCE No.: 12593831

BOREHOLE REPORT

BOREHOLE No.: BH6-23

ELEVATION: 223.6 m

CLIENT: Crown (Bradford) Developments Inc.

PROJECT: Geotechnical Investigation

LOCATION: 126 and 140 Bradford Street, Barrie, Ontario

DESCRIBED BY: Brice Zanne CHECKED BY: Anuj Choudhari

DATE (START): 21 February 2023 DATE (FINISH): 22 February 2023

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- ▼ - WATER LEVEL (MEASURED)
- ▽ - WATER LEVEL (OBSERVED)

NORTHING: 4914936.6 EASTING: 604037.2 DRILLING TYPE: CME 55 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Mud Rotary

Chart legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △³ Number refer to Sensitivity; ○ Water content (%); W_p W_l Atterberg limits (%); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm)

COMMENTS: PIEZOMETER/ STANDPIPE INSTALLATION

Depth (Feet)	Depth (Metres)	Elevation (m)	DESCRIPTION OF SOIL	Samples: Type and Number	Lab Testing: Comments (Gravel-Sand-Silt-Clay %)	Unit Weight (Dry) (KN/m3)	Moisture Content (%)	Recovery/TCR(%)	Blows per/15cm/ RQD(%)	'N' Value SCR(%)
		223.6	GROUND SURFACE							
33			SANDY SILT, trace clay, grey, wet, loose to very dense							
36	11.0			SS10			24	83.3	6-4-3-2	7
41				SS11	0-22-68-10 Non-Plastic		22	100	3-4-4-3	8
45	13.7	209.9	(compact to very dense below)							
46	14.0			SS12			20	66.7	0-8-8-8	16
56				SS13			18	54.2	8-10-12-11	22
62–63				PMT1			--	--	--	--
64			PMT TEST at 19.3 m bgs							
65				SS14			18	100	50/114mm	50/114

REFERENCE No.: 12593831

BOREHOLE No.: BH6-23

ELEVATION: 223.6 m

BOREHOLE REPORT

CLIENT: Crown (Bradford) Developments Inc.

PROJECT: Geotechnical Investigation

LOCATION: 126 and 140 Bradford Street, Barrie, Ontario

DESCRIBED BY: Brice Zanne CHECKED BY: Anuj Choudhari

DATE (START): 21 February 2023 DATE (FINISH): 22 February 2023

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- VA - VANE SHEAR
- AU - AUGER PROBE
- WATER LEVEL (MEASURED)
- WATER LEVEL (OBSERVED)

NORTHING: 4914936.6 EASTING: 604037.2 DRILLING TYPE: CME 55 Truck Rig DRILLING METHOD: Hollow Stem Auger (108 mm I.D.)/Mud Rotary

Chart legend: △ Undisturbed Vane Value (kPa); □ Remoulded Field Vane Value (kPa); △ Number refer to Sensitivity; ○ Water content (%); Atterberg limits (%) (W_p, W_l); ● "N" Value (blows/12 in.-30 cm); ★ "DCPT" Value (blows/12 in.-30 cm)

Depth (m)	Elevation (m)	Description of Soil	Sample Type and Number	Comments (Gravel Sand Silt Clay %)	Moisture Content (%)	Recovery/TCR (%)	Blows per/15cm/ RQD(%)	'N' Value SCR(%)	Comments
	223.6	GROUND SURFACE						mm	
		SANDY SILT, trace clay, grey, wet, loose to very dense							
			PMT2		--	--	--	--	
22.9	200.8	PMT TEST at 22.7 m bgs							
		SILTY CLAY, grey, very moist, firm to very stiff	SS15	1-0-57-42	29	100	0-3-4-14	7	Wp 20, Wl 37
		VANE TEST at 23.5 m bgs Undisturbed Value >150 kPa (maxed out) Remolded Value = N/A			--	--	--	--	>100 kPa
			PMT3		--	--	--	--	
		PMT TEST at 25.5 m bgs							
25.9	197.7	SILT, trace sand, trace clay, grey, wet, very dense	SS16		18	83.3	50-26-31-42	57	
			SS17	Non-Plastic	19	100	44-55-32-30	87	
28.0	195.6	END OF BOREHOLE							

NOTES:

- End of Borehole at 28.0 m bgs.
- Groundwater was encountered at 1.6 m bgs during drilling.
- bgs denotes 'below ground surface'.

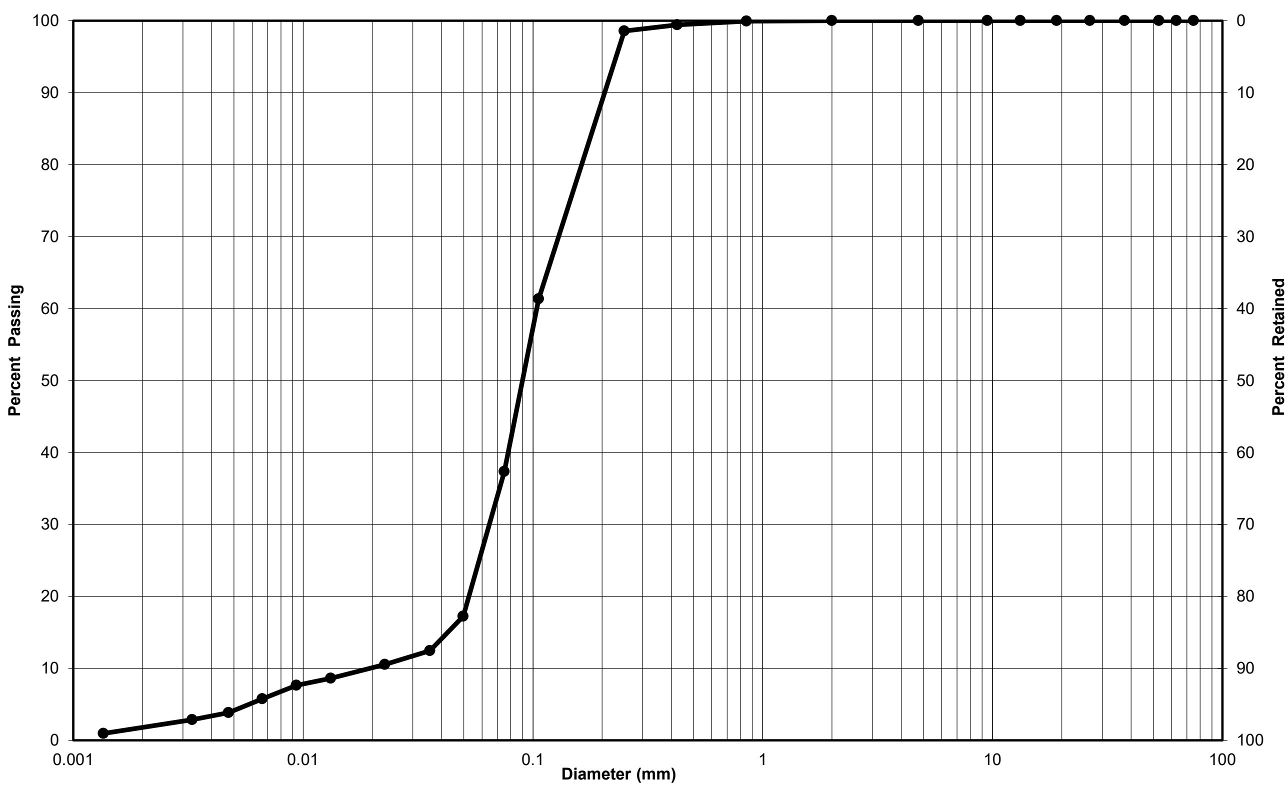
Appendix B

Laboratory Test Results

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-1
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001

Borehole No.:	BH5-23	Sample No.:	SS6
Depth:	15.0 ft - 17.0 ft (4.57 m - 5.18 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

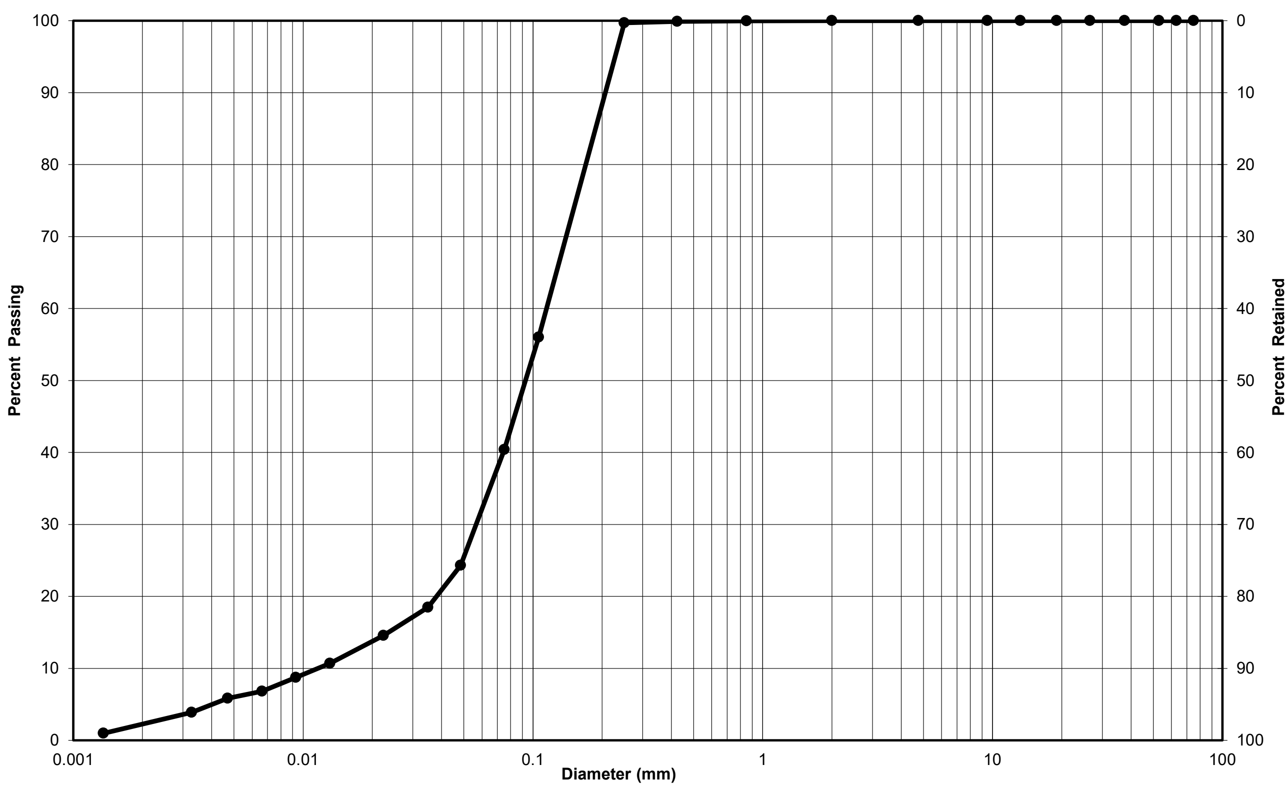
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sand and Silt, trace clay	0	63	37
Clay-size particles (<0.002 mm):			2 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-2
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH5-23	Sample No.:	SS9
Depth:	30.0 ft - 32.0 ft (9.15 m - 9.76 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

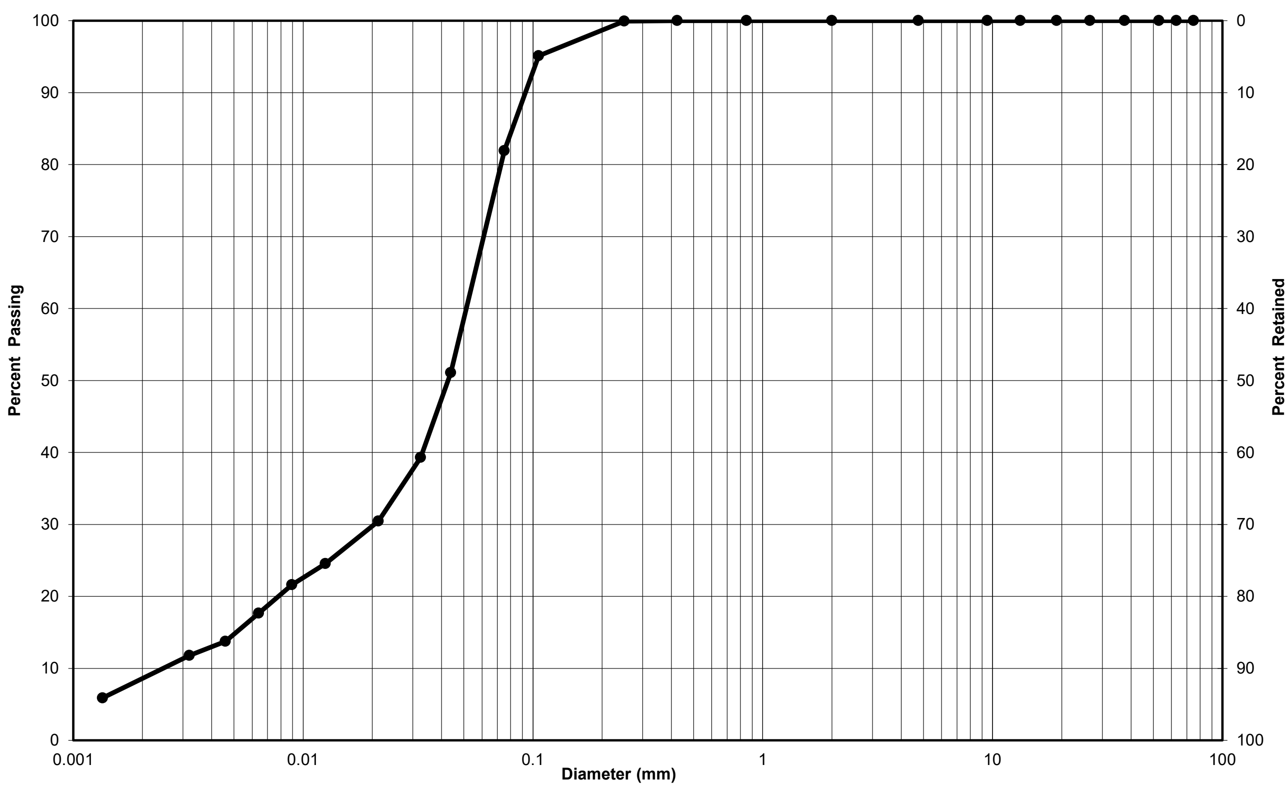
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sand and Silt, trace clay	0	60	40
Clay-size particles (<0.002 mm):			2 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-3
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH5-23	Sample No.:	SS12
Depth:	45.0 ft - 47.0 ft (13.72 m - 14.33 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

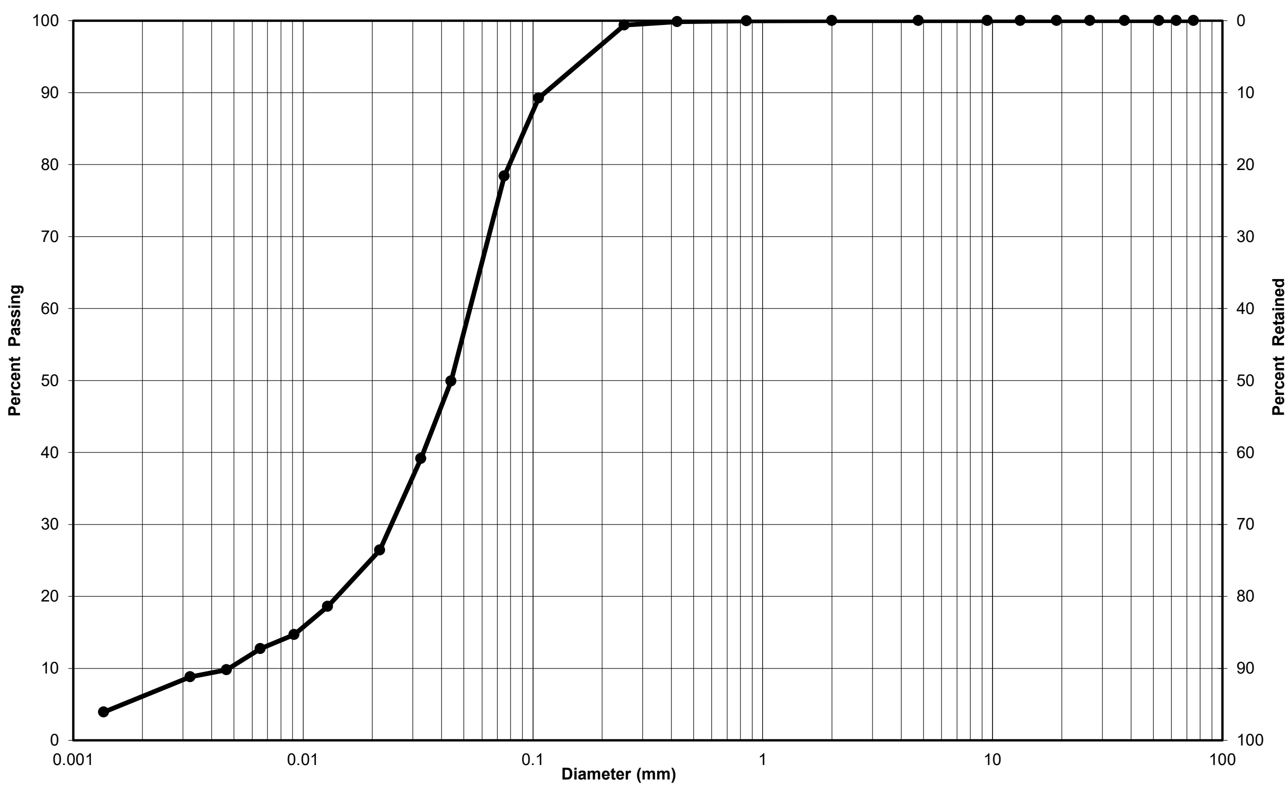
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sand, some silt, trace clay	0	18	82
Clay-size particles (<0.002 mm):			8 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-4
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH5-23	Sample No.:	SS16
Depth:	65.0 ft - 67.0 ft (19.82 m - 20.43 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

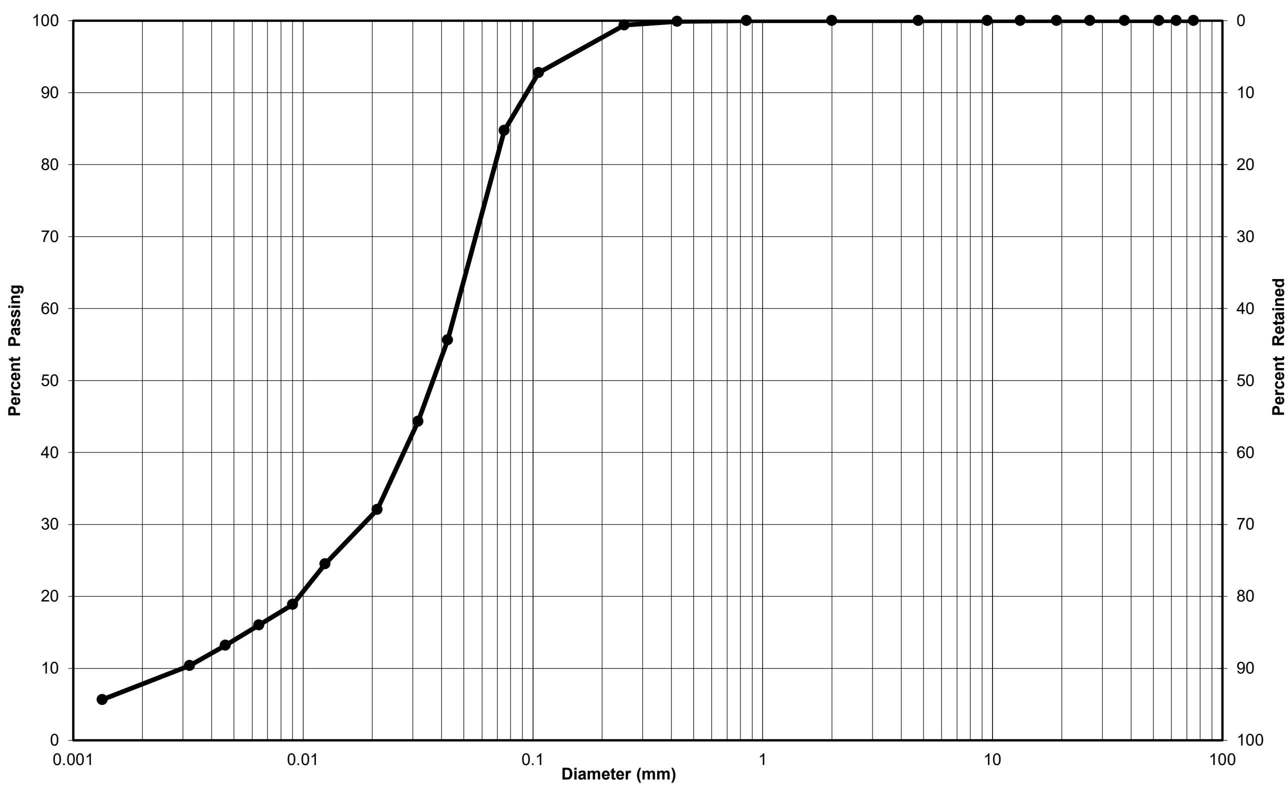
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
sandy Silt, trace clay	0	22	78
Clay-size particles (<0.002 mm):			6 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-5
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH5-23	Sample No.:	SS19
Depth:	80.0 ft - 82.0 ft (24.39 m - 25.00 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

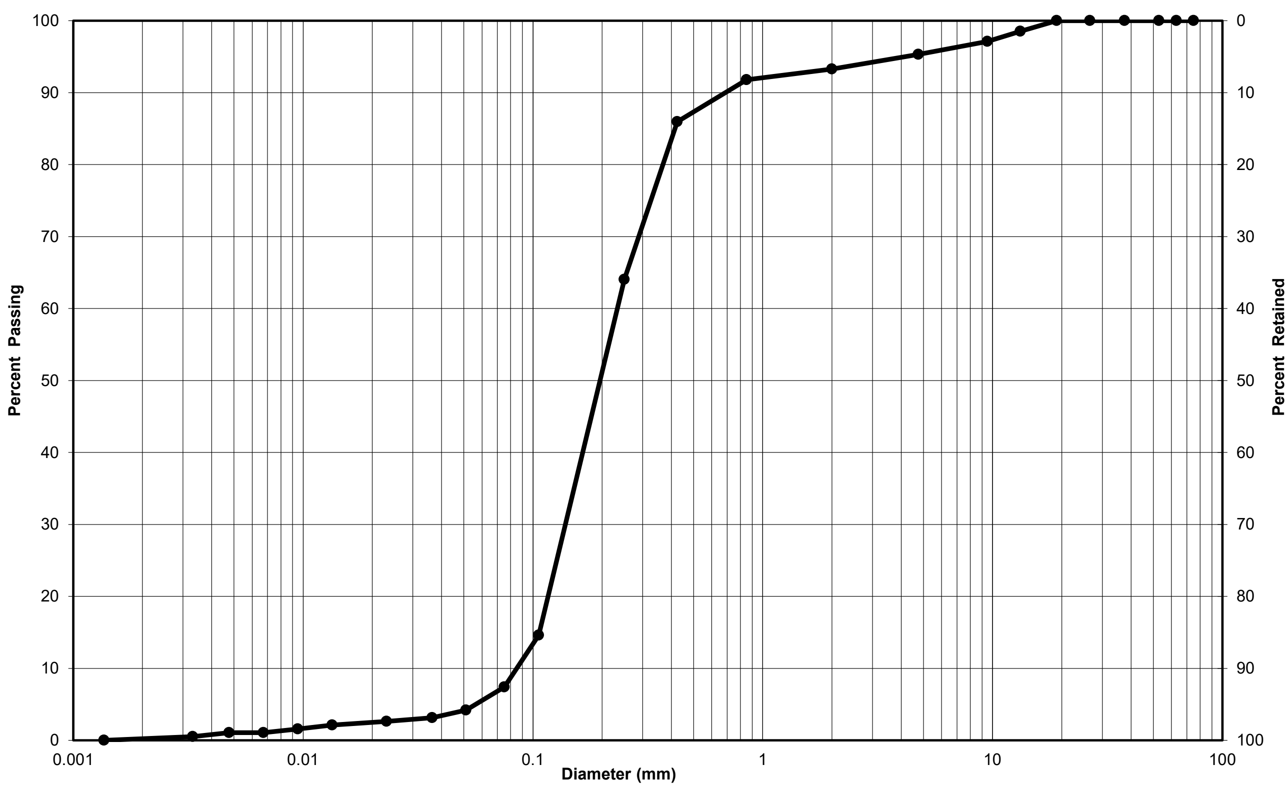
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silt, some sand, trace clay	0	15	85
Clay-size particles (<0.002 mm):			7 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-7
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH6-23	Sample No.:	SS3
Depth:	5.0 ft - 7.0 ft (1.52 m - 2.135 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

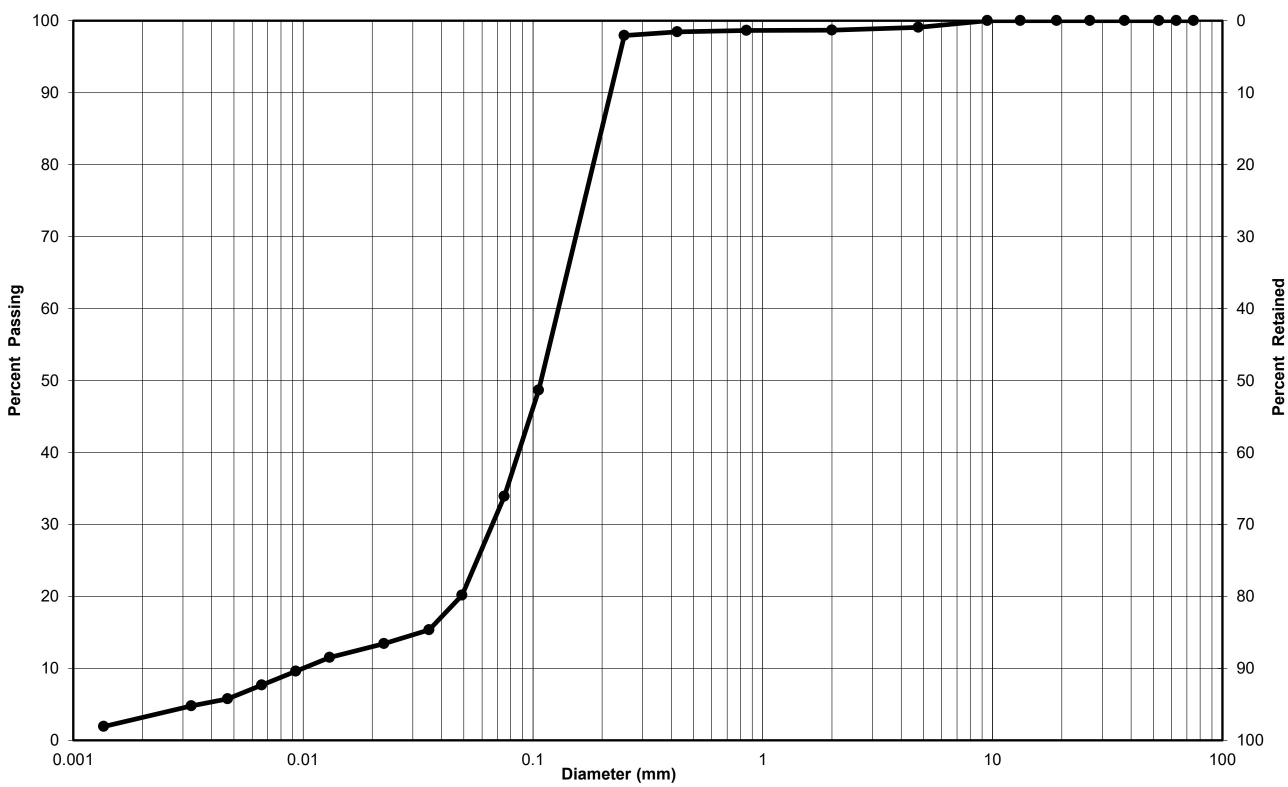
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sand, trace silt, trace gravel	5	88	7
			0 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-8
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH6-23	Sample No.:	SS7
Depth:	20.0 ft - 22.0 ft (6.10 m - 6.71 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

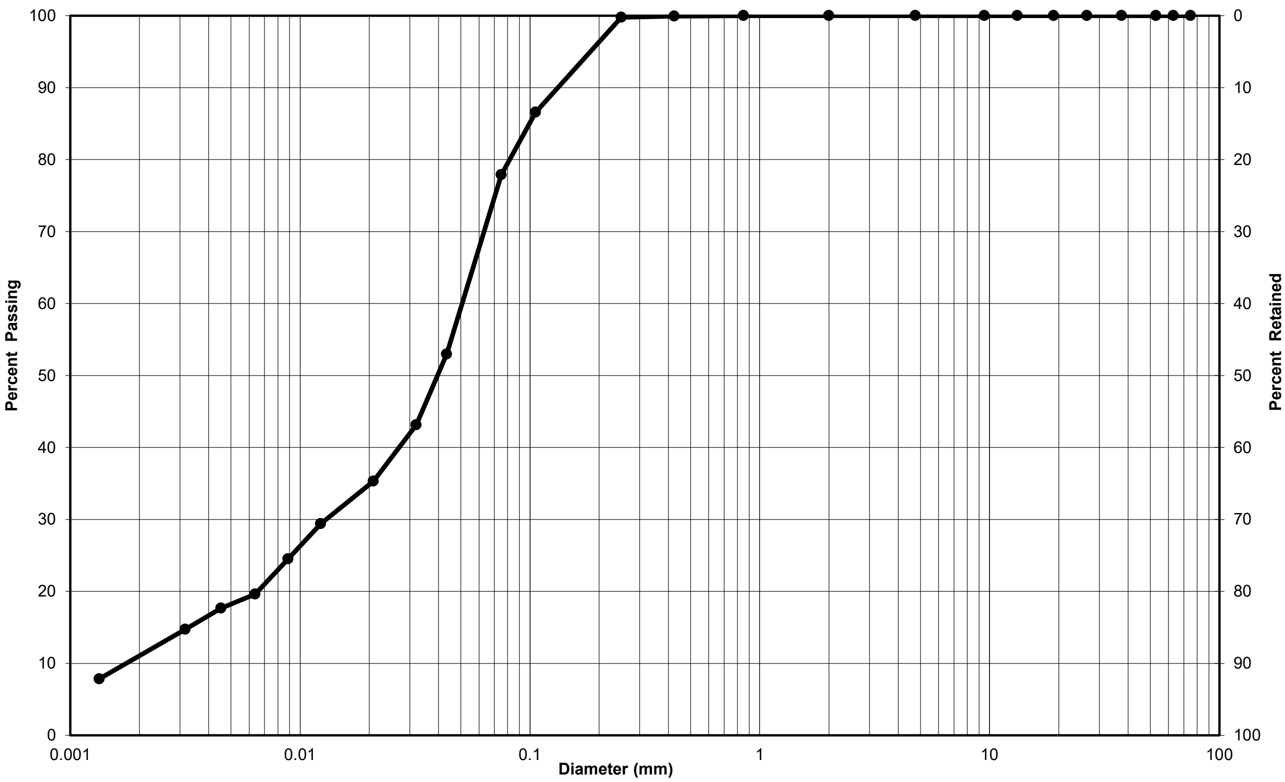
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
silty Sand, trace clay, trace gravel	1	65	34
Clay-size particles (<0.002 mm):			3 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-9
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001
Borehole No.:	BH6-23	Sample No.:	SS11
Depth:	40.0 ft - 42.0 ft (12.20 m - 12.80 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
sandy Silt, trace clay	0	22	78
Clay-size particles (<0.002 mm):			10 %

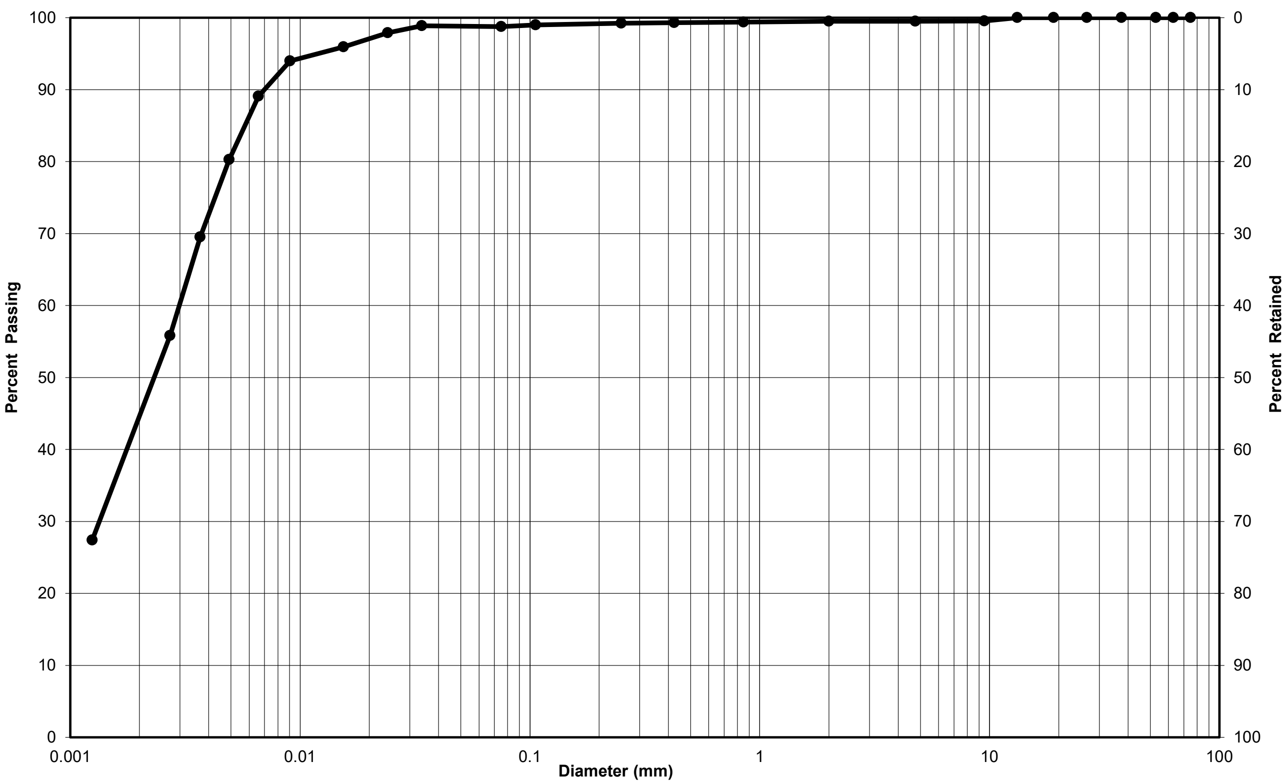
Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

Particle-Size Analysis of Soils
MTO LS-702 (Geotechnical)

Client:	Crown (Bradford) Developments Inc.	**Lab No.:**	WLA 1321-10
Project, Site:	Geotechnical Investigation 126 and 140 Bradford Street, Barrie, ON	**Project No.:**	12593831.04.001

Borehole No.:	BH6-23	Sample No.:	SS15
Depth:	75.0 ft - 77.0 ft (22.87 m - 23.48 m)	Enclosure:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silty Clay, trace gravel	1	0	99
Clay-size particles (<0.002 mm):			42 %

Remarks:

Performed by:	Mathew Russell / Jadon Manson-Hennig	**Date:**	March 1 - 6,2023
Verified by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023

GHD

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-2
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH5-23	Sample no.:	SS9	Sample Depth:	30.0 ft - 32.0 ft (9.15 m - 9.76 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Bean
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

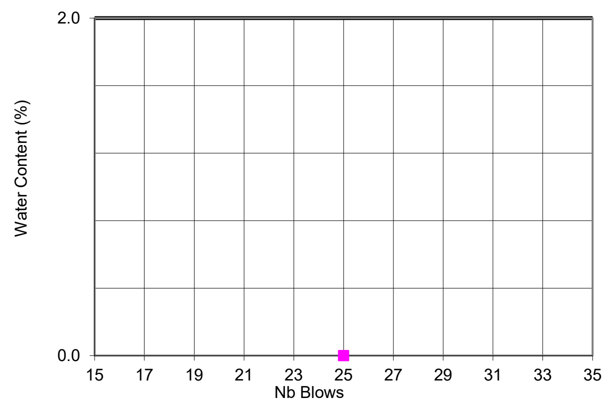
Natural Water Content (W^n):

Tare no.	XC26	
Wet soil+tare, g	84.70	
Dry soil+tare, g	70.50	
Mass of water, g	14.20	
Tare, g	4.00	
Mass of soil, g	66.50	
Water content %	21.4%	

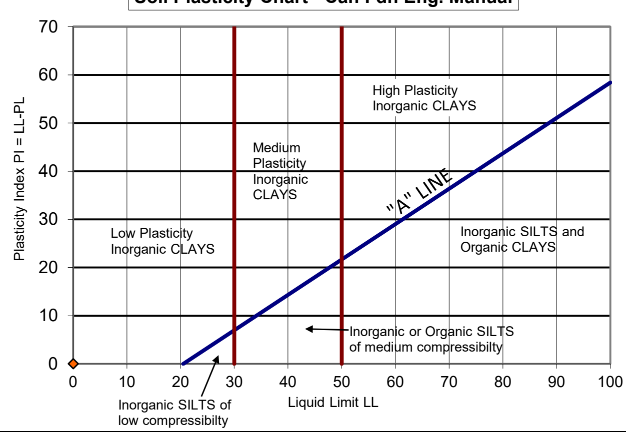
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results



Soil Plasticity Chart - Can Fdn Eng. Manual



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			21

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-3
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH5-23	Sample no.:	SS12	Sample Depth:	45.0 ft - 47.0 ft (13.72 m - 14.33 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Gr
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

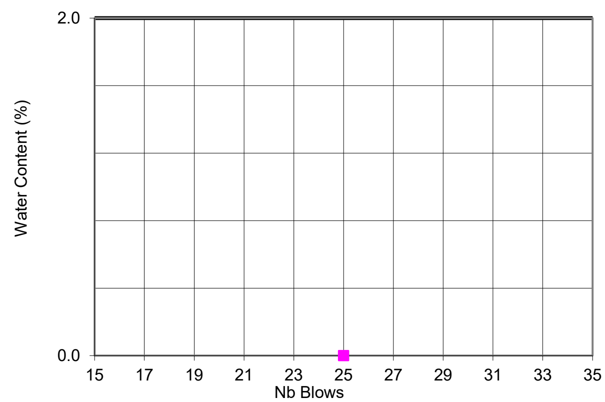
Natural Water Content (W^n):

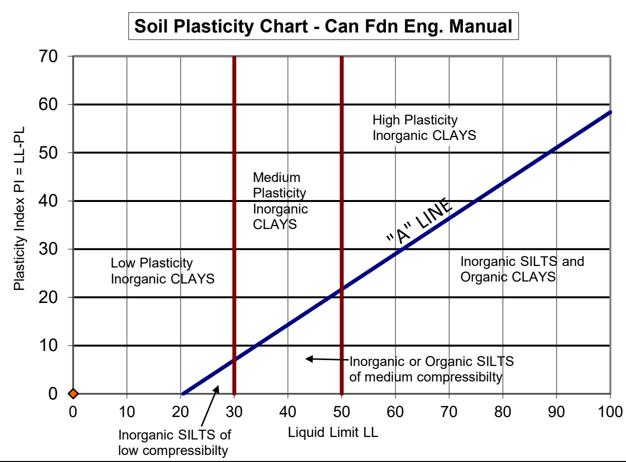
Tare no.	M57	
Wet soil+tare, g	78.80	
Dry soil+tare, g	64.80	
Mass of water, g	14.00	
Tare, g	4.70	
Mass of soil, g	60.10	
Water content %	23.3%	

Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			23

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-4
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH5-23	Sample no.:	SS16	Sample Depth:	65.0 ft - 67.0 ft (19.82 m - 20.43 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Ch
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

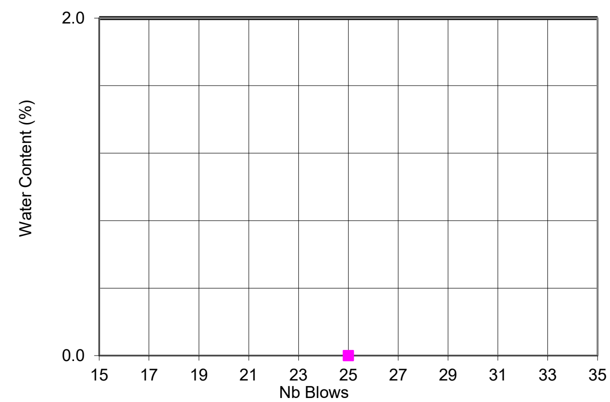
Natural Water Content (W^n):

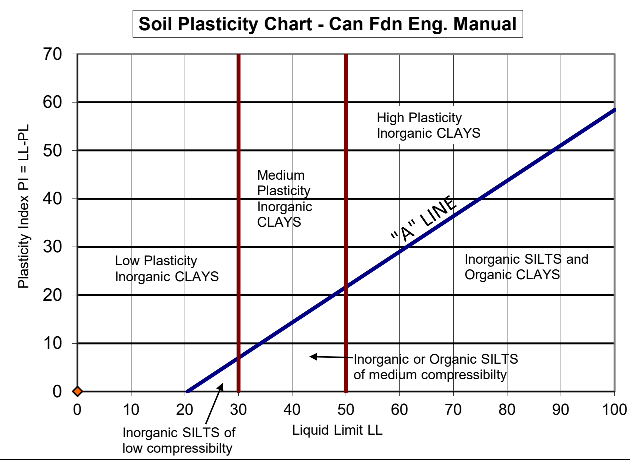
Tare no.	XC12	
Wet soil+tare, g	74.60	
Dry soil+tare, g	66.10	
Mass of water, g	8.50	
Tare, g	4.00	
Mass of soil, g	62.10	
Water content %	13.7%	

Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			14

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-5
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH5-23	Sample no.:	SS19	Sample Depth:	80.0 ft - 82.0 ft (24.39 m - 25.00 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Tr
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

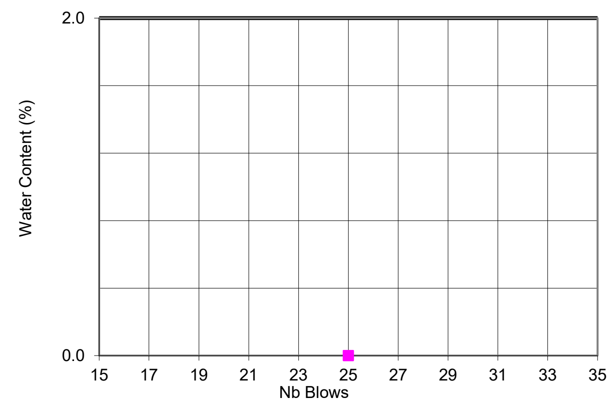
Natural Water Content (W^n):

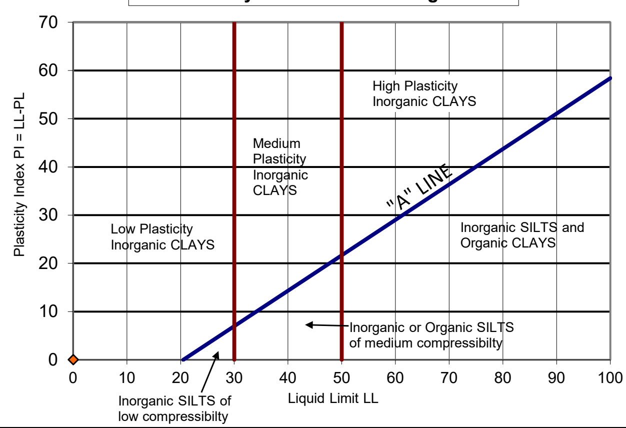
Tare no.	Z11	
Wet soil+tare, g	70.80	
Dry soil+tare, g	61.90	
Mass of water, g	8.90	
Tare, g	4.70	
Mass of soil, g	57.20	
Water content %	15.6%	

Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			16

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

GHD

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-6
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH5-23	Sample no.:	SS21	Sample Depth:	90.0 ft - 92.0 ft (27.44 m - 28.05 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Ic
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

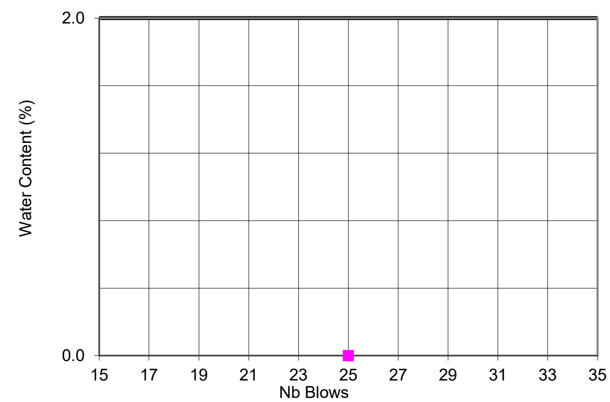
Natural Water Content (W^n):

Tare no.	V68	
Wet soil+tare, g	92.60	
Dry soil+tare, g	79.10	
Mass of water, g	13.50	
Tare, g	4.80	
Mass of soil, g	74.30	
Water content %	18.2%	

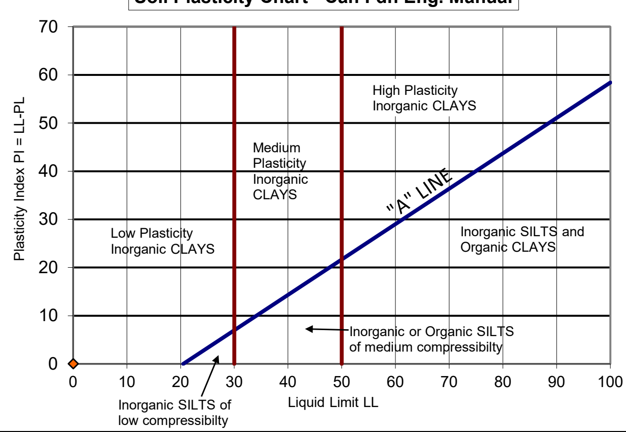
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results



Soil Plasticity Chart - Can Fdn Eng. Manual



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			18

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-9
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH6-23	Sample no.:	SS11	Sample Depth:	40.0 ft - 42.0 ft (12.20 m - 12.80 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	B3
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

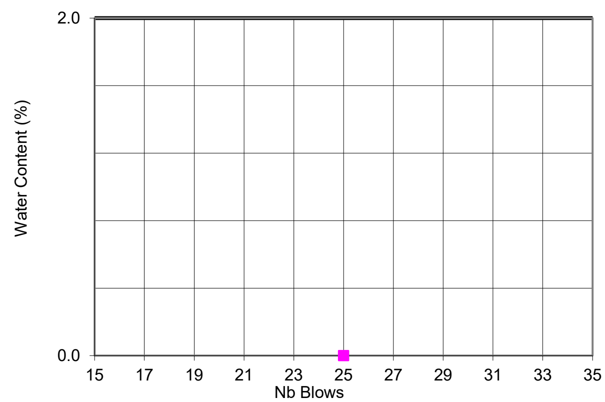
Natural Water Content (W^n):

Tare no.	UW6	
Wet soil+tare, g	92.00	
Dry soil+tare, g	76.00	
Mass of water, g	16.00	
Tare, g	4.00	
Mass of soil, g	72.00	
Water content %	22.2%	

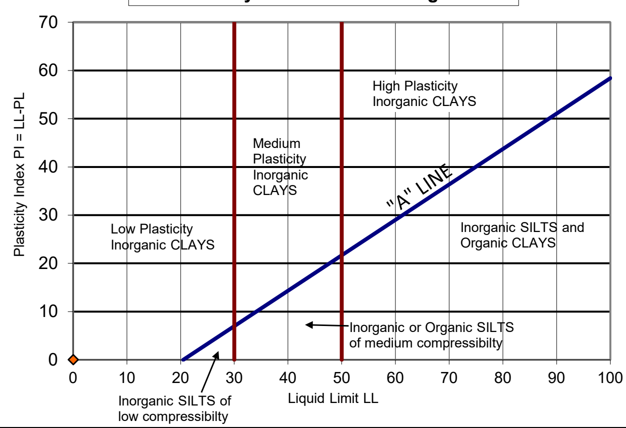
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results



Soil Plasticity Chart - Can Fdn Eng. Manual



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			22

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-10
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH6-23	Sample no.:	SS15	Sample Depth:	75.0 ft - 77.0 ft (22.87 m - 23.48 m)
Soil Description:	Medium Plasticity Inorganic Clay			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Ram
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	29	23	18
Water Content:			
Tare no.	Z11	Z38	Z17
Wet soil+tare, g	28.44	27.43	28.53
Dry soil+tare, g	26.61	25.81	26.56
Mass of water, g	1.83	1.62	1.97
Tare, g	21.54	21.35	21.47
Mass of soil, g	5.07	4.46	5.09
Water content %	36.1%	36.3%	38.7%

Plastic Limit (PL) - Water Content:

Tare no.	Q2	Z7
Wet soil+tare, g	24.07	31.34
Dry soil+tare, g	22.55	29.71
Mass of water, g	1.52	1.63
Tare, g	14.88	21.42
Mass of soil, g	7.67	8.29
Water content %	19.8%	19.7%
Average water content %	19.7%	

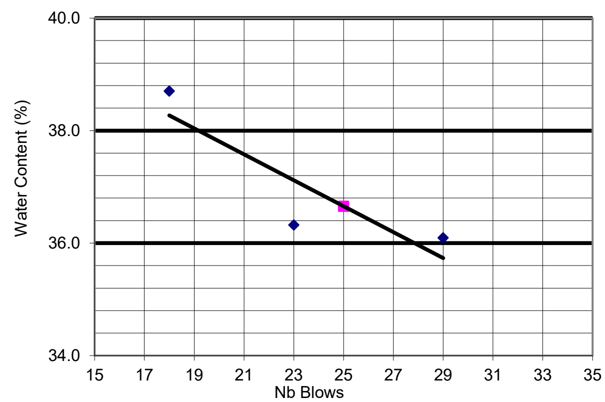
Natural Water Content (W^n):

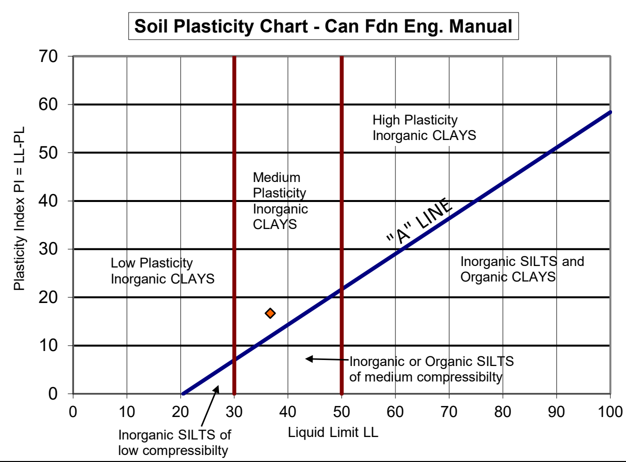
Tare no.	XC19	
Wet soil+tare, g	77.80	
Dry soil+tare, g	61.10	
Mass of water, g	16.70	
Tare, g	4.30	
Mass of soil, g	56.80	
Water content %	29.4%	

Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
37	20	17	29

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks:

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Crown (Bradford) Developments Inc.	**Lab no.:**	WLA 1321-11
Project/Site:	Geotechnical Investigation - 126 and 140 Bradford Street, Barrie, ON	**Project no.:**	12593831.04.001

Borehole no.:	BH6-23	Sample no.:	SS17	Sample Depth:	90.0 ft - 92.0 ft (27.44 m - 28.05 m)
Soil Description:	Non-Plastic			Date sampled:	

Apparatus:	Hand Crank	Balance no.:	WLG-15	Porcelain bowl no.:	Tu
Liquid limit device no.:	WLSA-3B	Oven no.:	WLG-2	Spatula no.:	2
Sieve no.:	WLS-47	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows			
Water Content:			
Tare no.			
Wet soil+tare, g			
Dry soil+tare, g			
Mass of water, g			
Tare, g			
Mass of soil, g			
Water content %			

Plastic Limit (PL) - Water Content:

Tare no.		
Wet soil+tare, g		
Dry soil+tare, g		
Mass of water, g		
Tare, g		
Mass of soil, g		
Water content %		
Average water content %		

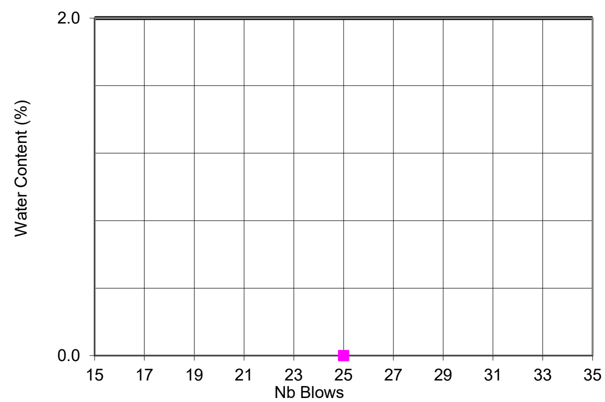
Natural Water Content (W^n):

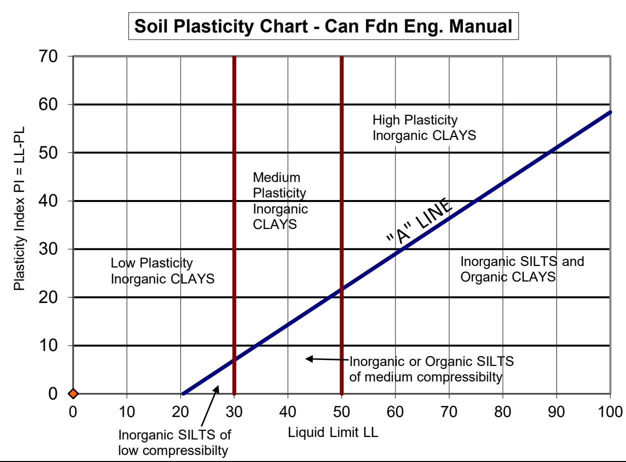
Tare no.	RX07	
Wet soil+tare, g	66.40	
Dry soil+tare, g	56.50	
Mass of water, g	9.90	
Tare, g	4.00	
Mass of soil, g	52.50	
Water content %	18.9%	

Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation (oven dried))
- ☐ Dry preparation (air dried)
- ☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content Wn
			19

Plasticity Chart based on Canadian Foundation Engineering Manual, 2017, and Casagrande, 1948. Additional laboratory reporting information available upon request.

Remarks: Non-Plastic

Performed by:	Melanie Mitchell	**Date:**	March 1-3, 2023
Reviewed by:	Abdul Hafeez Khan, P.Eng.; Laboratory Manager	**Date:**	March 6, 2023
Laboratory Location:	140 Bathurst Drive, Waterloo		

Appendix C

Pressuremeter Test Results

In-Depth
Geotechnical Inc.

In-Situ Pressuremeter Testing **Project No. IDG 230721**
126-140 Bradford Street, Barrie
Boring BH 6-23-PMT
Revised on March 18th, 2023

Prepared for:
Mr. Mrinmoy Kanungo, M.E.Sc., P.Eng.

GHD Limited
111 Brunel Road, Suite #200
Mississauga, Ontario
L4Z 1X3

In-Depth Geotechnical Inc.
20 Ravenscliffe Avenue
Hamilton, Ontario
L8P 3M4
Phone: (905) 541 9937
Fax: (877) 624 0140

Table of Contents

1. Introduction

In-Depth Geotechnical Inc. was retained by GHD Limited to conduct Pressuremeter testing in relation to their Geotechnical Investigation for the 126-140 Bradford Street site, in Barrie, Ontario.

This report presents the results of pressuremeter testing (PMT) carried out at one borehole location with the purpose of evaluating specific parameters related to a) shear strength; and b) deformation properties of the encountered soils.

This report includes data obtained by use of a pre-bored pressuremeter system. Inferred characteristics of the data are also presented including initial contact pressure, limit pressure, secant deformation modulus values during loading, unloading and reloading cycles, and yield pressure if and when justified by the data. Multiple methods are available for interpretation of this data to estimate engineering properties of soils but such methods are not discussed or included in this report except for the characteristics of the data plots as described above.

2. Field Testing Procedures

Pressuremeter testing was performed at one borehole, located at 126-140 Bradford Street, in Barrie.

Details of tested boring are:

Borehole	Number of Tests	Ground Elevation (masl)	Water Elevation (masl)	Maximum Depth (m)
BH 6-23-PMT	3	223.6	222.0	27.5

Field work was completed on February 21 and 22, 2023. Drilling procedures were undertaken by 3D Drilling Contractor. The borehole was advanced using mud rotary drilling technique with a truck-mounted CME 75 drill rig. This boring was dedicated to SPT testing and sampling, vane testing, and PMT testing.

HW casing was installed to a depth of about 1.5 m below the ground surface to prevent soil collapse on the upper part of the boring (collar).

The test sections of the borings were drilled with a tricone bit. The bit was advanced using continuous circulation of drilling mud to flush soil cuttings, producing a controlled diameter hole for the pressuremeter probe. A positive water head was kept inside the surface casing throughout drilling and in-situ testing procedures. In general, the drilling fluid remained at the top of casing.

Pre-boring pressuremeter testing was completed using a TEXAM unit. The testing procedure was in general accordance with Procedure B, volume-controlled loading, as outlined in the ASTM D 4719-00 Standard Test Method for Pre-bored Pressuremeter Testing of Soils. The testing equipment was calibrated for pressure and volume losses as indicated in the above-mentioned standard. The Records of Calibration for the PMT probes utilized in this job are attached on Appendix Three. The control unit was de-aired prior to every test. Also, checks were completed to ensure that the probe, tubing, and control unit assembly were fully saturated, and that the probe membrane was leakage-free at high pressures. Two readings were taken for each volume step, namely for time delays of 15, and 30 seconds.

As per GHD instructions, test procedures also included completion of up to two unload-reload cycles per test, wherever possible.

3. Pressuremeter Test Results

3.1 PMT test parameters

Pressuremeter test data is presented in Appendix One, and the summary of test results are illustrated in Table No. 1, below.

Based on pressuremeter test data, we have included subsoil profiles for the tested borings, plotting the distributions of the interpreted PMT parameters. These profiles are shown in the following pages.

3.2 PMT-Inferred soil parameters

A general guideline to interpret and infer soil properties based on available PMT test data is attached to Appendix Two This guideline suggests accepted current procedures to estimate or infer shear strength, deformation properties, and other related soil parameters. These inferred properties are summarized in Table No. 2, below.

It is recognized that the values of in-situ total horizontal stresses, σ_{h0}, presented in this report correspond to best possible estimates. These estimates were obtained using the *corrected pressure* versus *1/Volume* method, and are used in this report to infer values of the at-rest stress ratio k_0. The following subsurface soil conditions were assumed to apply:

- Ground Surface and Ground Water elevations: as indicated on the Table No. 2 below
- Average wet and saturated unit weights: γ_{wet} = 19 kN/m^3 and γ_{sat} = 20 kN/m^3
- Total horizontal stresses taken as direct values of p_0 (PMT test results).

It is considered that stresses within the soil mass are defined by geostatic conditions, that is to say:

1. No surcharges are applied on the surface (structural loads from existing buildings nearby are negligible),
2. Static groundwater conditions (no seepage occurs),
3. Surface topography is horizontal (no slopes or excavations), and
4. Total vertical stresses are defined by the *wet* (unsaturated soils) and *saturated* (submerged soils) unit weights, γ_{wet} and γ_{sat}, respectively.

Using the *Pressiorama* and the associated *Pressiorama Cyclique Charts* inferred values of Young's Moduli (E_Y), Classification Index (I_c), and drained friction angle (ϕ') are also shown in Table No. 2.

TABLE No. 1 Summary of Pressuremeter Test Results | **Boring BH 6-23-PMT**

Test No.	Surface Elevation (m): 223.60		Contact Pressure	PMT Modulus	Unload - Reload Cycles								Yield Pressure	Net Limit Pressure		
					$E_{Unload\ 1}$	$E_{Reload\ 1}$	Stresses			Strains $\Delta R/R_0$						
	Depth	Elevation	p_0	E_{PMT}			Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	p_y	p^*_L	E_{PMT} / p^*_L	p^*_L / p_y
	[m]	[m]	[kPa]	[MPa]	[MPa]	[MPa]	[kPa]	[kPa]	[kPa]	[%]	[%]	[%]	[kPa]	[kPa]		
1	19.28	204.3	276	13.5	53.7	25.8	629.3	382.3	599.2	7.7	7.1	8.2	742	1412	9.6	1.9
					84.7	42.8	890.7	538.9	858.5	11.4	10.8	11.8				
2	22.68	200.9	321	28.8	71.9	31.3	1075.3	762.2	1036.5	9.5	8.9	10.0	677	1037	27.8	1.5
					68.4	30.0	1147.7	861.7	1117.1	12.2	11.6	12.7				
3	25.45	198.2	350	32.9	108.4	46.7	873.4	394.9	787.9	5.8	5.2	6.3	605	1497	22.0	2.5
					115.2	53.8	1096.6	614.5	1047.0	8.6	8.0	9.1				

Table No. 2			PMT-Inferred Parameters							Boring BH 6-23-PMT			
PMT Test No.	z depth [m]	z_w water [m]	Hydrostatic Pressure [kPa]	Total Stresses		Effective Stresses		Stress Ratio	Young's Modulus		Shear Strength		Classification Index
				Vertical [kPa]	Horizontal [kPa]	Vertical [kPa]	Horizontal [kPa]	k_0	α Menard's Parameter	E_Y [MPa]	Undrained Cohesive Behavior c_u [kPa]	Drained Cohesionless Behavior ϕ' [degrees]	I_c
1	19.28	17.68	173	384	276	211	103	**0.49**	0.59	23	153	24	2.15
2	22.68	21.08	207	452	321	245	114	**0.47**	1.00	29	121	18	1.78
3	25.45	23.85	234	507	350	274	116	**0.42**	0.93	35	160	20	1.93

Notes:

1. Ground elevation (m) 223.60 Water elevation (m) 222.00 Water depth (m) 1.60
2. Wet unit weight of soil 19.0 [kN/m^3] Saturated unit weight of soil 20.0 [kN/m^3]
3. Observations on Shear Strength Parameters (SSP):
   SSP are considered either for Undrained Conditions (Short Term) or Drained Conditions (Long Term). These two conditions are mutually exclusive.
   Undrained Conditions imply cohesion is c_u , and ϕ = 0. ***Drained Conditions*** imply negligible cohesion or c'=0, and $\phi = \phi'$
   Based on the Classification Index I_C (Soil Behavior Type), the suggested values of the SSP are highlighted in green (Thick box border)
4. The Classification Index parameter, I_C, is indicative of the soil type of behavior. It does not exactly relate to the Soil Classification types as those obtained via Grain-Size Distribution analyses. I_C varies from 1.0 to 4.5, from soft clays (cohesive) to dense coarse sands (frictional), correspondingly.

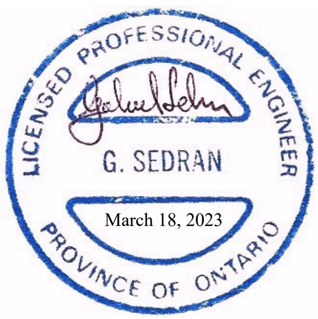
4. Closure

The subsoils data presented in this report is based on in-situ PMT testing and interpretation procedures. It should be noted that soil conditions may vary within the site and interpreted data may not be entirely representative of conditions at locations away from the tested boring. Therefore, care should be exercised when extrapolating or inferring subsoil conditions away from the borehole location.

We trust that the present report fulfills your requirements. Should you have any question, please feel free to contact the undersigned.

Sincerely,

In-Depth Geotechnical Inc.



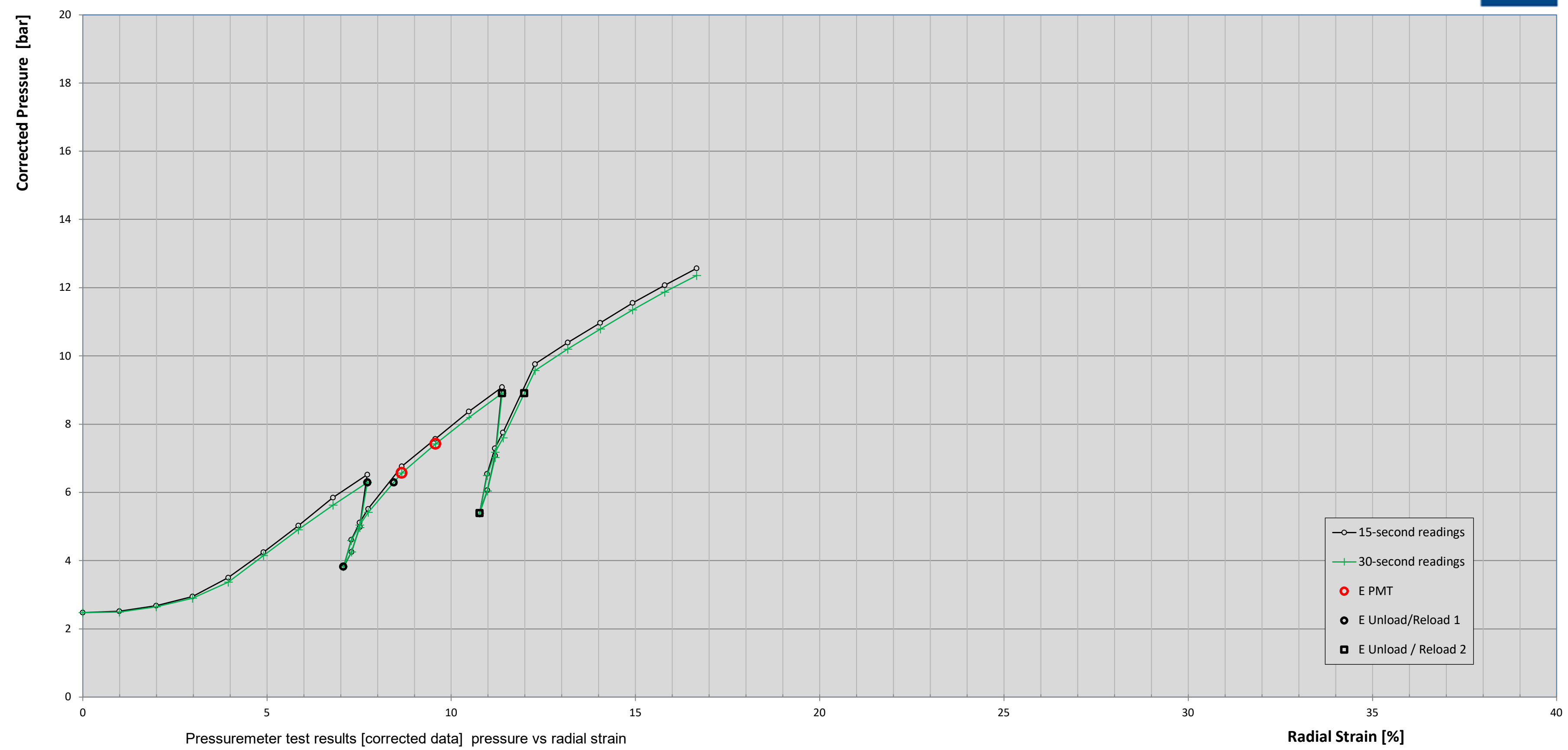
Gabriel Sedran, P.Eng., Ph.D.
President

Appendix One

Pressuremeter Results - Data

BH 6-23-PMT pages 1 to 3

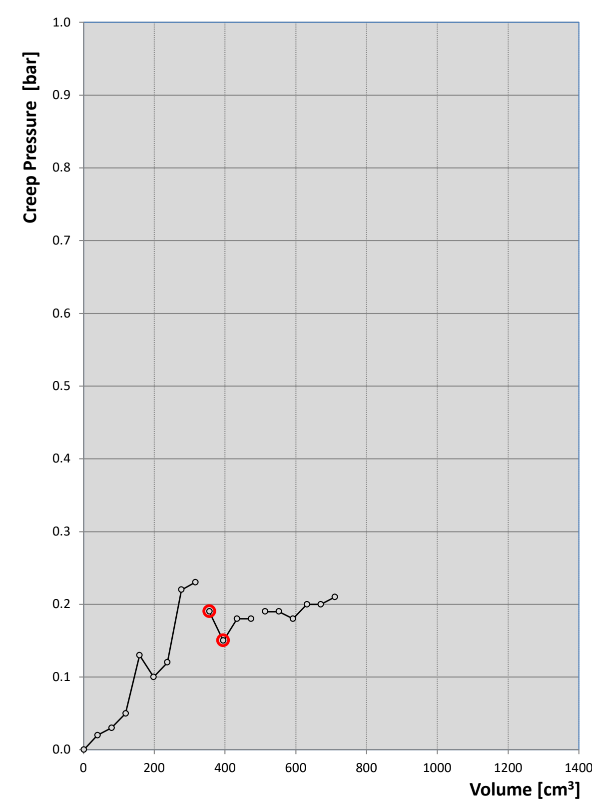
Field Test Data (uncorrected)			Corrected Test data						Creep		Auxiliary Data	
			15-second readings			30-second readings					30 sec	
Volume [cm³]	Pressure [bar] 15 sec	Pressure [bar] 30 sec	Pressure [bar]	Volume [cm³]	$\Delta r/r_0$ [%]	Pressure [bar]	Volume [cm³]	$\Delta r/r_0$ [%]	Volume [cm³]	$\Delta p_{30\text{-}15}$ [bar]	Pressure [bar]	1 / V
1	0.67	0.67	2.48	1	0.00	2.48	1	0.00	1	0.00	2.48	1.66819
40	0.75	0.73	2.52	39.3	0.99	2.50	39.3	0.99	39.3	0.02	2.50	0.02542
80	0.96	0.93	2.68	79.1	1.99	2.65	79.2	1.99	79.2	0.03	2.65	0.01263
120	1.26	1.21	2.95	118.9	2.98	2.90	118.9	2.98	118.9	0.05	2.90	0.00841
160	1.83	1.70	3.50	158.4	3.95	3.37	158.5	3.95	158.5	0.13	3.37	0.00631
200	2.60	2.50	4.25	197.7	4.90	4.15	197.8	4.90	197.8	0.10	4.15	0.00506
240	3.40	3.28	5.03	237.0	5.85	4.91	237.1	5.85	237.1	0.12	4.91	0.00422
280	4.24	4.02	5.85	276.2	6.79	5.63	276.4	6.79	276.4	0.22	5.63	0.00362
320	4.93	4.70	6.52	315.6	7.72	6.29	315.8	7.73	315.8	0.23	6.29	0.00317
310	3.39	3.37	4.99	307.0	7.52	4.97	307.0	7.52			4.97	0.00326
300	2.66	2.66	4.26	297.6	7.30	4.26	297.6	7.30			4.26	0.00336
290	2.19	2.22	3.79	288.0	7.07	3.82	288.0	7.07			3.82	0.00347
300	3.01	2.99	4.61	297.3	7.29	4.59	297.3	7.29			4.59	0.00336
310	3.52	3.45	5.12	306.8	7.51	5.05	306.9	7.52			5.05	0.00326
320	3.92	3.82	5.51	316.5	7.74	5.41	316.6	7.74			5.41	0.00316
360	5.18	4.99	6.76	355.4	8.66	6.57	355.5	8.66	355.5	0.19	6.57	0.00281
400	6.00	5.85	7.57	394.6	9.57	7.42	394.8	9.57	394.8	0.15	7.42	0.00253
440	6.82	6.64	8.37	433.9	10.48	8.19	434.0	10.48	434.0	0.18	8.19	0.00230
480	7.55	7.37	9.09	473.2	11.38	8.91	473.4	11.38	473.4	0.18	8.91	0.00211
470	5.54	5.48	7.08	465.0	11.19	7.02	465.1	11.19			7.02	0.00215
460	4.52	4.50	6.07	455.9	10.98	6.05	456.0	10.98			6.05	0.00219
450	3.85	3.84	5.40	446.5	10.77	5.39	446.6	10.77			5.39	0.00224
460	5.00	4.95	6.55	455.5	10.97	6.50	455.6	10.97			6.50	0.00220
470	5.75	5.64	7.29	464.8	11.19	7.18	464.9	11.19			7.18	0.00215
480	6.22	6.06	7.76	474.4	11.40	7.60	474.6	11.41			7.60	0.00211
520	8.24	8.05	9.76	512.6	12.27	9.57	512.8	12.28	512.8	0.19	9.57	0.00195
560	8.88	8.69	10.39	552.0	13.16	10.20	552.2	13.17	552.2	0.19	10.20	0.00181
600	9.47	9.29	10.97	591.5	14.04	10.79	591.7	14.05	591.7	0.18	10.79	0.00169
640	10.06	9.86	11.55	631.0	14.92	11.35	631.2	14.92	631.2	0.20	11.35	0.00158
680	10.59	10.39	12.07	670.5	15.79	11.87	670.7	15.79	670.7	0.20	11.87	0.00149
720	11.09	10.88	12.57	710.1	16.66	12.36	710.2	16.66	710.2	0.21	12.36	0.00141



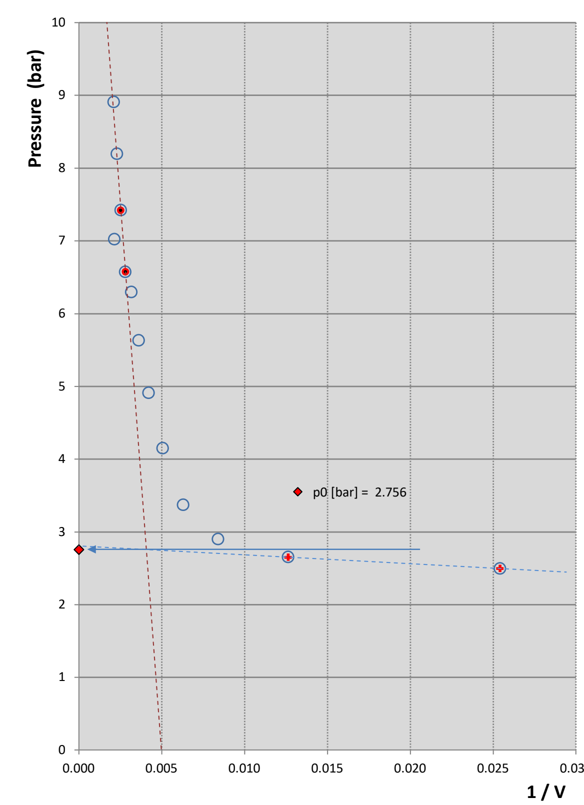
Pressuremeter test results [corrected data] pressure vs radial strain

Interpreted PMT Test Results

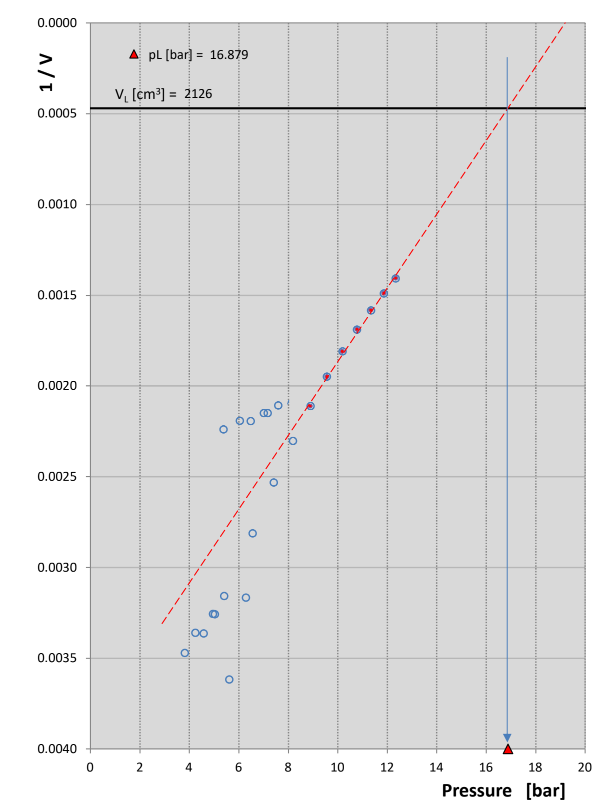
[30-second readings]			volume [cm³]	radial strain [%]	strain range [%]
p_0	2.76	[bar]	79.2	2.0	
p_L	16.88	[bar]			
p^*_L	14.12	[bar]			
p_Y	7.42	[bar]	395	9.6	
E_{PMT}	135	[bar]	356	8.7	{8.7 - 9.6 %}
E_{PMT} / p^*_L	9.5				
$E_{Unload\ 1}$	537	[bar]	288	7.1	
$E_{Reload\ 1}$	258	[bar]			
$E_{Unload\ 2}$	847	[bar]	447	10.8	
$E_{Reload\ 2}$	428	[bar]			



Pressure difference from 15 to 30 sec. readings $\Delta p_{[15\text{-}30\ sec]}$



Determination of total contact pressure p_0



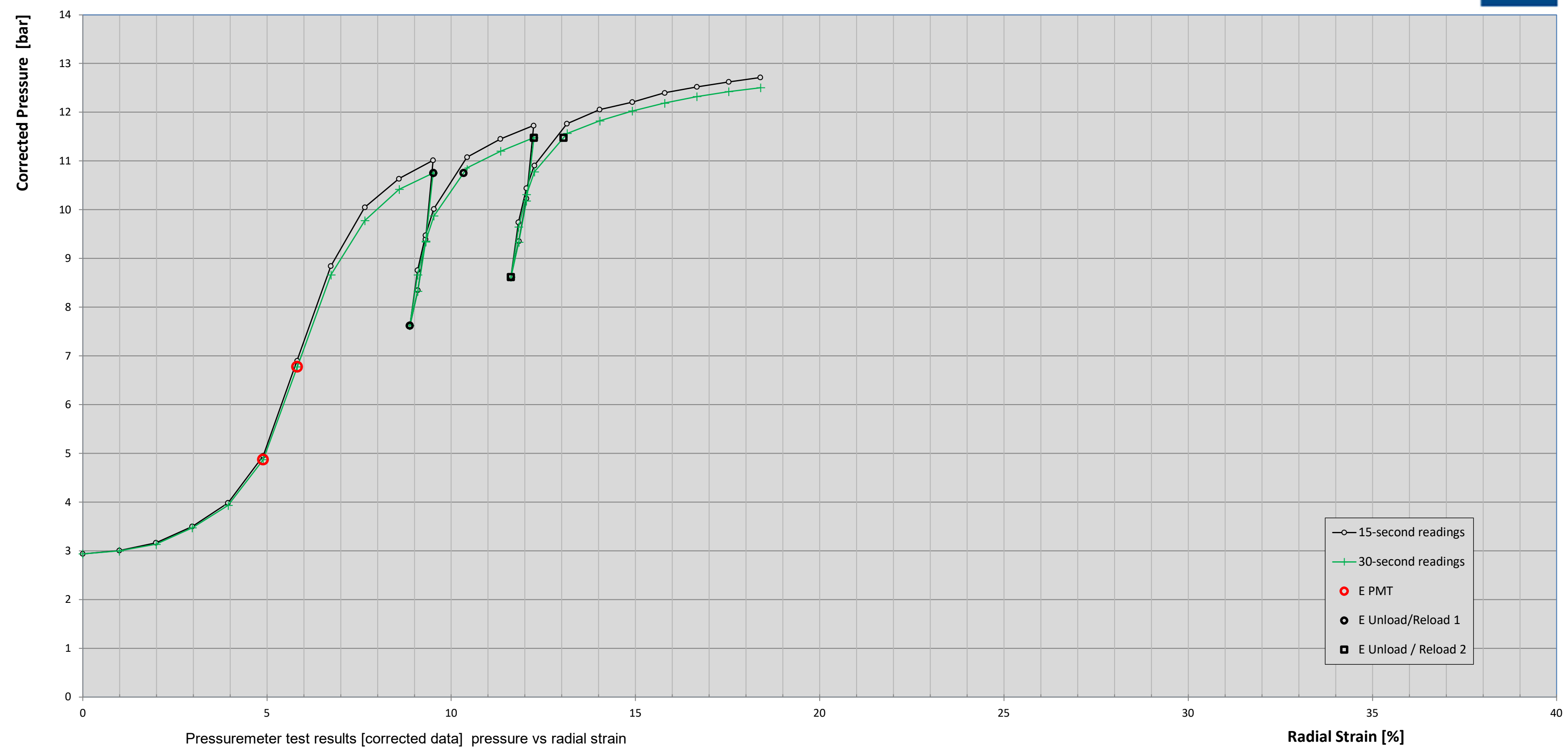
Determination of Limit Pressure p_L

Pressuremeter Equipment: TEXAM Model	Probe Designation : NX Probe (76 mm OD)	Drilling Method: Mud Rotary Drilling Drilling Bit: Tricone Bit	Test Date: February 21, 2023
Volume-controlled test as per ASTM D4719 Method B	Probe No.: B 504 Calibration Record No.: 1	Time elapsed from hole drilling to testing ~ 5 minutes	Test Depth [m]: 19.28 (center of the probe)
Volume increments: 40 cm³ Maximum Volume: 1400 cm³ Maximum Pressure: 100 bar	Tubing Length: 180 [ft] Probe Lenght: 0.46 [m] Probe Initial Volume: 1968 cm³	Engineer: Gabriel Sedran, P.Eng., Ph.D. Operator: G.S.	Drilling Company: 3D Drilling

Project:	**140 Brantford Street, Barrie**	**PMT TEST No.: 1**
Client:	GHD Limited	**Borehole No.: BH 6-23-PMT**
In-Depth Geotechnical Project No.:	IDG 230721	

In-Depth Geotechnical Inc.

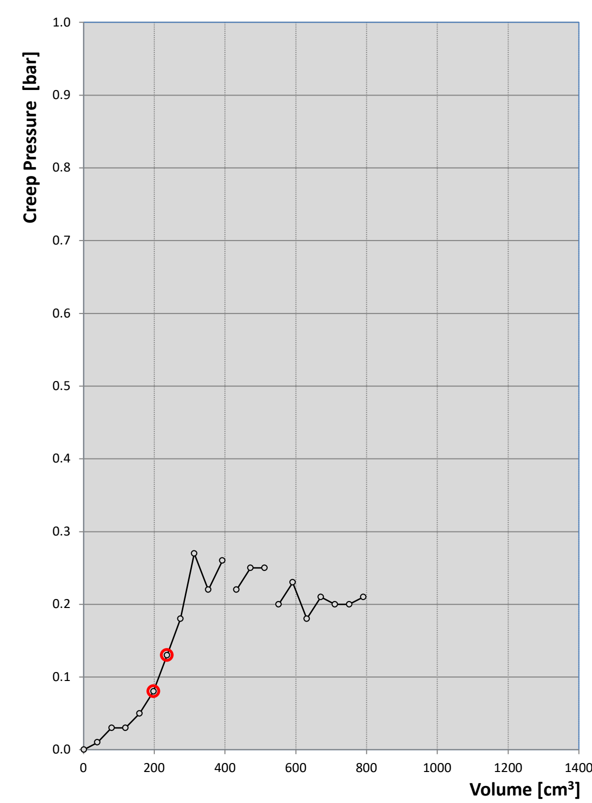
Field Test Data (uncorrected)			Corrected Test data						Creep		Auxiliary Data	
			15-second readings			30-second readings					30 sec	
Volume [cm³]	Pressure [bar] 15 sec	Pressure [bar] 30 sec	Pressure [bar]	Volume [cm³]	$\Delta r/r_0$ [%]	Pressure [bar]	Volume [cm³]	$\Delta r/r_0$ [%]	Volume [cm³]	Δp_{30-15} [bar]	Pressure [bar]	1 / V
1	0.80	0.80	2.94	0	0.00	2.94	0	0.00	0	0.00	2.94	2.07070
40	0.91	0.90	3.01	39.2	0.99	3.00	39.2	0.99	39.2	0.01	3.00	0.02551
80	1.11	1.08	3.16	79.0	1.99	3.13	79.0	1.99	79.0	0.03	3.13	0.01265
120	1.48	1.45	3.50	118.7	2.97	3.47	118.7	2.97	118.7	0.03	3.47	0.00842
160	1.98	1.93	3.98	158.2	3.94	3.93	158.3	3.94	158.3	0.05	3.93	0.00632
200	2.97	2.89	4.95	197.3	4.89	4.87	197.4	4.90	197.4	0.08	4.87	0.00507
240	4.94	4.81	6.90	235.6	5.82	6.77	235.7	5.82	235.7	0.13	6.77	0.00424
280	6.90	6.72	8.84	273.8	6.73	8.66	274.0	6.74	274.0	0.18	8.66	0.00365
320	8.12	7.85	10.05	312.7	7.65	9.78	313.0	7.66	313.0	0.27	9.78	0.00320
360	8.72	8.50	10.63	352.2	8.58	10.41	352.4	8.59	352.4	0.22	10.41	0.00284
400	9.11	8.85	11.01	391.8	9.50	10.75	392.1	9.51	392.1	0.26	10.75	0.00255
390	7.49	7.43	9.40	383.3	9.31	9.34	383.3	9.31			9.34	0.00261
380	6.44	6.42	8.35	374.2	9.10	8.33	374.2	9.10			8.33	0.00267
370	5.70	5.71	7.61	364.9	8.88	7.62	364.9	8.88			7.62	0.00274
380	6.85	6.75	8.76	373.9	9.09	8.66	373.9	9.09			8.66	0.00267
390	7.57	7.45	9.48	383.2	9.30	9.36	383.3	9.31			9.36	0.00261
400	8.11	7.97	10.01	392.7	9.53	9.87	392.9	9.53			9.87	0.00255
440	9.19	8.97	11.08	431.8	10.43	10.86	432.0	10.43	432.0	0.22	10.86	0.00232
480	9.58	9.33	11.45	471.4	11.34	11.20	471.6	11.34	471.6	0.25	11.20	0.00212
520	9.87	9.62	11.73	511.2	12.24	11.48	511.4	12.24	511.4	0.25	11.48	0.00196
510	8.37	8.32	10.23	502.5	12.04	10.18	502.5	12.04			10.18	0.00199
500	7.49	7.47	9.35	493.3	11.83	9.33	493.3	11.83			9.33	0.00203
490	6.76	6.75	8.63	483.9	11.62	8.62	483.9	11.62			8.62	0.00207
500	7.88	7.78	9.74	492.9	11.83	9.64	493.0	11.83			9.64	0.00203
510	8.58	8.45	10.44	502.3	12.04	10.31	502.4	12.04			10.31	0.00199
520	9.05	8.92	10.91	511.9	12.26	10.78	512.0	12.26			10.78	0.00195
560	9.92	9.72	11.76	551.1	13.14	11.56	551.3	13.14	551.3	0.20	11.56	0.00181
600	10.22	9.99	12.05	590.8	14.03	11.82	591.0	14.03	591.0	0.23	11.82	0.00169
640	10.38	10.20	12.20	630.7	14.91	12.02	630.9	14.92	630.9	0.18	12.02	0.00159
680	10.58	10.37	12.40	670.5	15.79	12.19	670.7	15.80	670.7	0.21	12.19	0.00149
720	10.71	10.51	12.52	710.4	16.66	12.32	710.6	16.67	710.6	0.20	12.32	0.00141
760	10.82	10.62	12.62	750.3	17.53	12.42	750.5	17.53	750.5	0.20	12.42	0.00133
800	10.92	10.71	12.71	790.2	18.39	12.50	790.4	18.39	790.4	0.21	12.50	0.00127



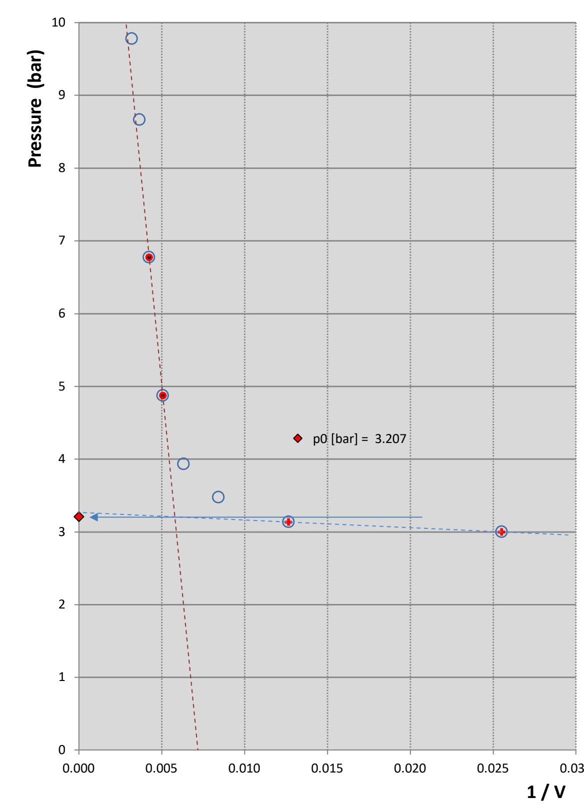
Pressuremeter test results [corrected data] pressure vs radial strain

Interpreted PMT Test Results

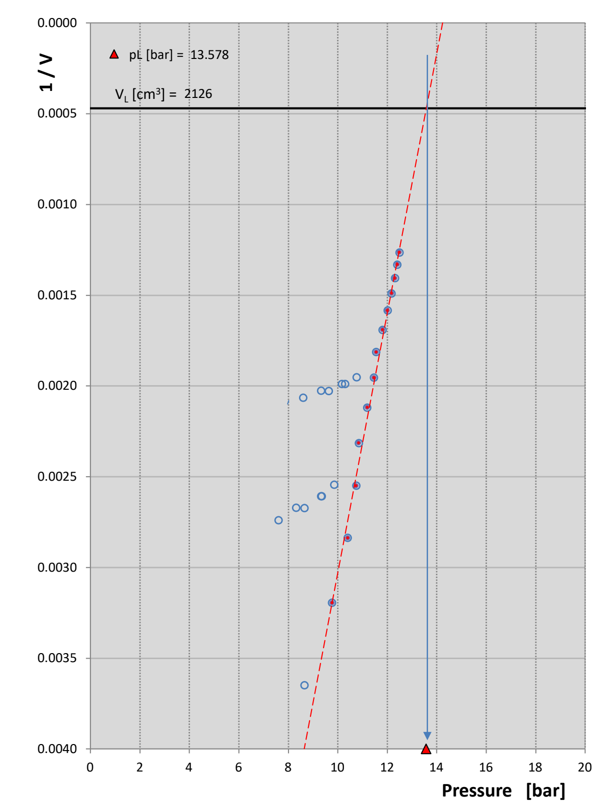
[30-second readings]			volume [cm³]	radial strain [%]	strain range [%]
p_0	3.21	[bar]	79.0	2.0	
p_L	13.58	[bar]			
p^*_L	10.37	[bar]			
p_Y	6.77	[bar]	236	5.8	
E_{PMT}	288	[bar]	197	4.9	{4.9 - 5.8 %}
E_{PMT} / p^*_L	27.8				
$E_{Unload\ 1}$	719	[bar]	365	8.9	
$E_{Reload\ 1}$	313	[bar]			
$E_{Unload\ 2}$	684	[bar]	484	11.6	
$E_{Reload\ 2}$	300	[bar]			



Pressure difference from 15 to 30 sec. readings $\Delta p_{[15-30\ sec]}$



Determination of total contact pressure p_0



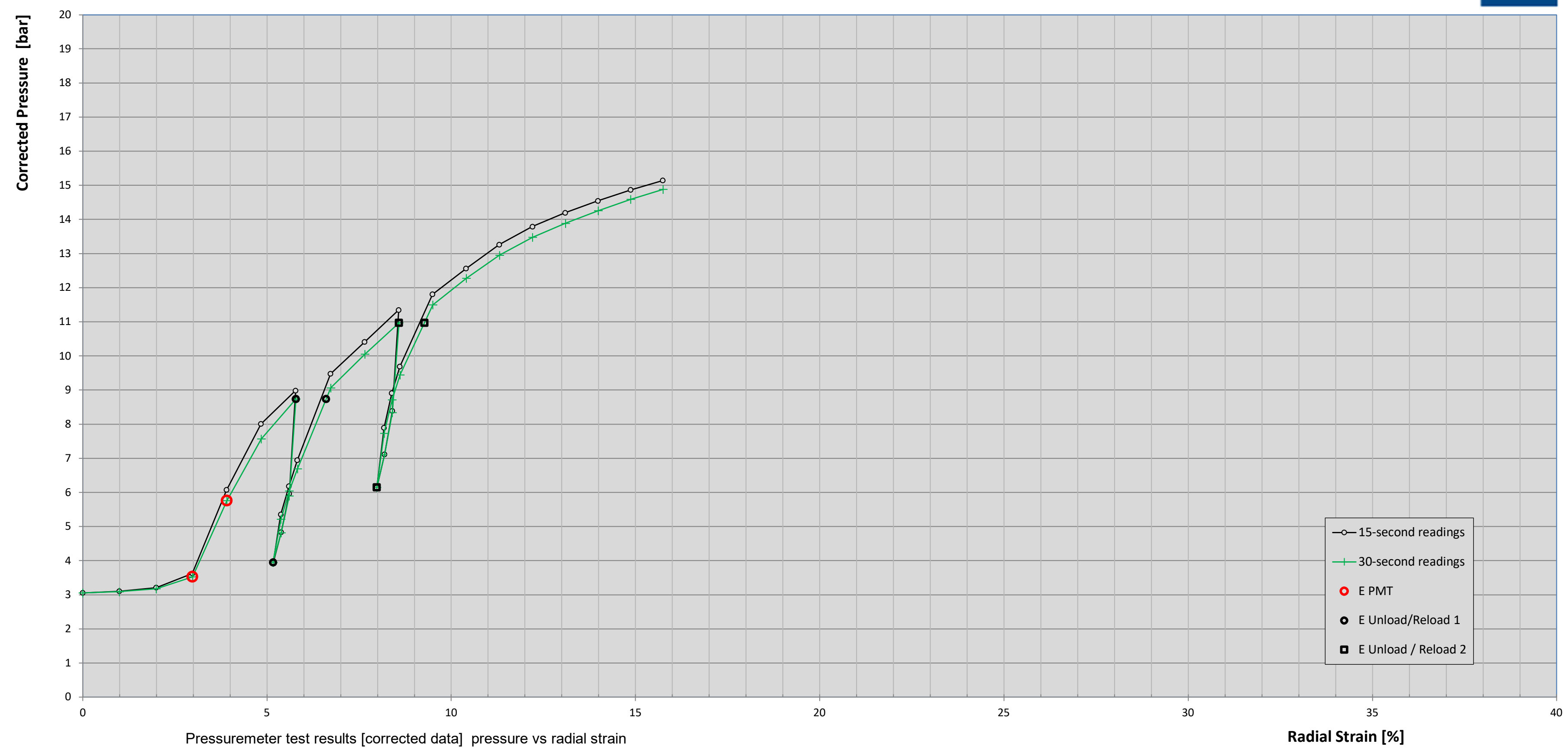
Determination of Limit Pressure p_L

Pressuremeter Equipment: TEXAM Model	Probe Designation : NX Probe (76 mm OD)	Drilling Method: Mud Rotary Drilling Drilling Bit: Tricone Bit	Test Date: February 21, 2023
Volume-controlled test as per ASTM D4719 Method B	Probe No.: B 504 Calibration Record No.: 1	Time elapsed from hole drilling to testing ~ 5 minutes	Test Depth [m]: 22.68 (center of the probe)
Volume increments: 40 cm³ Maximum Volume: 1400 cm³ Maximum Pressure: 100 bar	Tubing Length: 180 [ft] Probe Lenght: 0.46 [m] Probe Initial Volume: 1968 cm³	Engineer: Gabriel Sedran, P.Eng., Ph.D. Operator: G.S.	Drilling Company: 3D Drilling

Project:	**140 Brantford Street, Barrie**	**PMT TEST No.: 2**
Client:	GHD Limited	**Borehole No.: BH 6-23-PMT**
In-Depth Geotechnical Project No.:	IDG 230721	

In-Depth Geotechnical Inc.

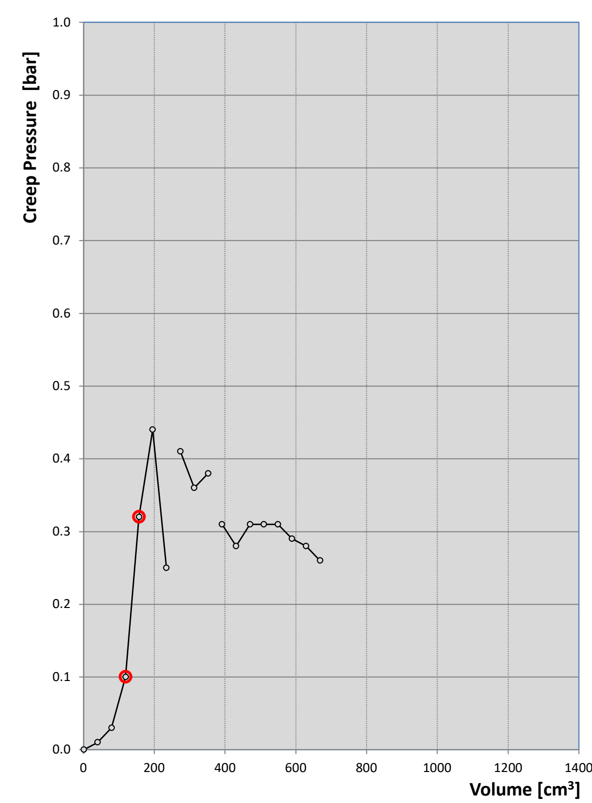
Field Test Data (uncorrected)			Corrected Test data						Creep		Auxiliary Data	
			15-second readings			30-second readings					30 sec	
Volume	Pressure [bar]		Pressure	Volume	$\Delta r/r_0$	Pressure	Volume	$\Delta r/r_0$	Volume	Δp_{30-15}	Pressure	1 / V
[cm³]	15 sec	30 sec	[bar]	[cm³]	[%]	[bar]	[cm³]	[%]	[cm³]	[bar]	[bar]	
1	0.64	0.64	3.05	1	0.00	3.05	1	0.00	1	0.00	3.05	1.59657
40	0.73	0.72	3.10	39.3	0.99	3.09	39.4	1.00	39.4	0.01	3.09	0.02541
80	0.88	0.85	3.21	79.2	1.99	3.18	79.2	1.99	79.2	0.03	3.18	0.01262
120	1.33	1.23	3.62	118.8	2.97	3.52	118.9	2.98	118.9	0.10	3.52	0.00841
160	3.80	3.48	6.07	156.6	3.90	5.75	156.9	3.91	156.9	0.32	5.75	0.00637
200	5.75	5.31	8.00	194.8	4.83	7.56	195.2	4.84	195.2	0.44	7.56	0.00512
240	6.75	6.50	8.98	233.9	5.78	8.73	234.2	5.78	234.2	0.25	8.73	0.00427
230	3.72	3.66	5.96	226.7	5.60	5.90	226.7	5.60			5.90	0.00441
220	2.59	2.57	4.83	217.7	5.39	4.81	217.7	5.39			4.81	0.00459
210	1.68	1.70	3.93	208.5	5.16	3.95	208.5	5.16			3.95	0.00480
220	3.11	2.97	5.35	217.2	5.37	5.21	217.3	5.38			5.21	0.00460
230	3.94	3.80	6.18	226.5	5.60	6.04	226.6	5.60			6.04	0.00441
240	4.71	4.46	6.94	235.8	5.82	6.69	236.0	5.83			6.69	0.00424
280	7.26	6.85	9.47	273.5	6.72	9.06	273.9	6.73	273.9	0.41	9.06	0.00365
320	8.21	7.85	10.41	312.6	7.65	10.05	313.0	7.66	313.0	0.36	10.05	0.00320
360	9.16	8.78	11.35	351.8	8.57	10.97	352.1	8.58	352.1	0.38	10.97	0.00284
350	6.20	6.15	8.39	344.4	8.40	8.34	344.5	8.40			8.34	0.00290
340	4.92	4.91	7.11	335.6	8.19	7.10	335.6	8.19			7.10	0.00298
330	3.94	3.95	6.14	326.5	7.98	6.15	326.5	7.98			6.15	0.00306
340	5.70	5.54	7.89	334.9	8.18	7.73	335.0	8.18			7.73	0.00298
350	6.72	6.52	8.91	344.0	8.39	8.71	344.2	8.39			8.71	0.00291
360	7.50	7.26	9.69	353.3	8.61	9.45	353.5	8.61			9.45	0.00283
400	9.63	9.32	11.80	391.4	9.49	11.49	391.6	9.50	391.6	0.31	11.49	0.00255
440	10.40	10.12	12.56	430.7	10.40	12.28	430.9	10.41	430.9	0.28	12.28	0.00232
480	11.12	10.81	13.26	470.0	11.30	12.95	470.3	11.31	470.3	0.31	12.95	0.00213
520	11.66	11.35	13.79	509.5	12.20	13.48	509.8	12.21	509.8	0.31	13.48	0.00196
560	12.08	11.77	14.20	549.2	13.10	13.89	549.5	13.10	549.5	0.31	13.89	0.00182
600	12.44	12.15	14.54	588.8	13.98	14.25	589.1	13.99	589.1	0.29	14.25	0.00170
640	12.77	12.49	14.87	628.6	14.87	14.59	628.8	14.87	628.8	0.28	14.59	0.00159
680	13.05	12.79	15.14	668.3	15.74	14.88	668.5	15.75	668.5	0.26	14.88	0.00150



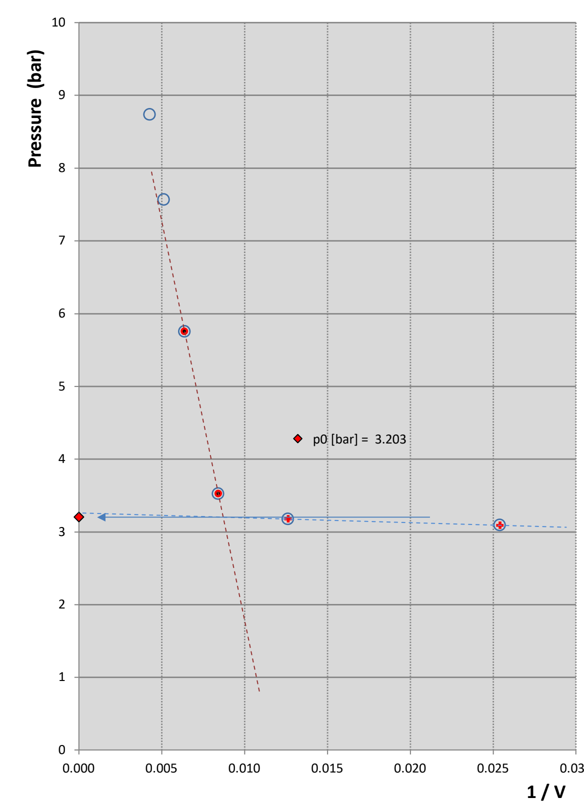
Pressuremeter test results [corrected data] pressure vs radial strain

Interpreted PMT Test Results

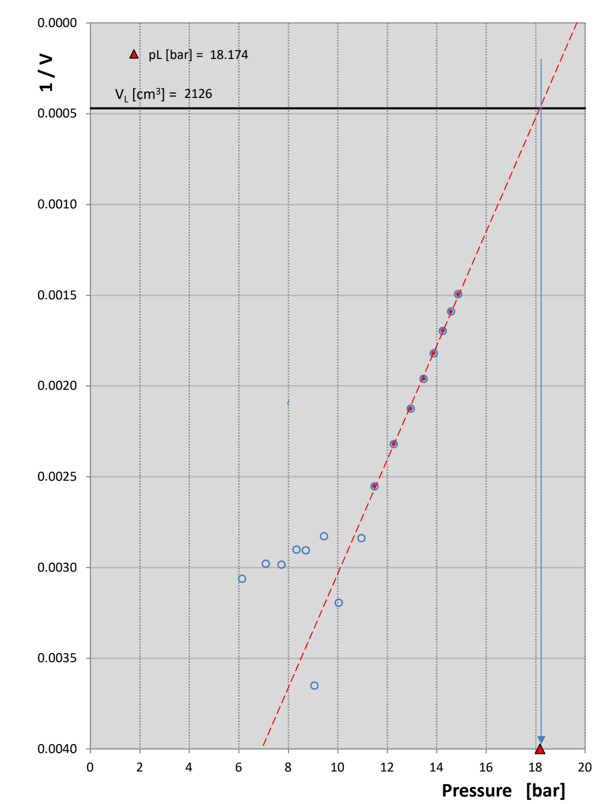
[30-second readings]			volume [cm³]	radial strain [%]	strain range [%]
p_0	3.20	[bar]	79.2	2.0	
p_L	18.17	[bar]			
p^*_L	14.97	[bar]			
p_Y	5.75	[bar]	157	3.9	
E_{PMT}	329	[bar]	119	3.0	{3.0 - 3.9 %}
E_{PMT} / p^*_L	22.0				
$E_{Unload\ 1}$	1084	[bar]	208	5.2	
$E_{Reload\ 1}$	467	[bar]			
$E_{Unload\ 2}$	1152	[bar]	326	8.0	
$E_{Reload\ 2}$	538	[bar]			



Pressure difference from 15 to 30 sec. readings $\Delta p_{[15-30\ sec]}$



Determination of total contact pressure p_0



Determination of Limit Pressure p_L

Pressuremeter Equipment: TEXAM Model	Probe Designation : NX Probe (76 mm OD)	Drilling Method: Mud Rotary Drilling Drilling Bit: Tricone Bit	Test Date: February 22, 2023
Volume-controlled test as per ASTM D4719 Method B	Probe No.: B 504 Calibration Record No.: 1	Time elapsed from hole drilling to testing ~ 5 minutes	Test Depth [m]: 25.45 (center of the probe)
Volume increments: 40 cm³ Maximum Volume: 1400 cm³ Maximum Pressure: 100 bar	Tubing Length: 180 [ft] Probe Lenght: 0.46 [m] Probe Initial Volume: 1968 cm³	Engineer: Gabriel Sedran, P.Eng., Ph.D. Operator: G.S.	Drilling Company: 3D Drilling

Project:	**140 Brantford Street, Barrie**	**PMT TEST No.:**	**3**
Client:	GHD Limited	**Borehole No.:**	**BH 6-23-PMT**
In-Depth Geotechnical Project No.:	IDG 230721		

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In-Depth
Geotechnical Inc.

Appendix Two

Pressuremeter Data Interpretation

Interpretation of Pressuremeter Test Results

Prebored pressuremeter test results are expressed in terms of applied pressure versus radial strain. Both pressure and strain measurements must be corrected for pressure and volume loses using the corresponding probe and system calibration curves.

The typical pressure versus radial strain curve features up to four distinctive portions which characterize the stress-strain behaviour of the soil, namely:

a) The linear pseudo-elastic stress-strain portion of the deformation curve;
b) The departure from linear elastic conditions starting at the yield pressure p_y;
c) The unload-reload portion of the test (usually two cycles are performed); and
d) The development of soil failure, which is represented by the net limit pressure p^*_L.

Based on these test features the following soil parameters are determined or estimated:

1. **Contact Pressure p_o**:

When using the prebored TEXAM unit, the initial contact pressure is taken as the pressure at the intersection of the two lines representing the pseudo elastic and the initial expansion portions of the pressure vs. 1/V plot, as shown in the PMT data sheets, in Appendix One.

2. **Pressuremeter modulus E_{PMT}**:

The pressuremeter modulus is represented by the slope of the pressure versus radial strain curve along its linear portion, and may be calculated as follows:

$$E_{PMT} = (1+\upsilon)(p_2 - p_1)\frac{\left(1+\left(\frac{\Delta R}{R_o}\right)_2\right)^2 + \left(1+\left(\frac{\Delta R}{R_o}\right)_1\right)^2}{\left(1+\left(\frac{\Delta R}{R_o}\right)_2\right)^2 - \left(1+\left(\frac{\Delta R}{R_o}\right)_1\right)^2}$$

where the sub-indices 1 and 2 indicate the beginning and the end of the linear portion of the curve, respectively. These two points are shown in pressuremeter curves with two red oversized circles. For the self-boring probe, the linear portion of the stress-strain response occurs between the very first data point (zero volume increase) and the subsequent two or three data points.

In this determination a value of the Poisson's ratio, typically $\nu = 0.33$ for most soils, must be assumed. For saturated clays a value of $\nu = 0.45$ is suggested.

3. **Yield Pressure** p_y:

The yield pressure indicates the end of the linear pseudo-elastic deformations and the onset of plasticity. This yield pressure is useful in indicating beyond which pressure significant creep deformations may occur.

4. **Unload-Reload Moduli** E_{Unload} and E_{Reload}

The unload and reload moduli are represented by the slope of the unload-reload loop, and they may be used to determine elastic soil deformations upon unloading or reloading conditions such as those typically encountered during excavations.

5. **Net Limit Pressure** p^*_L:

The net limit pressure is a measure of the strength of the soil (either under undrained conditions for cohesive soils, or drained conditions for non-cohesive soils). This parameter is defined as the pressure reached when the soil cavity has been extended to twice its original soil cavity volume V_c (minus the initial total contact pressure p_o).

The limit pressure is not always attained during testing. In such cases, the value of p_L is inferred by plotting pressure versus 1/V for the plastic phase of the deformations. This method of inferring p_L, known as the "upside down curve" method, is described in "*The Pressuremeter and Foundation Engineering*" textbook, by F. Baguelin, J.F. Jezequel, and D.H. Shields, published in 1978 by Trans Tech Publications, Section: Methods of extrapolating pressuremeter curves to p_L. See also ASTM D4719-00, Section 10.6.

It should be noted that radial strains are calculated from the volume of fluid (typically tap water) injected into the probe. In this regard, the radial strains shown in the results are related to the probe expansion, not the cavity's expansion. The cavity initial volume, V_c, is calculate by adding the probe initial volume, V_0, to the volume of water injected into the probe at the initial contact pressure p_0.

6. Some Additional PMT-based Parameters

In addition, two useful ratios, (E_{PMT} / p^*_L) and (p^*_L / p_y), may be used as a general guideline for soil identification, as follows:

for sands $7 < E_{PMT} / p^*_L < 12$

for clays $12 < E_{PMT} / p^*_L$

Many PMT tests completed in the glacial tills present in the geology of the Golden Shoe area (Ontario) registered much higher values than those listed above. In many cases, values for E_{PMT} / p^*_L in excess of 30 have been recorded.

The E_{PMT} / p^*_L value is known as the *mechanical ratio*, and it indicates whether a soil mass behaves in a ductile (high value) or brittle (low value) manner after yield stresses have been reached. This ration It is the PMT equivalent of the soil mechanic's Rigidity Index, e.g., G/σ_{max}.

Inferred Soil Parameters

7. Young's Modulus E_Y

The Pressuremeter modulus E_{PMT} corresponds to large strains, namely for radial strains in the 2 to 5 % range, and it is therefore considered to be a relatively low value of the elastic modulus. In practice, the Young's modulus E can be inferred from Pressuremeter testing using the empirical Menard α factor:

$$E_Y = E_{PMT} / \alpha$$

Typical values of the Menard α factor are suggested in the following Table:

Soil type	Peat		Clay		Silt		Sand		Sand and gravel	
	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α
Over consolidated		1	> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally consolidated	For all values	1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Weathered and/or remoulded		1	7-9	1/2		1/2		1/3		1/4
Rock	Extremely fractured				Other				Slightly fractured or extremely weathered	
	$\alpha = 1/3$				$\alpha = 1/2$				$\alpha = 2/3$	

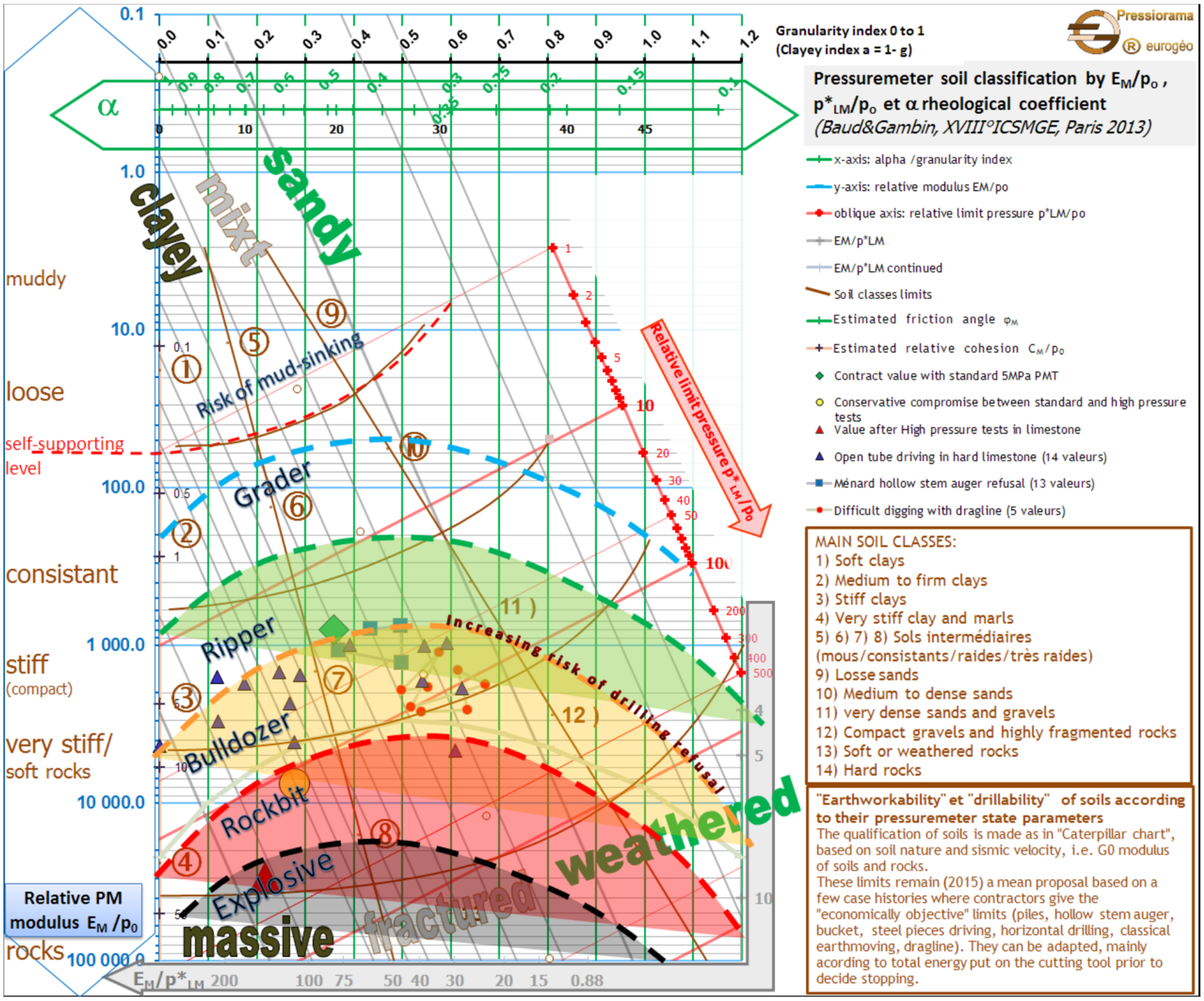
(from 'The Pressuremeter', J.L. Briaud. Balkema, 1992)

Alternatively, better-defined values of the Menard α parameter can be obtained using the following expression, as introduced by J.P. Baud

$$\alpha = \frac{\left({E_{PMT}}/{P_L^*} \right)^{1/n}}{k_E \left(\frac{P_L^*}{p_0} \right)^{m/n}}$$

With $n = 2$; $m = 0.5$; and $k_E = 3.5$.

This expression is based on empirical correlations and may also be visualized in the Pressiorama Chart illustrated in the next page:



Baud J.P., and Gambin M. 2013. *"Détermination du coefficient rhéologique α de Ménard dans le diagramme Pressiorama"*. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering. Paris, 2013, Parallel Session ISP 6, International Symposium on the Pressuremeter.

8. Undrained Shear Strength for Cohesive Soil Materials

The undrained shear strength of cohesive soils, c_u or S_u, may be estimated as:

$$\frac{S_u}{p_a} = 0.21\left(\frac{p_L^*}{p_a}\right)^{0.75}$$

where p_a represents a reference pressure (i.e., atmospheric pressure = 100 kPa), after J.L. Briaud ('The Pressuremeter', Balkema, 1992).

9. Drained Friction Angle for Cohesionless Soil Materials

The drained friction angle of cohesionless soils (for $c' = 0$) may be estimated using the empirical correlations illustrated in the graph shown below. This approach is outlined by Baguelin et.al., in *"The Pressuremeter and Foundation Engineering"* (F. Baguelin; J.F. Jézéquel; and D.H. Shields. TransTech Publications. 1978), and it requires some knowledge on the state or conditions of the cohesionless material. This approach only provides a likely range of friction angles for recorded values of the limit pressure.

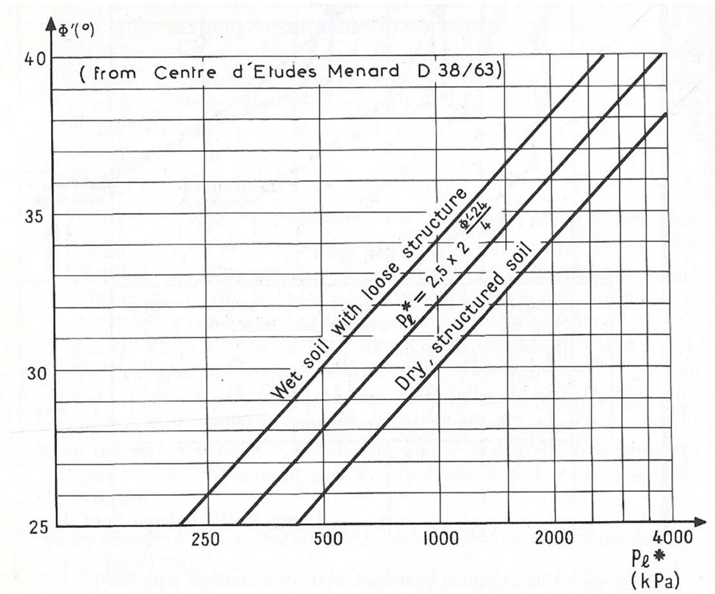


Fig. 6-86: MÉNARD's graph to determine Φ' from p_1^*.

Also alternatively, values of the drained friction angle ϕ' can be inferred using the modified Pressiorama Chart (*Pressiorama Cyclique, in French*) as introduced by Baud.

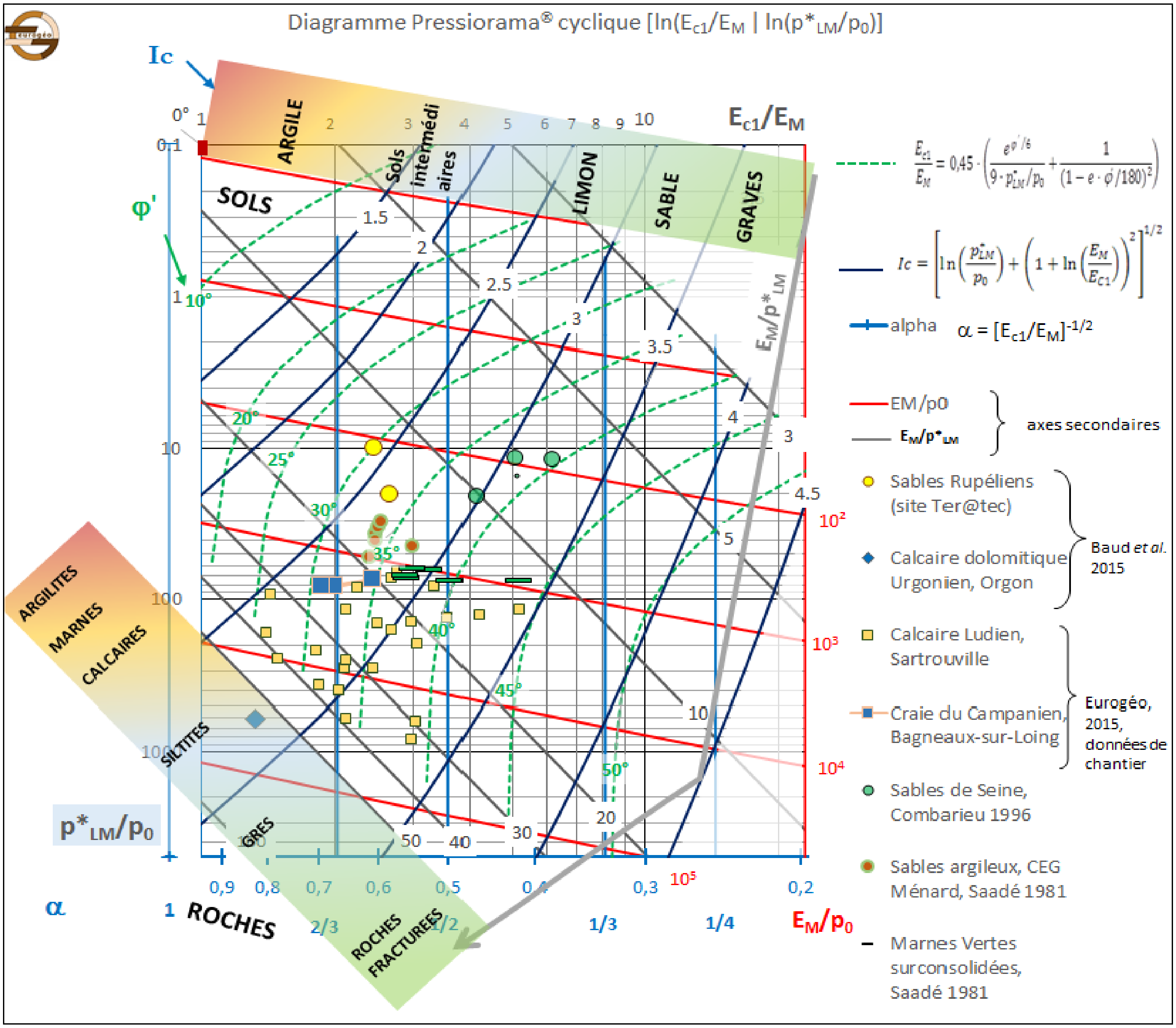


Figure 3. Diagramme Pressiorama® cyclique [ln(Ec1/EM | ln(p*LM/p0)].

The values of ϕ' plotted in the modified Pressiorama Chart are calculated with the following expression:

$$\phi' = 5.5\, ln\left(\frac{9}{\alpha^2}\ \frac{P_L^*}{p_0}\right)$$

with values of α calculated/inferred from the modified Pressiorama Chart.

Where this expression provides values of effective friction angle greater than a 45°, a maximum value of 45° should be assumed.

This expression was presented by J.P. Baud, in his publication *"Apport de L'Essai Cyclique a la Classification Pressiométrique des Sols et des Roches",* Journées Nationales de Geotechnique et de Géologie de l'Ingénieur, Nancy, 2016.

Shear strength parameters suggested in Table No. 3, are based on the guidelines provided by the *Pressiorama* and *Cyclique Pressiorama* charts. It should be noted that these guidelines are subject to changes, or improvements, as the correlations between pressuremeter parameters E_M, p'_L, and p_0 are being adjusted by ever increasing amount of field data. As such, care should be used when using these suggested parameters.

10. Soil Classification Index

Based on PMT testing procedures, soil behavior may be characterized as cohesive or frictional (cohesionless). Using the modified Pressiorama Chart, a Soil Classification Index, namely I_c, can be inferred with the following expression:

$$I_c = \left[\left(1 + log\left({P_L^*}/{p_0} \right) \right)^2 + \left(1 - log(\alpha) \right)^2 \right]^{1/2}$$

A minimum value of 1 would correspond to a cohesive soil, near its state of liquefaction. Whereas, a value of 4.5 would correspond to coarse gravel materials. A value of $I_c = 2.7$ would apply to a material which behaves mechanically as part frictional (drained for long-term loading conditions) and part cohesive (undrained for the short-term loading conditions). In general, Soil Type Behaviors corresponding to values of the Classification Index I_C are listed as:

1.0 to 1.5	Clays
1.5 to 2.5	Clay-Silt mixes
2.5 to 3.0	Silts
3.0 to 3.5	Sands
3.5 to 4.0	Gravels, and
4.0 to 4.5	Weathered Rocks

Appendix Three

Calibration Data

Calibration Date:	February 21, 2023
Probe Designation:	B 504
Calibration Record No.:	I
Length of Tubing:	180 feet
Calibrated by:	G.S.

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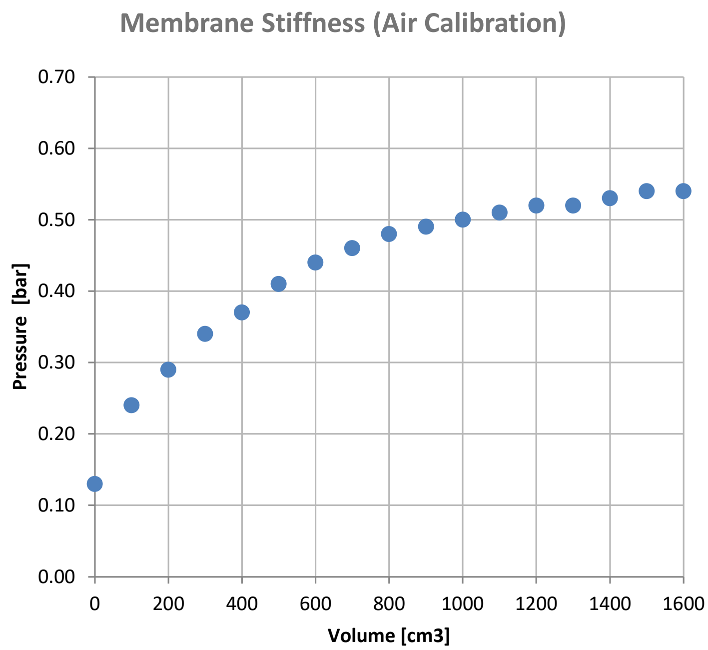
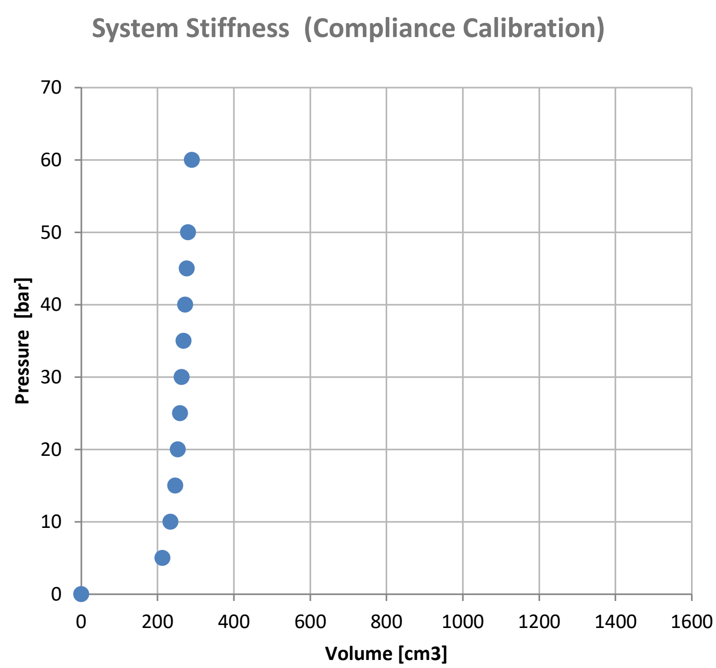
Membrane stiffness calibration

Pressure [bar]	Volume cm³
0.13	0
0.24	100
0.29	200
0.34	300
0.37	400
0.41	500
0.44	600
0.46	700
0.48	800
0.49	900
0.50	1000
0.51	1100
0.52	1200
0.52	1300
0.53	1400
0.54	1500
0.54	1600



Volume calibration

Pressure [bar]	Volume cm³
0	0.0
5	213.0
10	234.0
15	246.0
20	253.0
25	258.7
30	263.5
35	268.1
40	272.4
45	276.5
50	280.3
60	289.7
Reload Cal. Data	
25	262.9
50	280.8



Appendix D

Cone Penetration Test Results

PRESENTATION OF SITE INVESTIGATION RESULTS

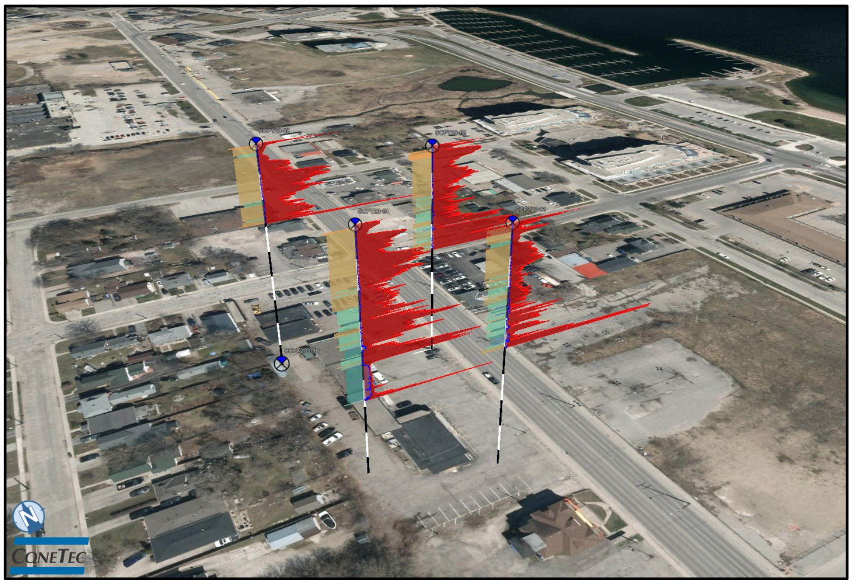
126-140 Bradford Street Barrie

Prepared for:

GHD Group

ConeTec Job No: 23-05-25396

Project Start Date: 27-Feb-2023
Project End Date: 27-Feb-2023
Report Date: 07-Mar-2023



Prepared by:

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www.conetec.com
www.conetecdataservices.com

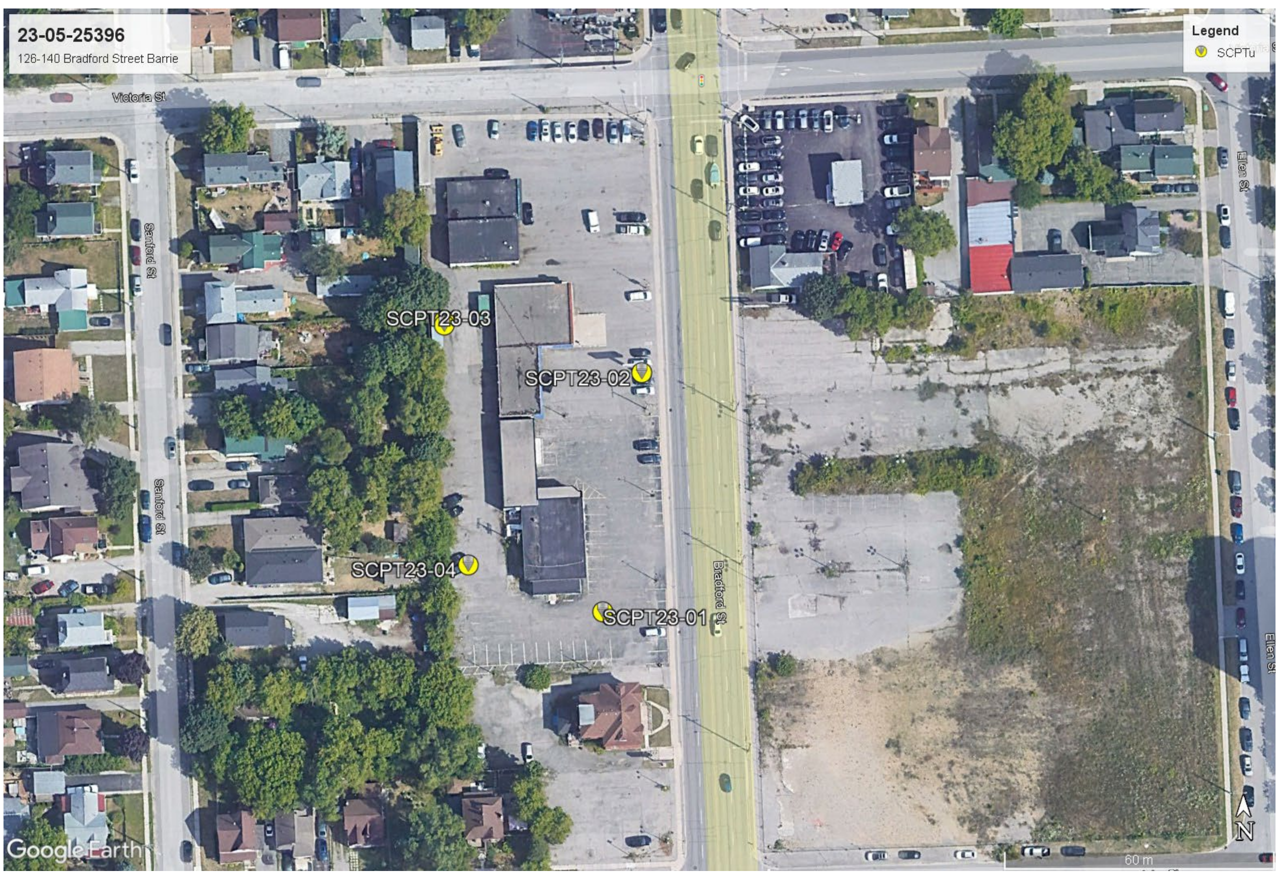
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for GHD Group at 126-140 Bradford Street in Barrie, ON. The program consisted of 5 seismic cone penetration tests (SCPTu). Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information

Project	
Client	GHD Group
Project	126-140 Bradford Street Barrie
ConeTec project number	23-05-25396

An aerial overview from Google Earth including the SCPTu test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
SCPTu	Consumer grade GPS	26917

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
824:T1000F10U35	824	15	225	1000	10	35
Cone 824 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	• Standard plots with dual pore pressures scales • Advanced plots with Su, OCR, Phi, DR, and K_o • Seismic plots with Vs and Gmax • Soil Behaviour Type (SBT) scatter plots

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

Limitations

3rd Party Disclaimer

This report titled “126-140 Bradford Street Barrie”, referred to as the (“Report”), was prepared by ConeTec for GHD Group. The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by GHD Group to collect and provide the raw data (“Data”) which is included in this report titled “126-140 Bradford Street Barrie”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

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

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

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

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

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

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

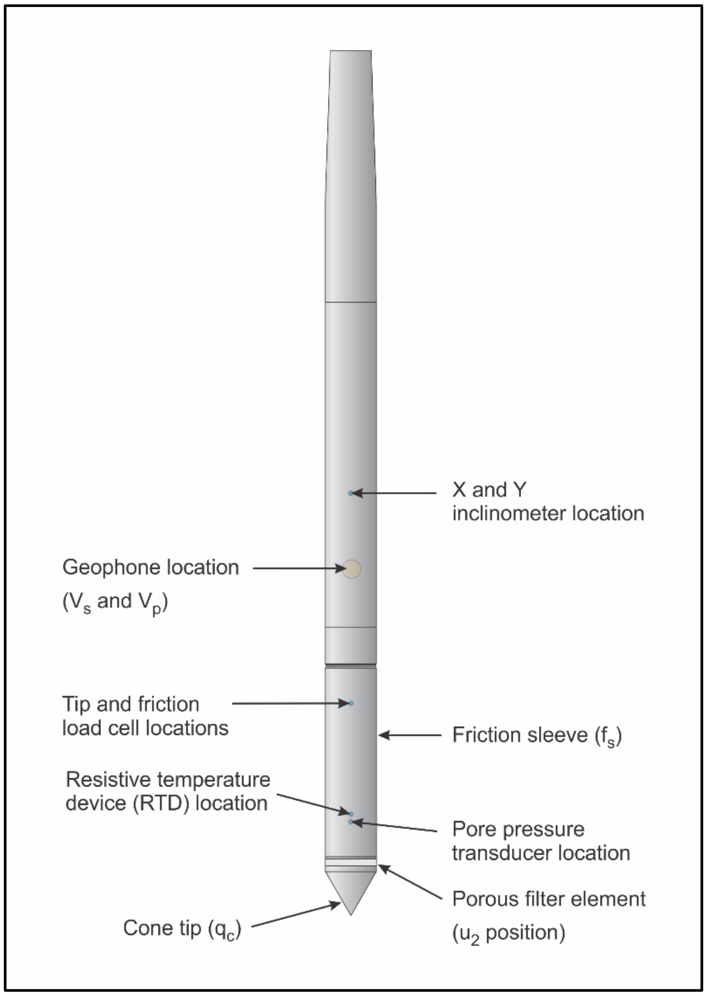


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

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

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

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

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

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

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

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

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

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

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

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

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

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

References

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

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

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

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

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

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

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

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

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

ConeTec's 15 cm^2 piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves; however, it is often affected by the compression wave travelling through the cone rods.

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

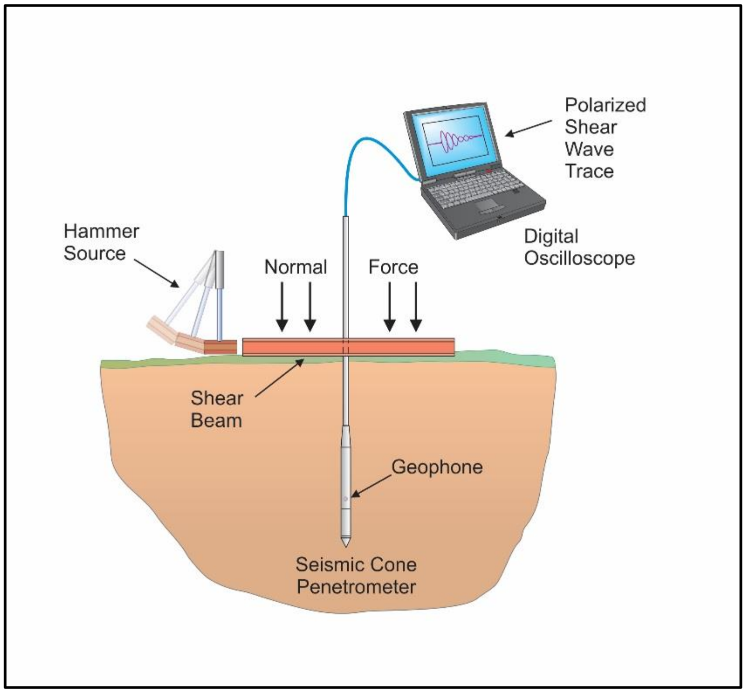


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

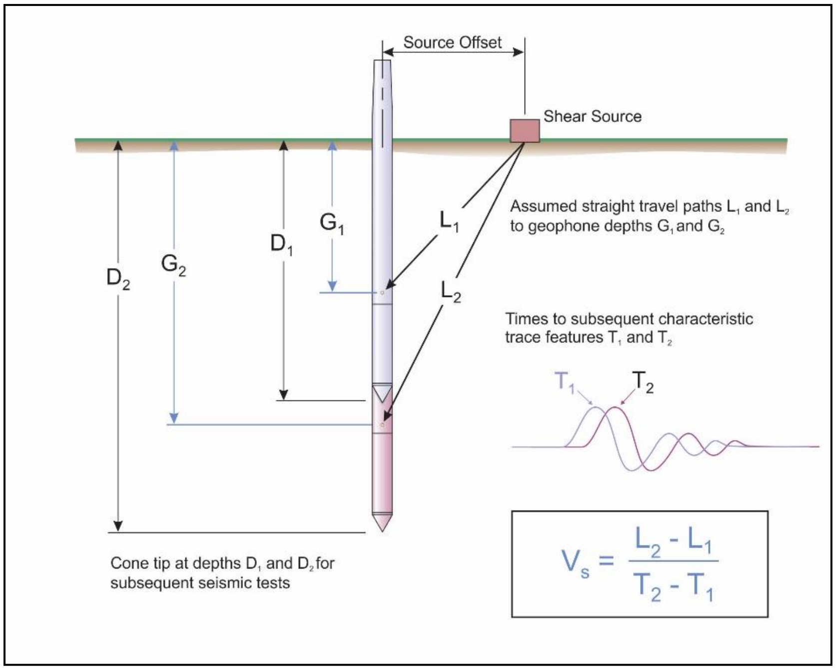


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the ray path is defined as the straight-line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

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

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

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

The average shear wave velocity to a depth of thirty meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al. (2012).

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer traveltimes})}$$

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

References

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

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

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, “National shear wave measurement guidelines for Canadian seismic site assessment”, GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

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

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

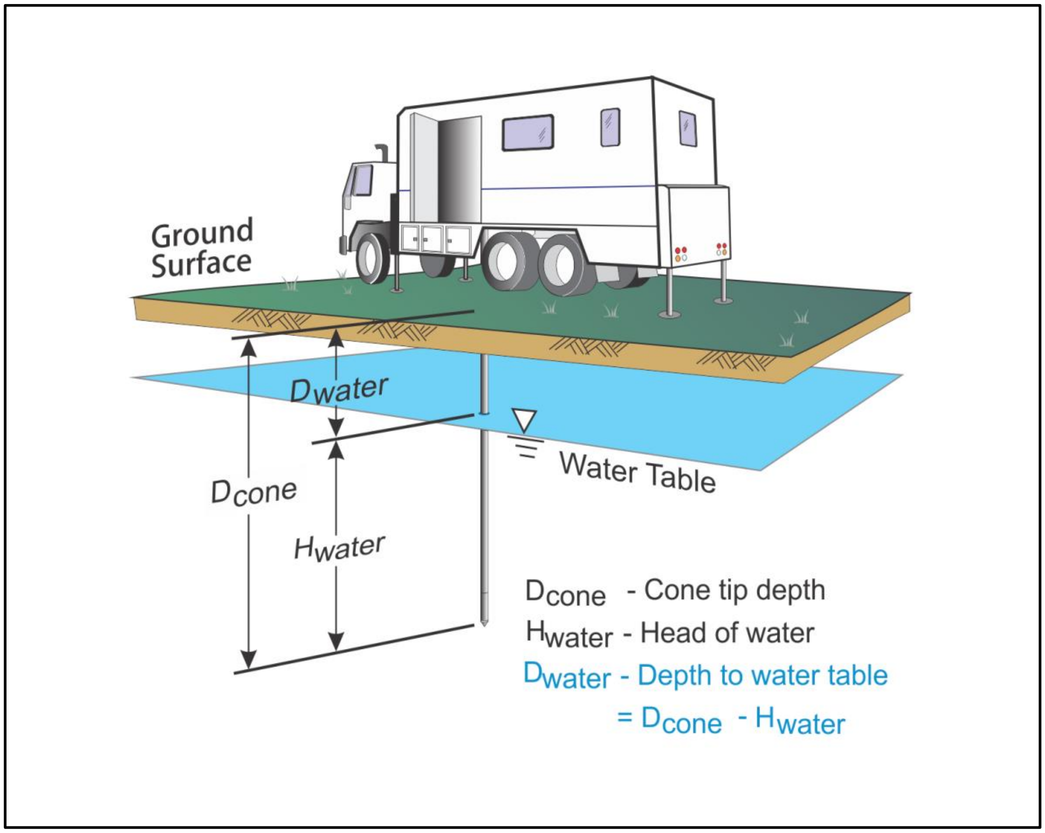


Figure PPD-1. Pore pressure dissipation test setup

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

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

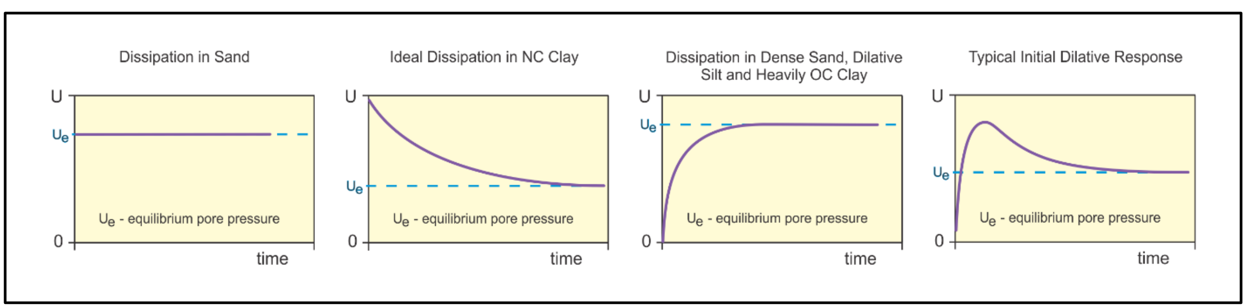


Figure PPD-2. Pore pressure dissipation curve examples

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

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T*	is the dimensionless time factor (Table Time Factor)
a	is the radius of the cone
I_r	is the rigidity index
t	is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

References

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Cone Penetration Test Plots with Dual Pore Pressure Scales
- Advanced Cone Penetration Test Plots with Su(Nkt), OCR (JS1978), Phi, DR, and K_o (Mayne)
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 23-05-25396
Client: GHD Group
Project: 126-140 Bradford Street Barrie
Start Date: 27-Feb-2023
End Date: 27-Feb-2023

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
SCPT23-01	23-05-25396_SP01	27-Feb-2023	824:T1000F10U35	15	2.0	20.225	4914943	604040	
SCPT23-02	23-05-25396_SP02	27-Feb-2023	824:T1000F10U35	15	2.2	20.250	4914996	604048	
SCPT23-03	23-05-25396_SP03	27-Feb-2023	824:T1000F10U35	15	2.9	14.375	4915006	604004	
SCPT23-03B	23-05-25396_SP03B	27-Feb-2023	824:T1000F10U35	15	2.9	14.575	4915007	604004	3
SCPT23-04	23-05-25396_SP04	27-Feb-2023	824:T1000F10U35	15	2.4	28.900	4914953	604010	

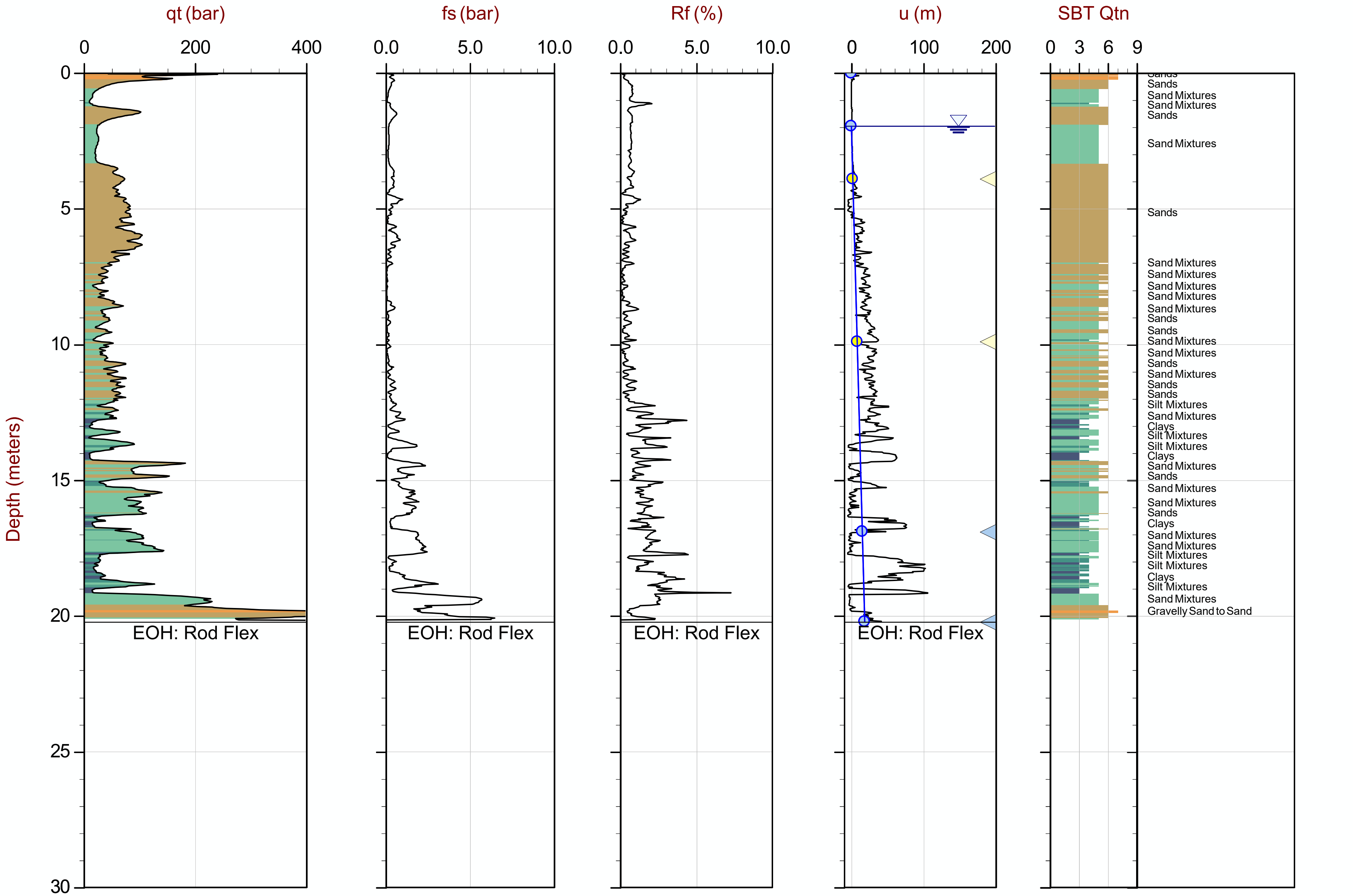
1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with consumer grade GPS equipment. Datum: NAD 1983 / UTM Zone 17 North.
3. Offset of sounding SCPT23-03 to attempt pushing beyond the original refusal depth. Only one shear wave velocity interval was measured at end of hole.
The complete shear wave velocity profile was collected at the original location, SCPT23-03.

ConeTec

GHD

Job No: 23-05-25396
Date: 2023-02-27 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914943m E: 604040m
Sheet No: 1 of 1

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

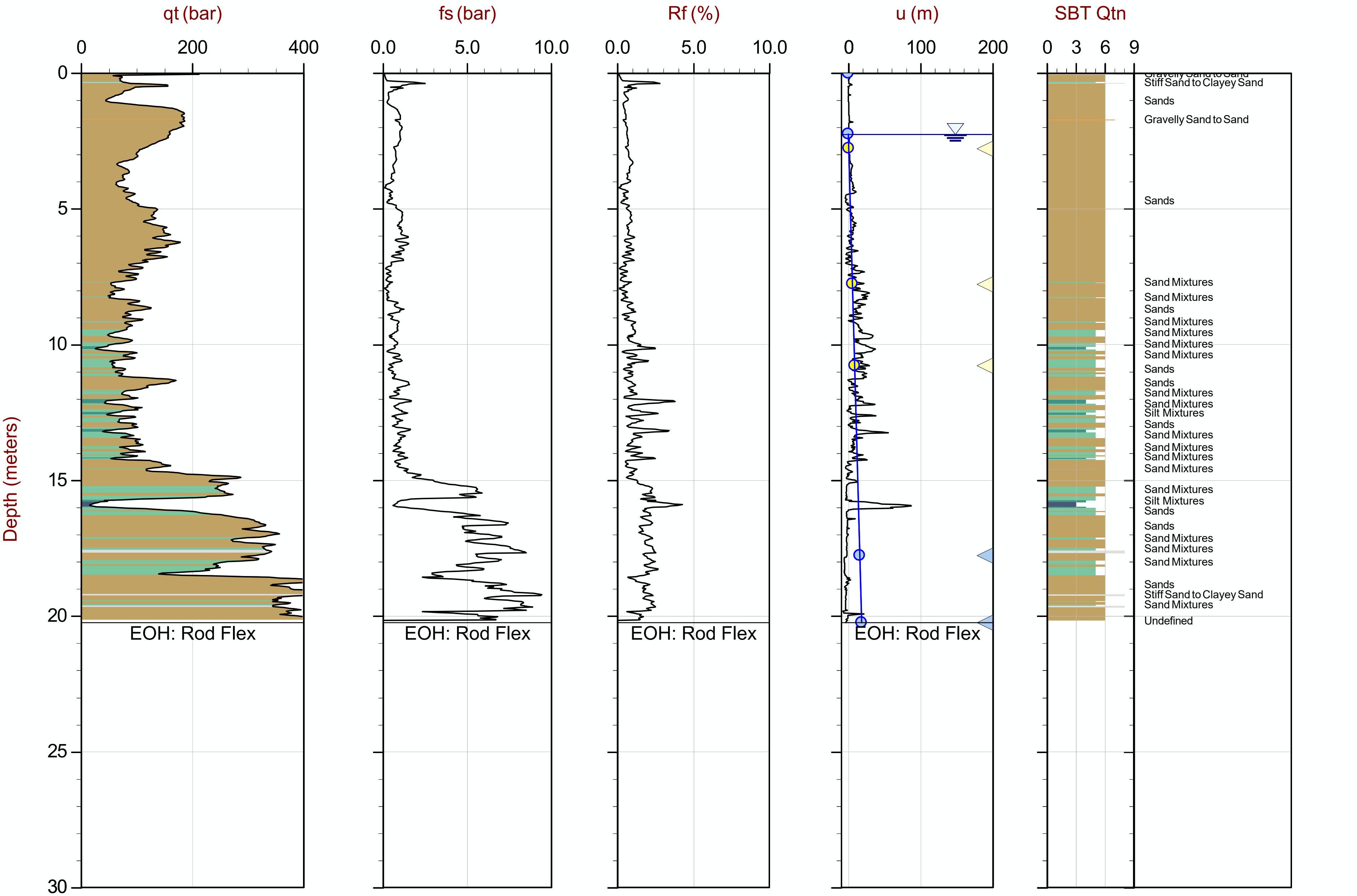
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35



Max Depth: 20.250 m / 66.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914996m E:604048m
Sheet No: 1 of 1

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

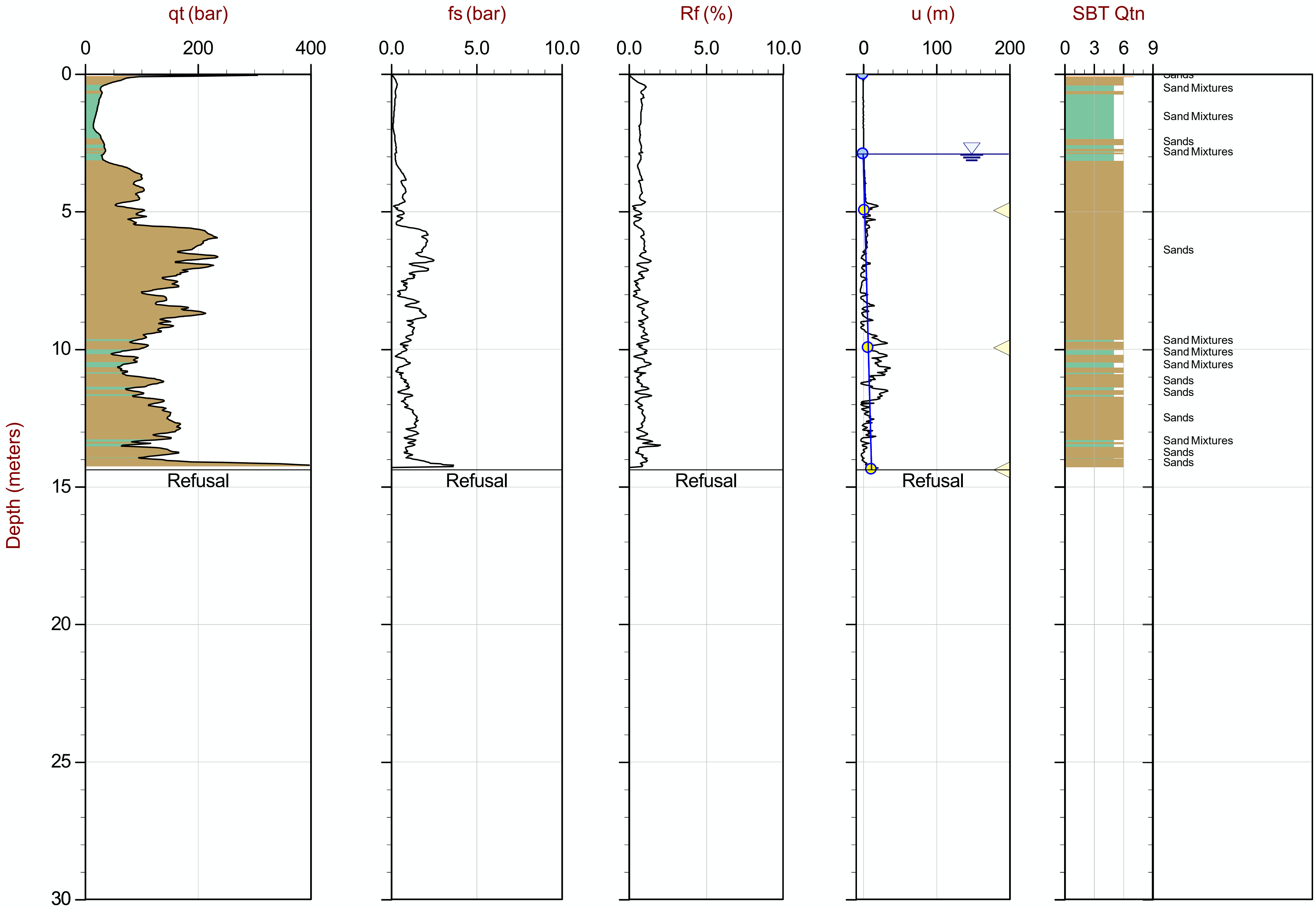
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915006m E:604004m
Sheet No: 1 of 1

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

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

ConeTec

GHD

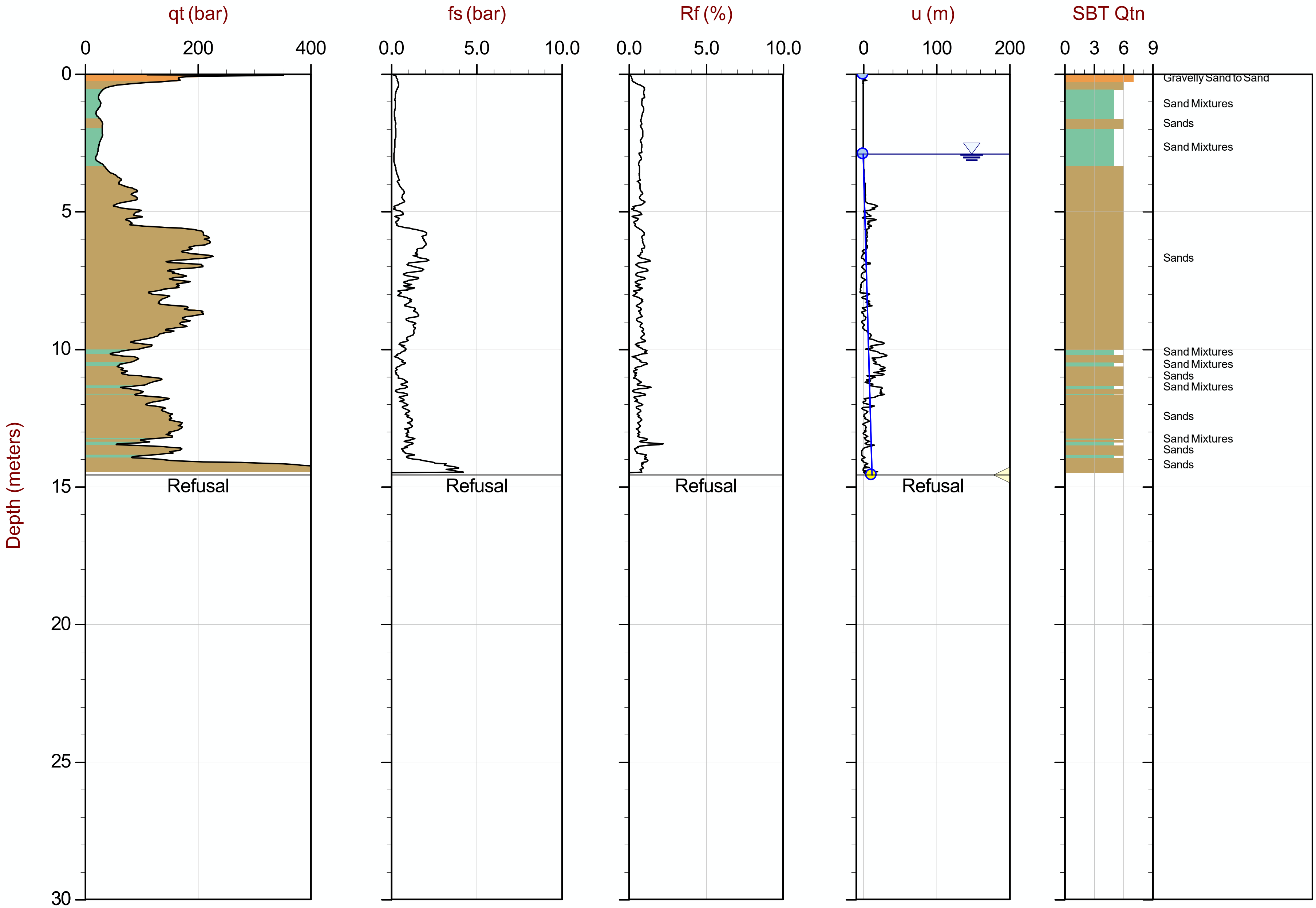
Job No: 23-05-25396

Date: 2023-02-27 11:00

Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B

Cone: 824:T1000F10U35



Max Depth: 14.575 m / 47.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915007m E:604004m
Sheet No: 1 of 1

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

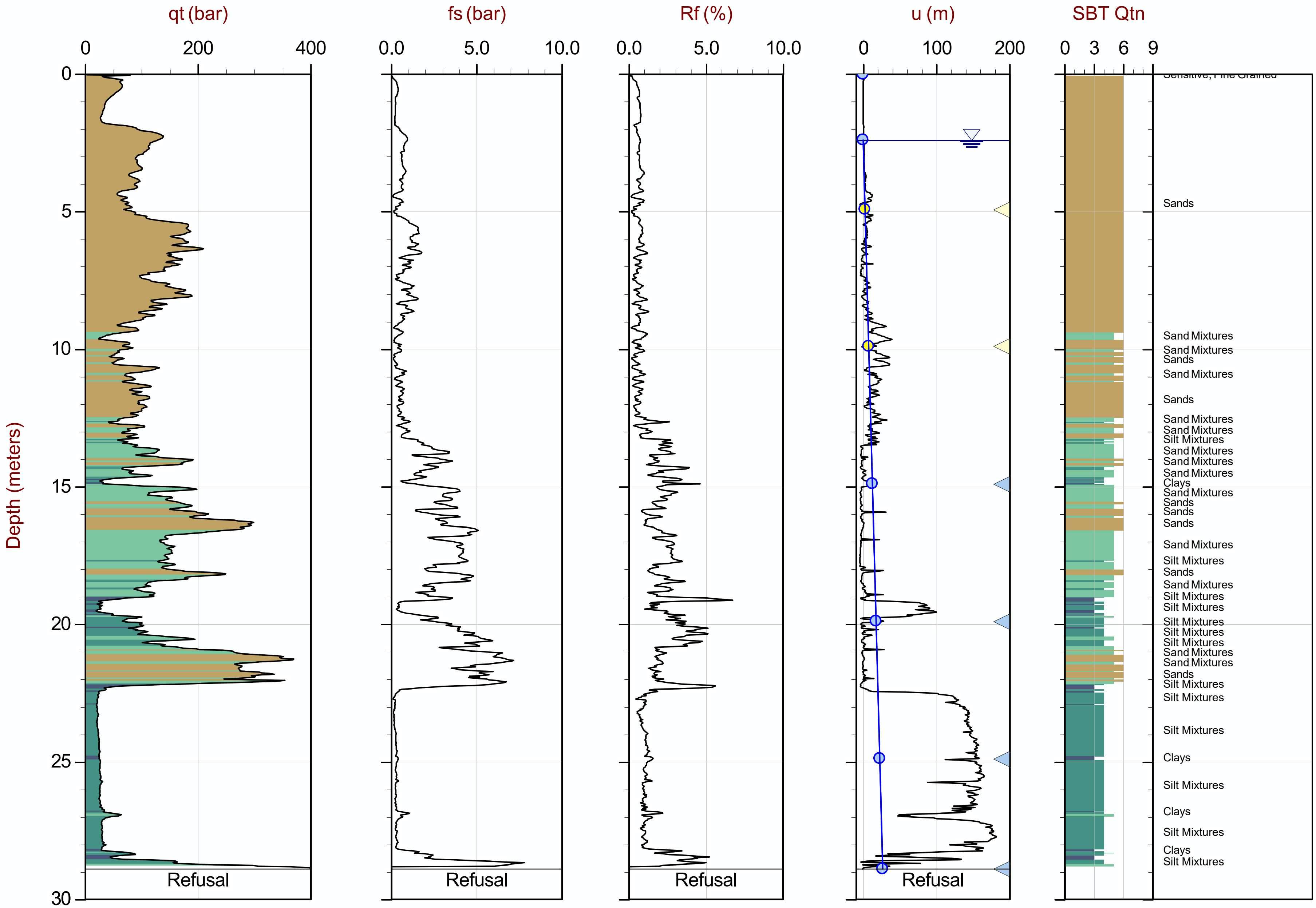
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35



Max Depth: 28.900 m / 94.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914953m E:604010m
Sheet No: 1 of 1

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

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

Cone Penetration Test Plots with Dual Pore Pressure Scales

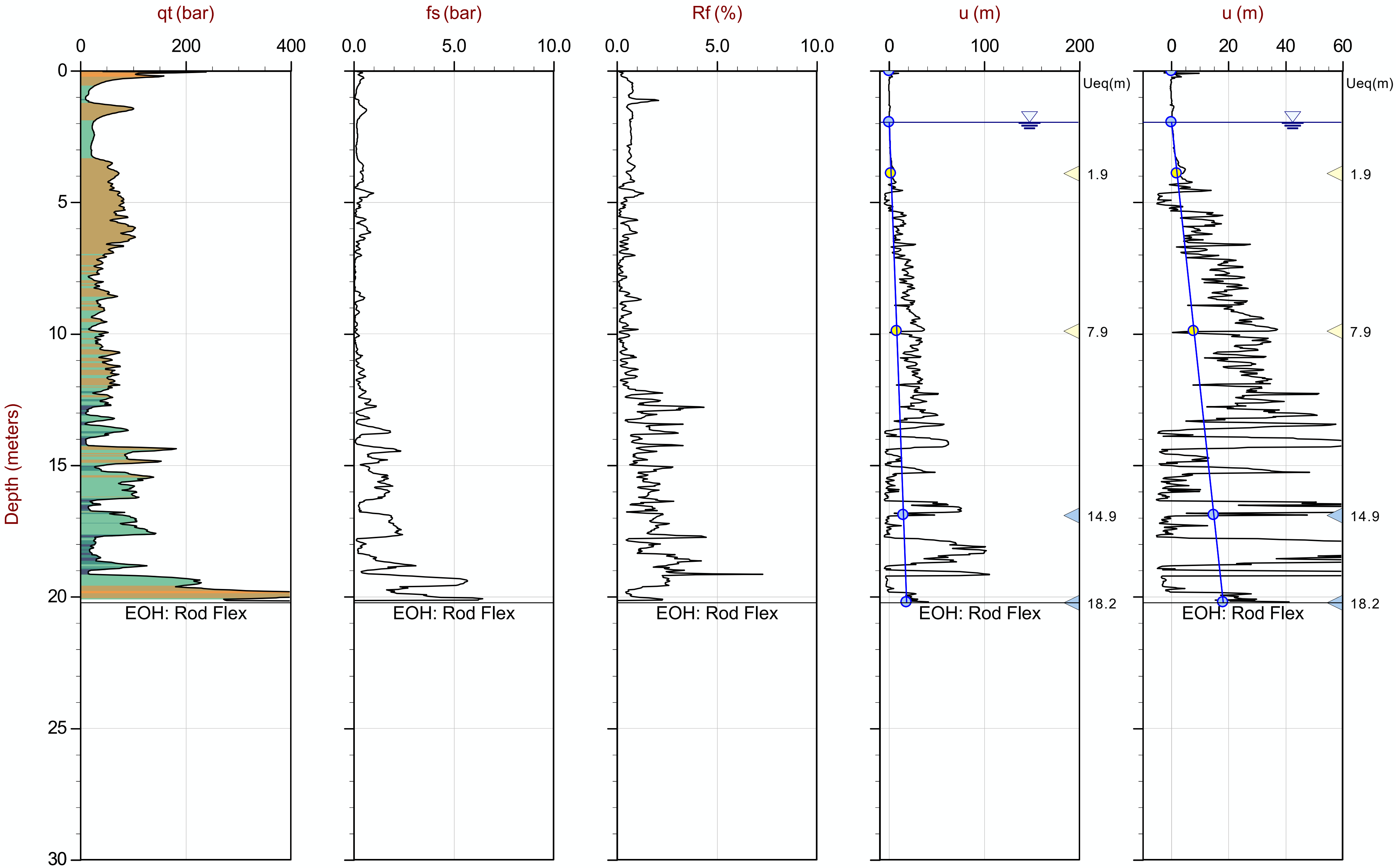
ConeTec

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

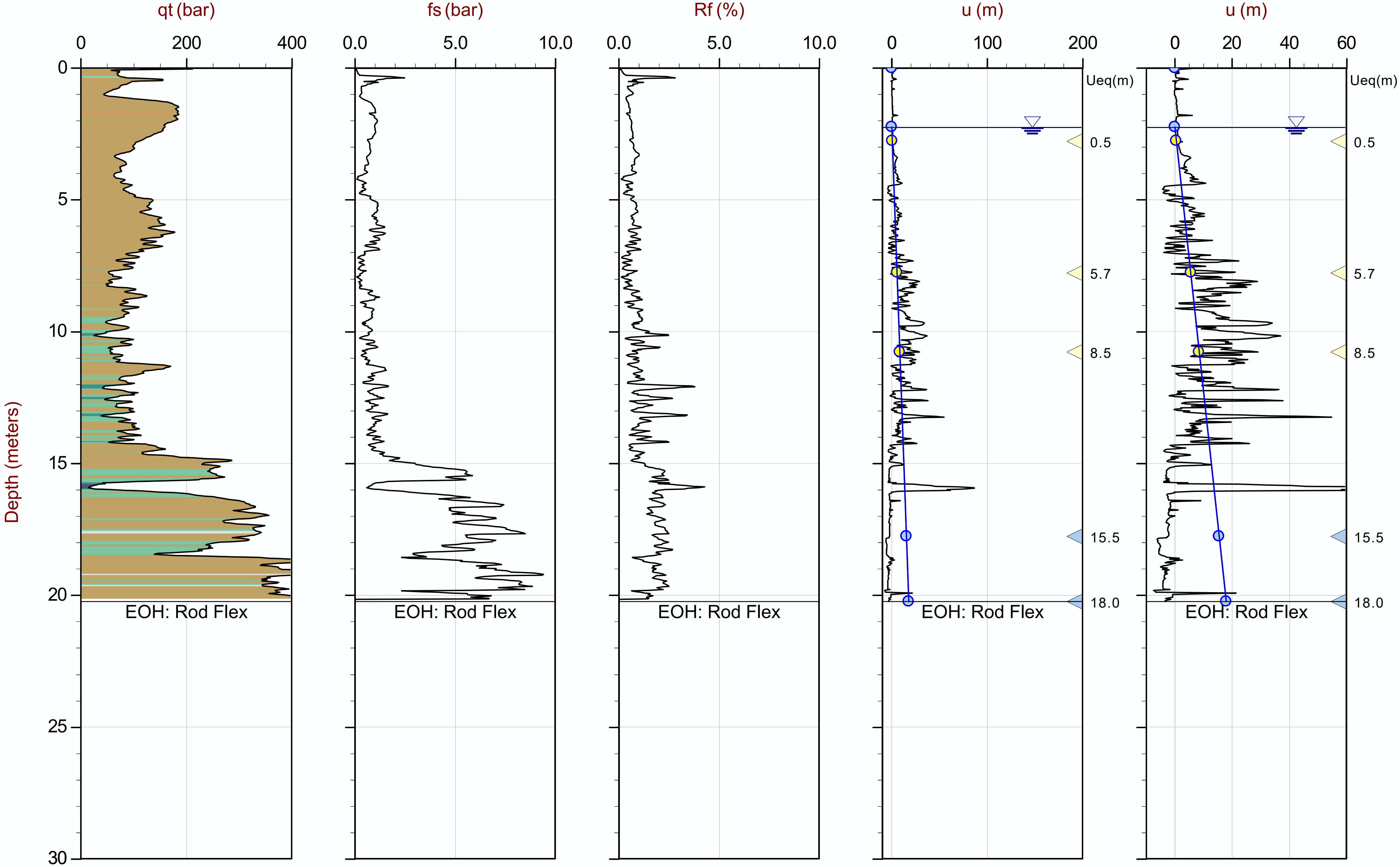
File: 23-05-25396_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914943m E: 604040m
Sheet No: 1 of 1

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

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

CONETEC	GHD	Job No: 23-05-25396	Sounding: SCPT23-02
		Date: 2023-02-27 07:41	Cone: 824:T1000F10U35
		Site: 126-140 Bradford St, Barrie	



Max Depth: 20.250 m / 66.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914996m E:604048m
Sheet No: 1 of 1

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

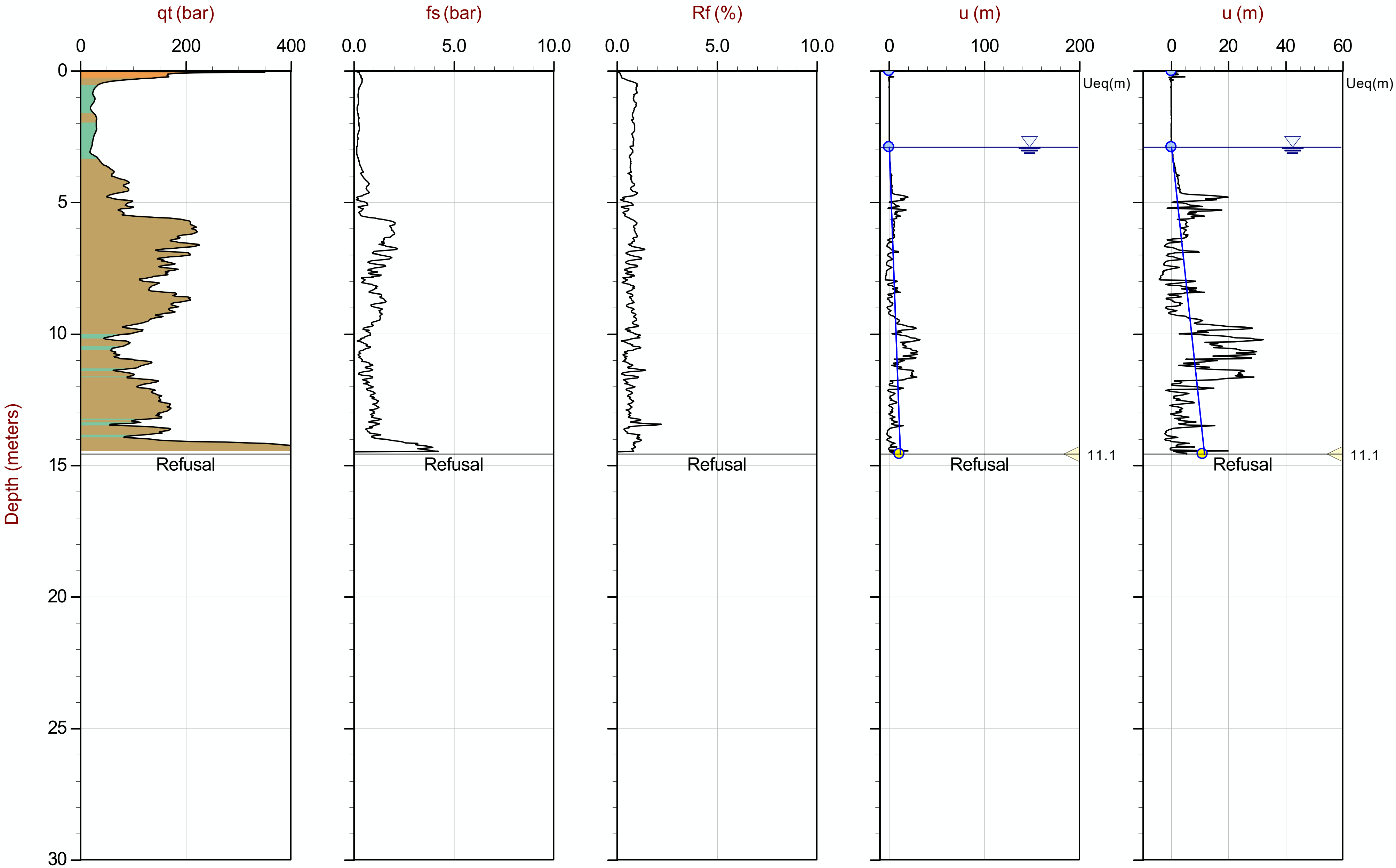
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 11:00
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B
Cone: 824:T1000F10U35



Max Depth: 14.575 m / 47.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915007m E:604004m
Sheet No: 1 of 1

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

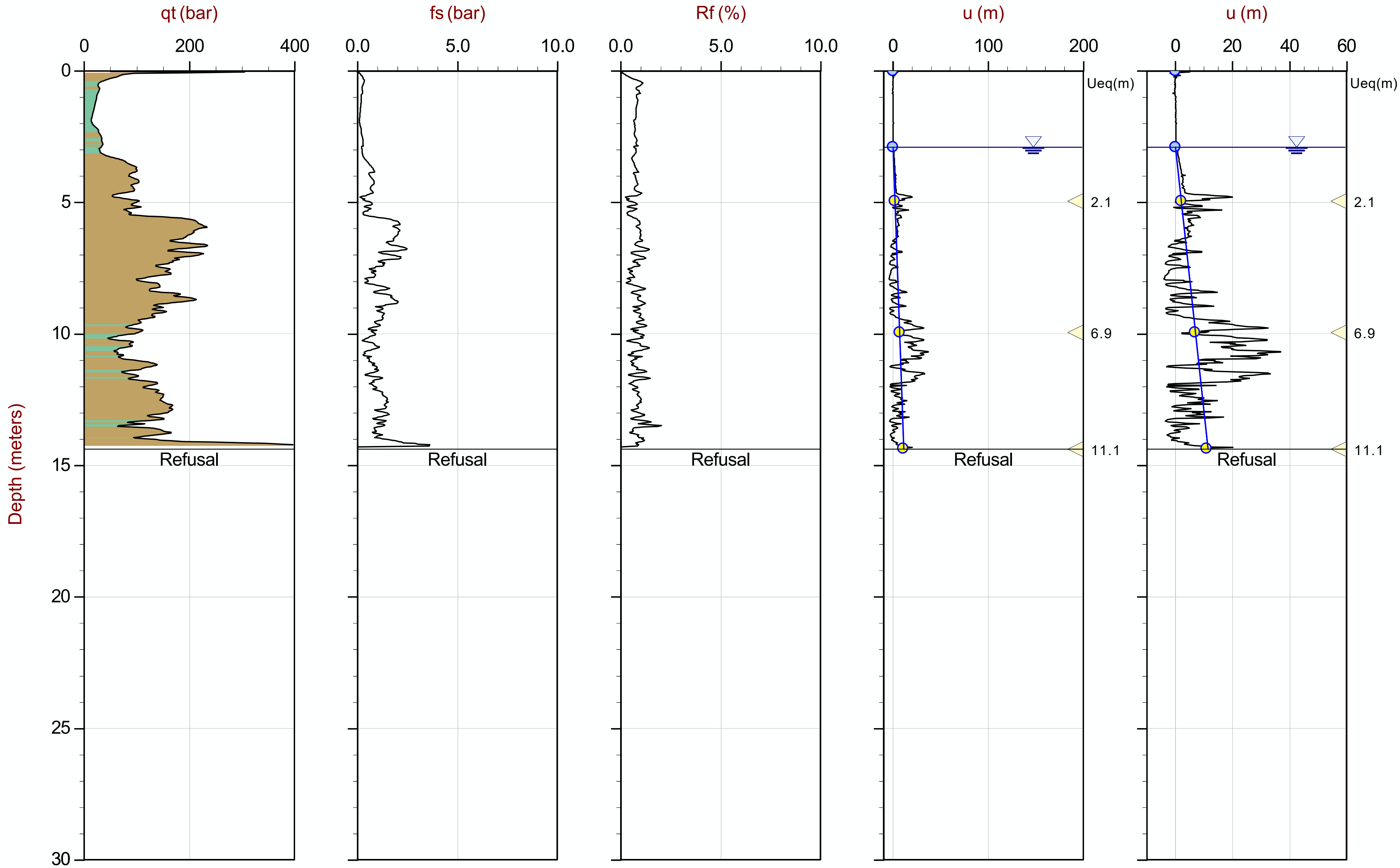
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

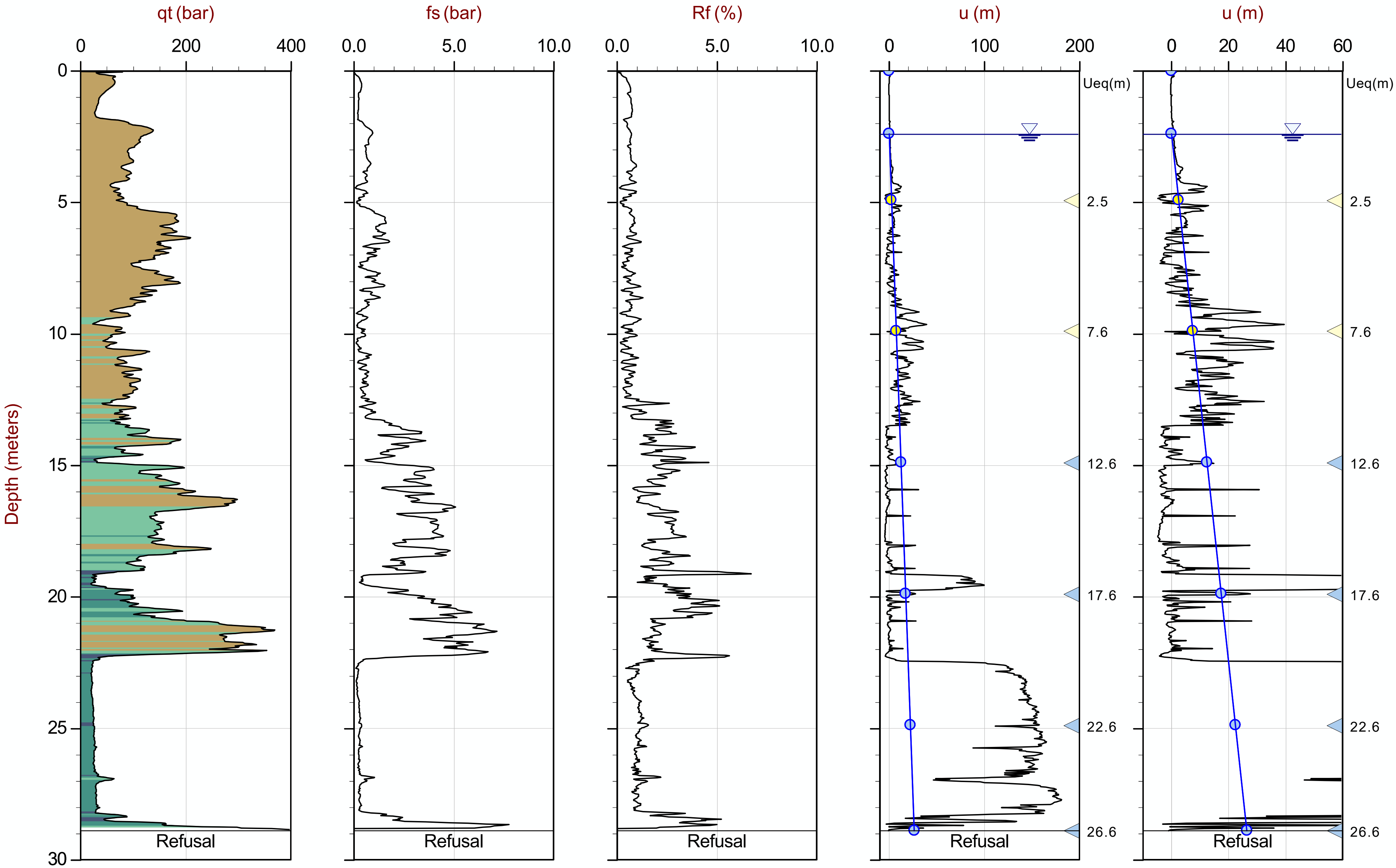
File: 23-05-25396_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915006m E:604004m
Sheet No: 1 of 1

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

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

CONETEC GHD	Job No: 23-05-25396 Date: 2023-02-27 12:10 Site: 126-140 Bradford St, Barrie	Sounding: SCPT23-04 Cone: 824:T1000F10U35



Max Depth: 28.900 m / 94.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914953m E: 604010m
Sheet No: 1 of 1

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

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

Advanced Cone Penetration Plots with Su(Nkt), OCR (JS1978), Phi, Dr, and Ko(Mayne)

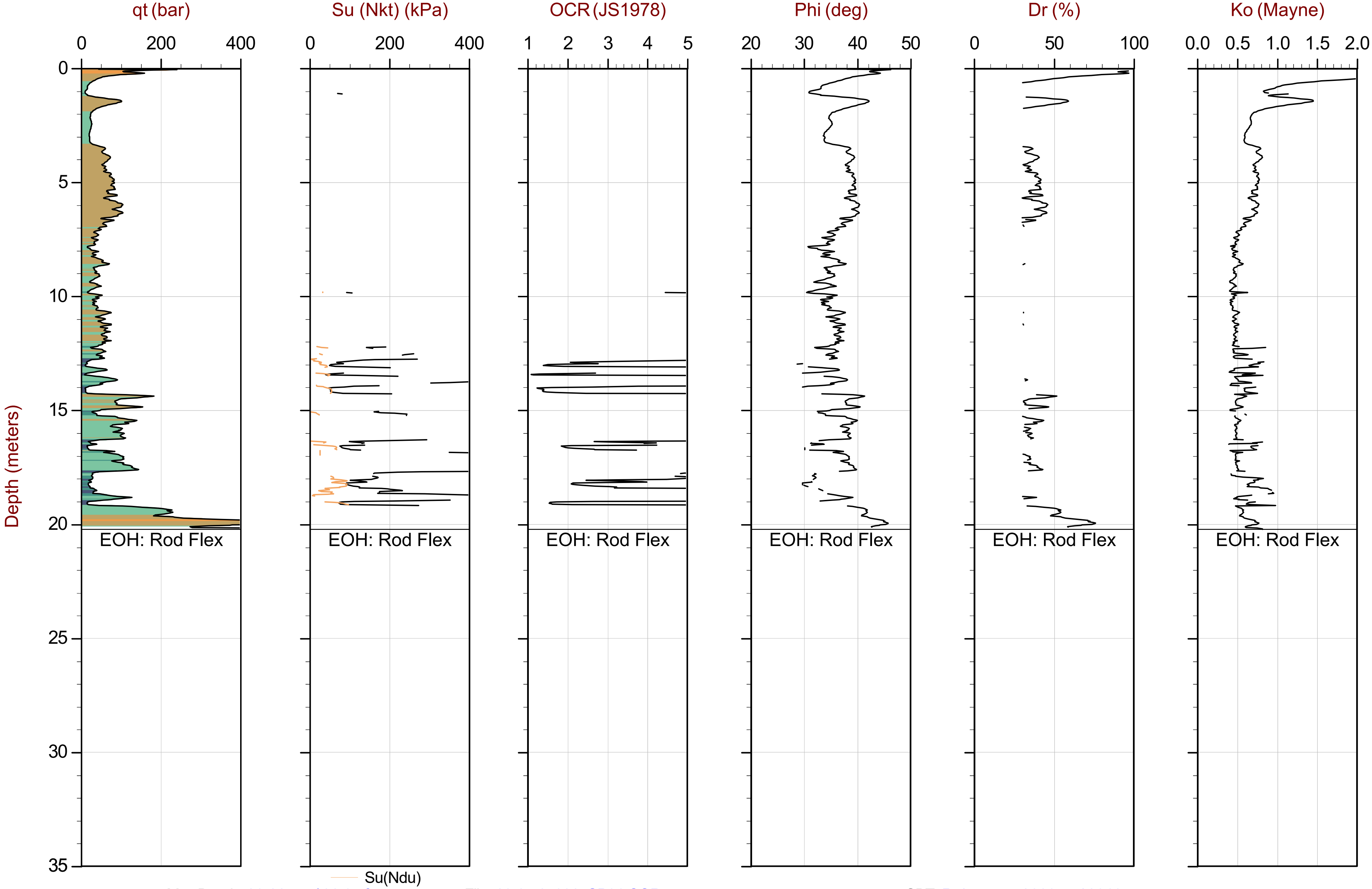
ConeTec

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914943m E:604040m
Sheet No: 1 of 1

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

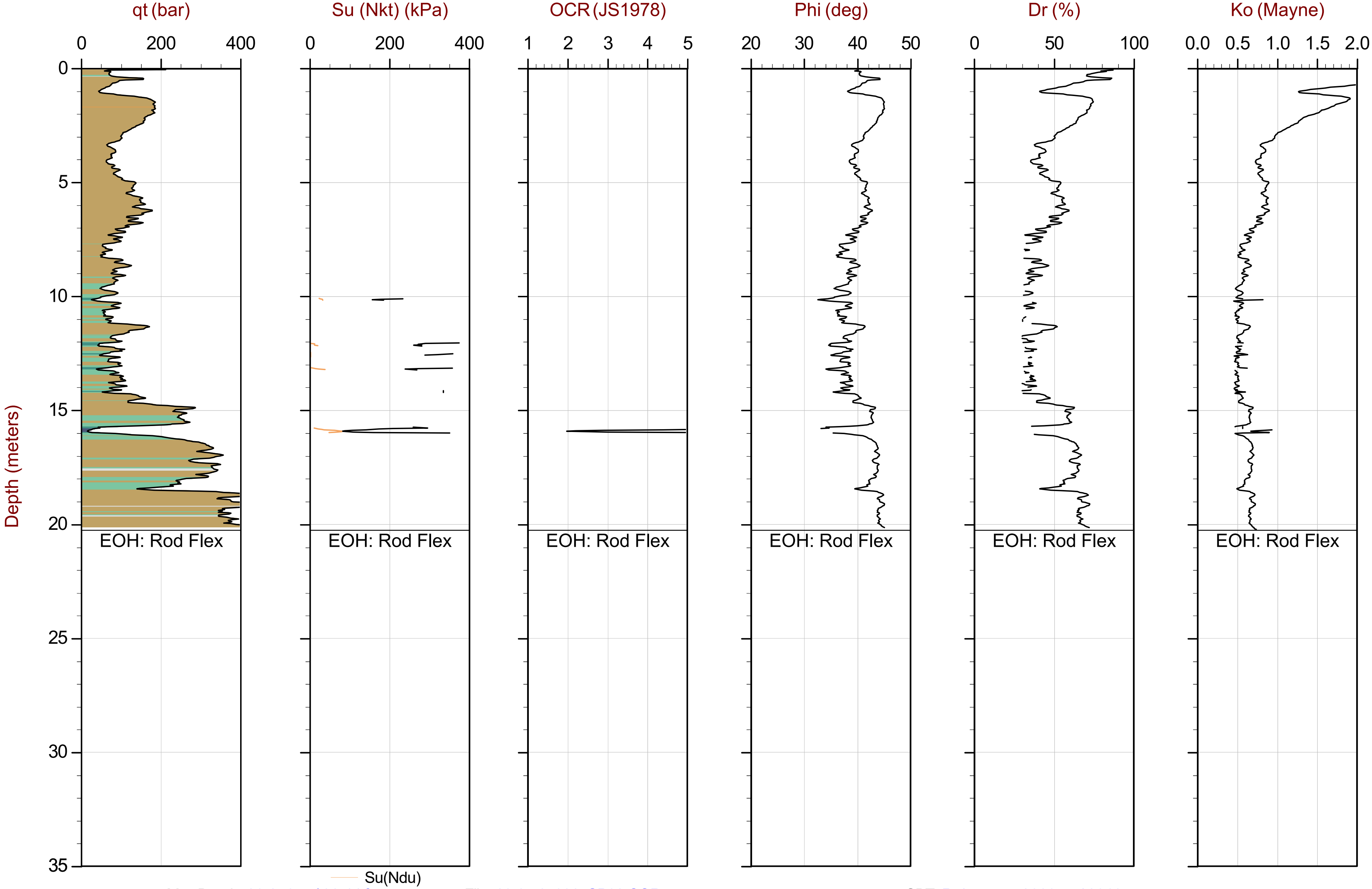
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

GHD

Job No: 23-05-25396
Date: 2023-02-27 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35



Max Depth: 20.250 m / 66.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914996m E:604048m
Sheet No: 1 of 1

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

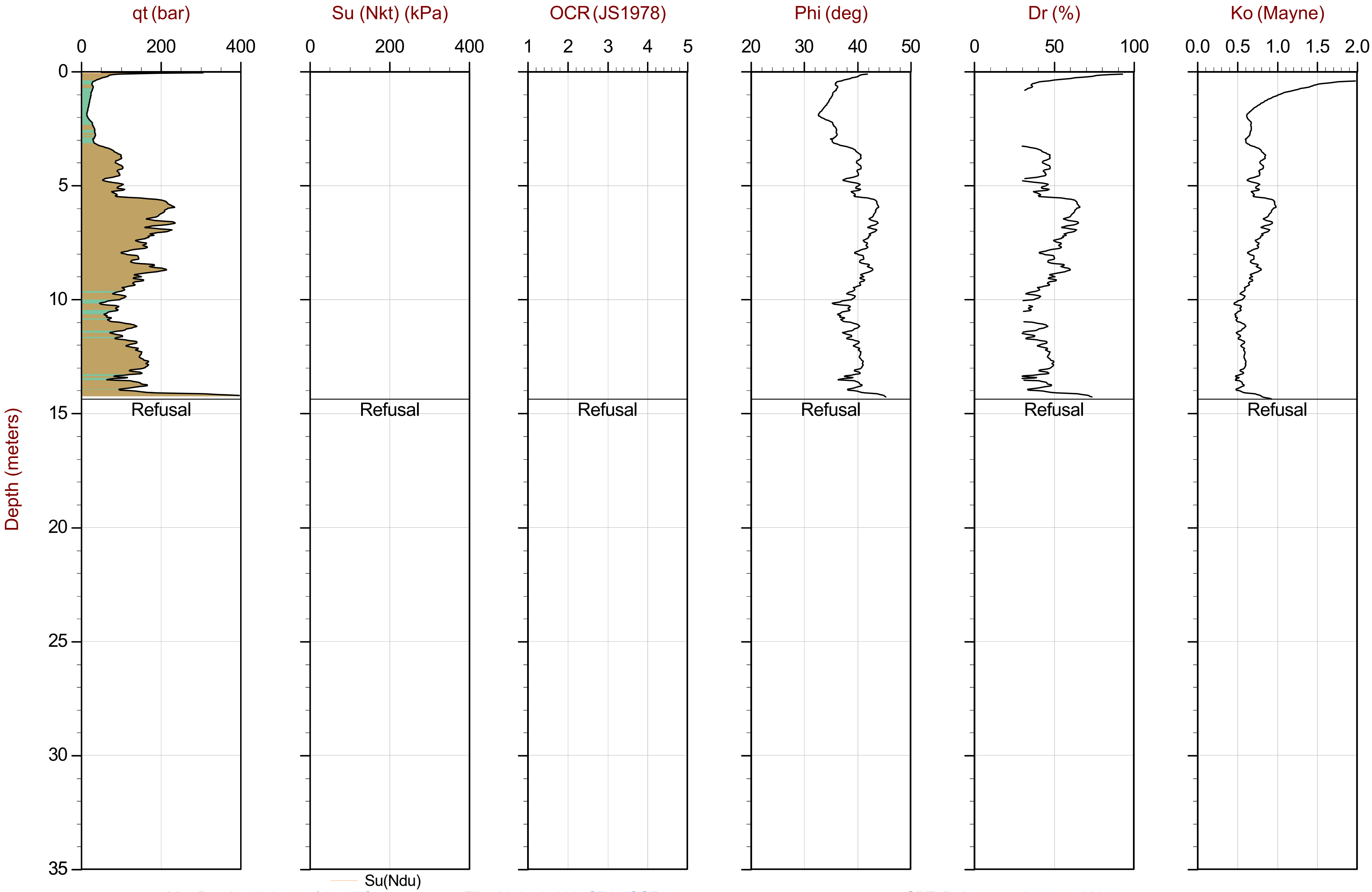
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915006m E:604004m
Sheet No: 1 of 1

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

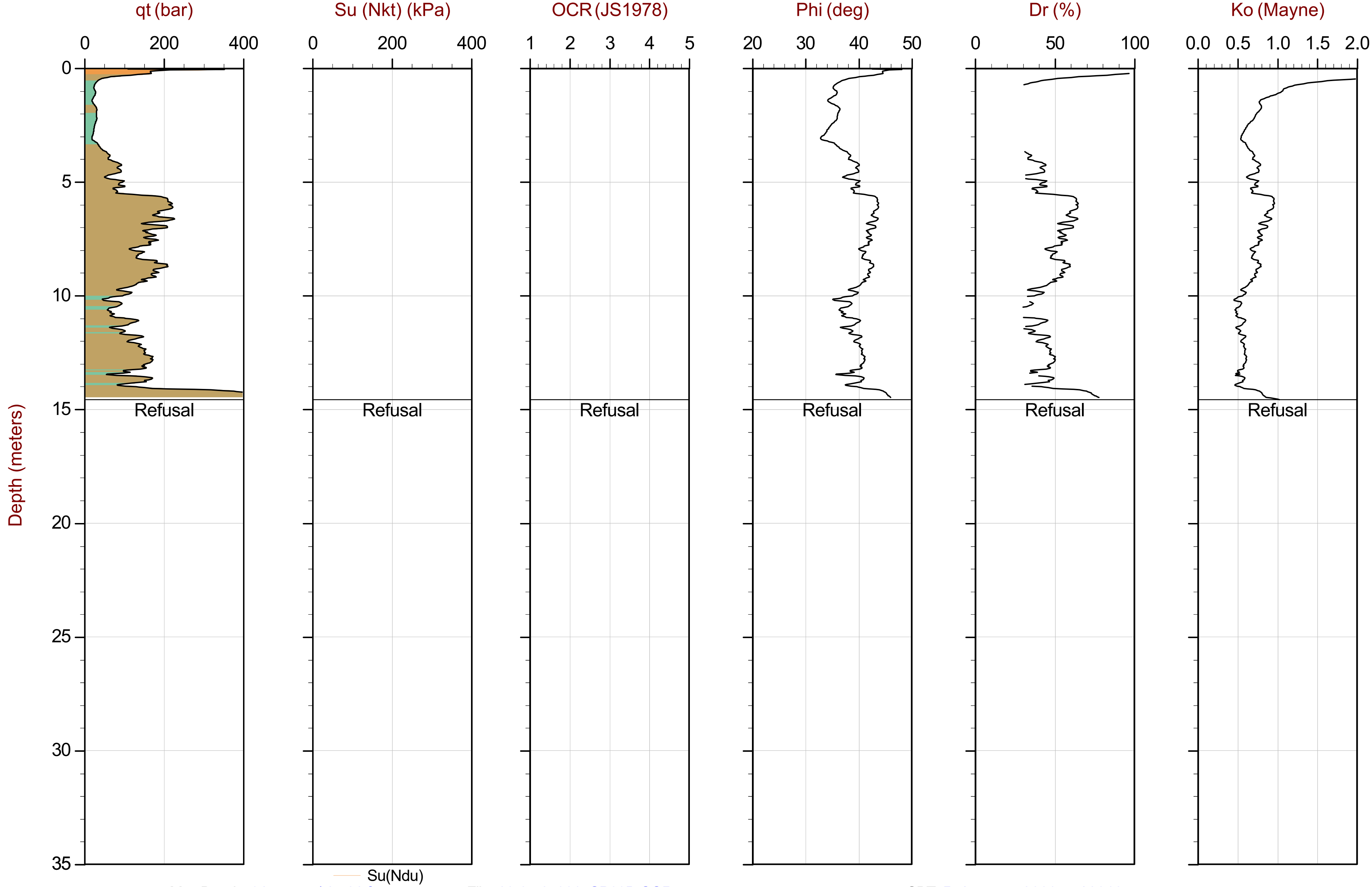
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 11:00
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B
Cone: 824:T1000F10U35



Max Depth: 14.575 m / 47.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4915007m E: 604004m
Sheet No: 1 of 1

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

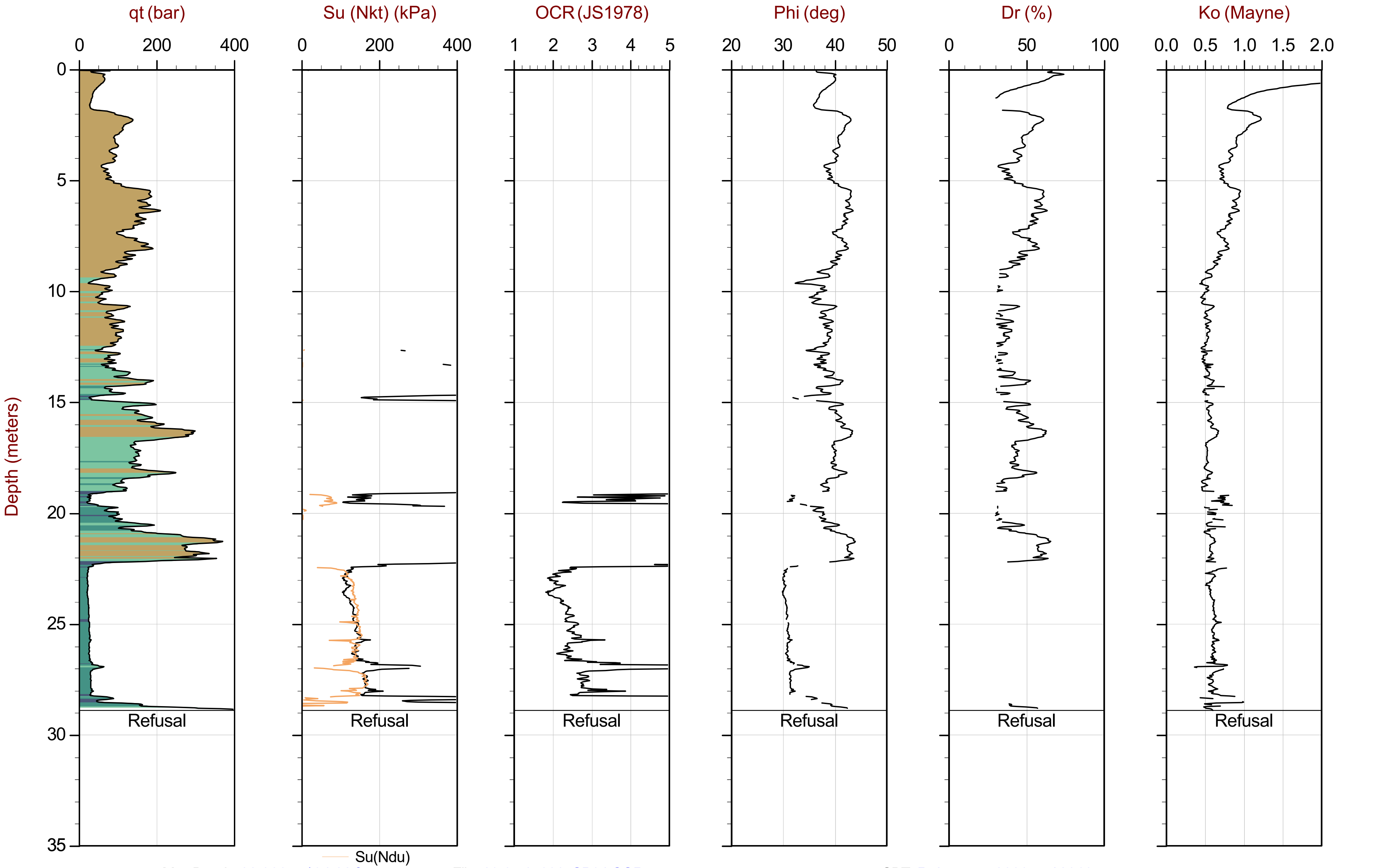
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

GHD

Job No: 23-05-25396
Date: 2023-02-27 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35



Max Depth: 28.900 m / 94.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914953m E:604010m
Sheet No: 1 of 1

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

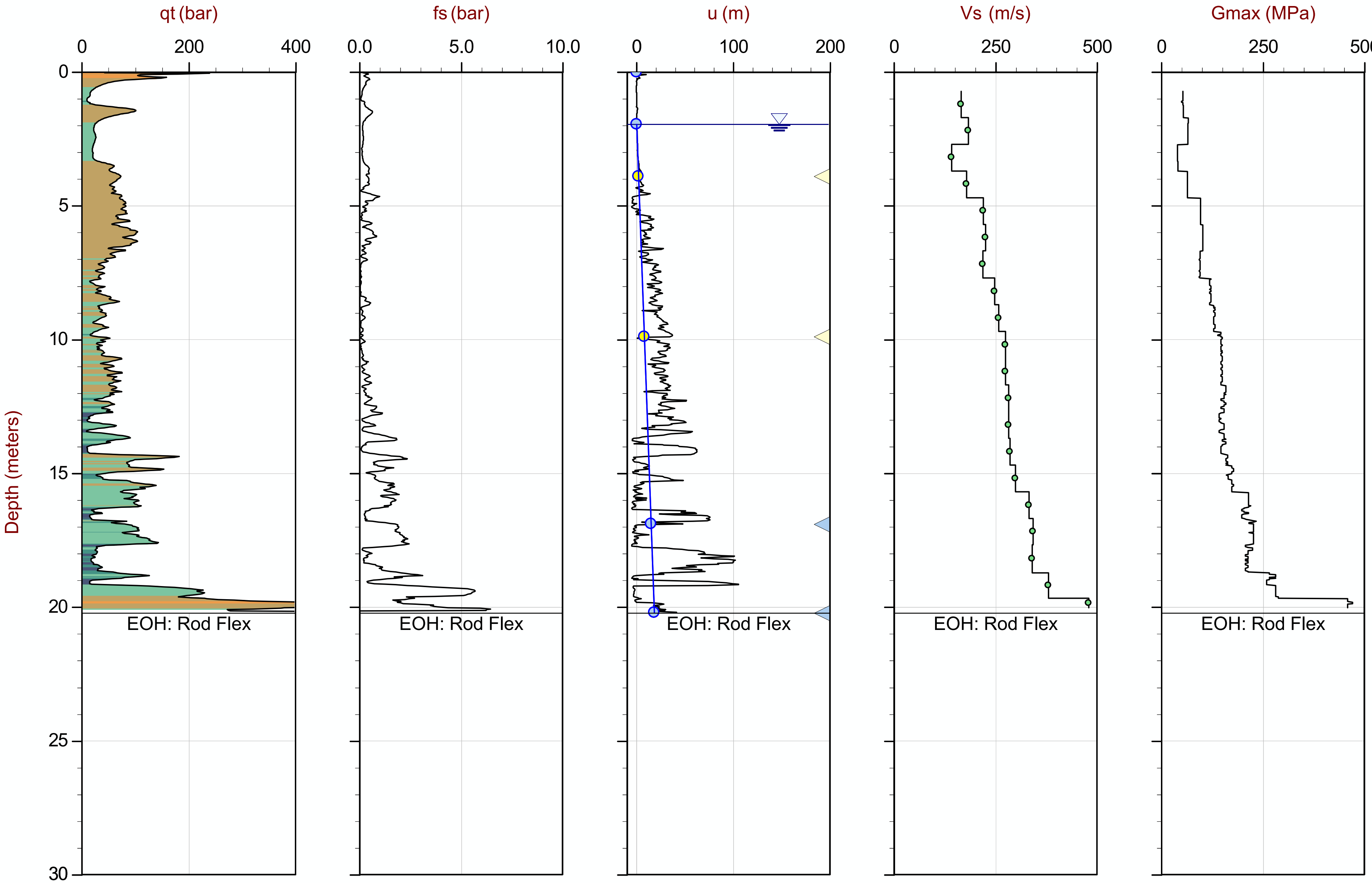
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots

GHD

Job No: 23-05-25396
Date: 2023-02-27 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35



Max Depth: 20.225 m / 66.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914943m E:604040m
Sheet No: 1 of 1

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

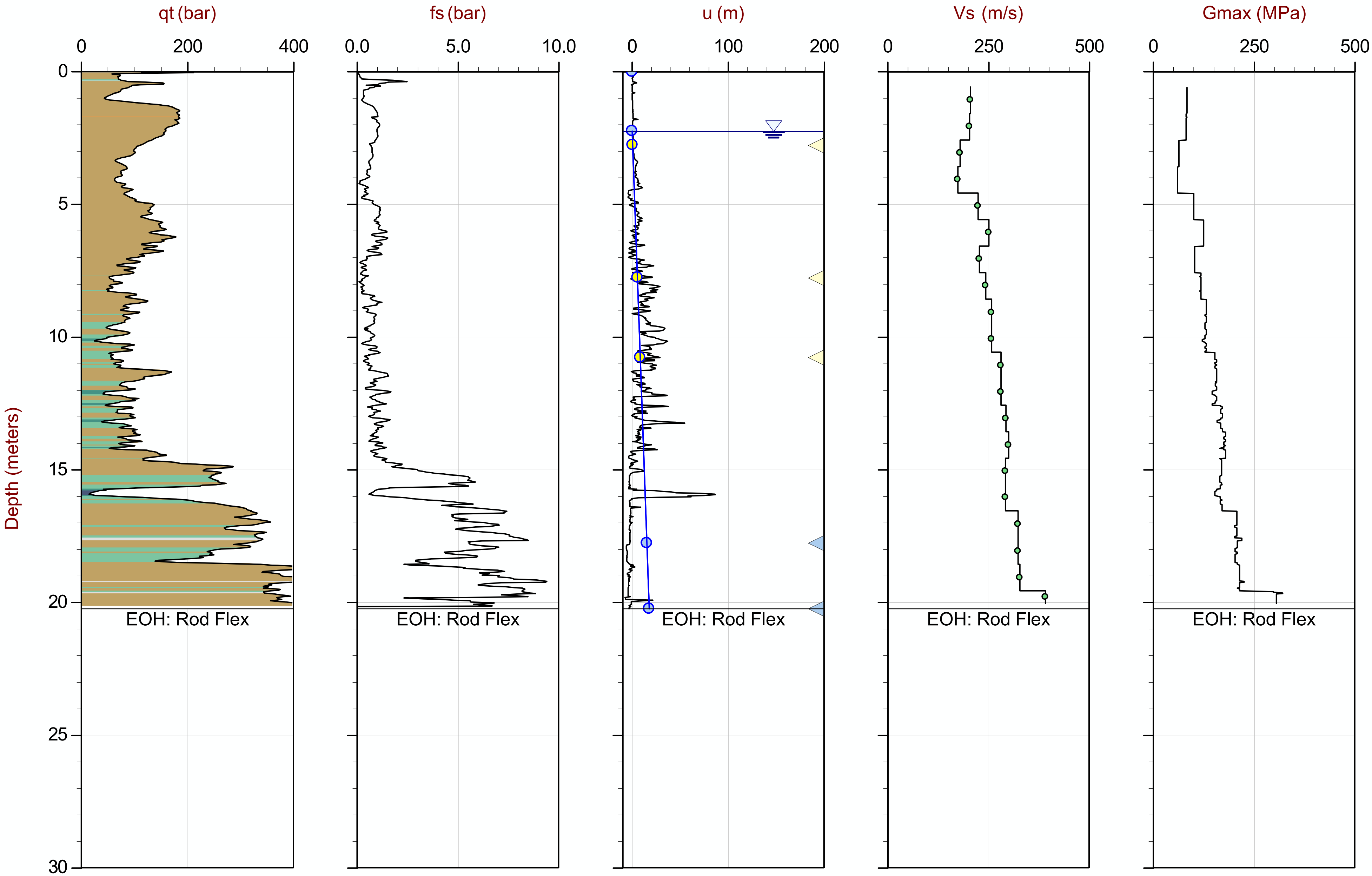
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

GHD

Job No: 23-05-25396
Date: 2023-02-27 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35



Max Depth: 20.250 m / 66.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914996m E: 604048m
Sheet No: 1 of 1

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

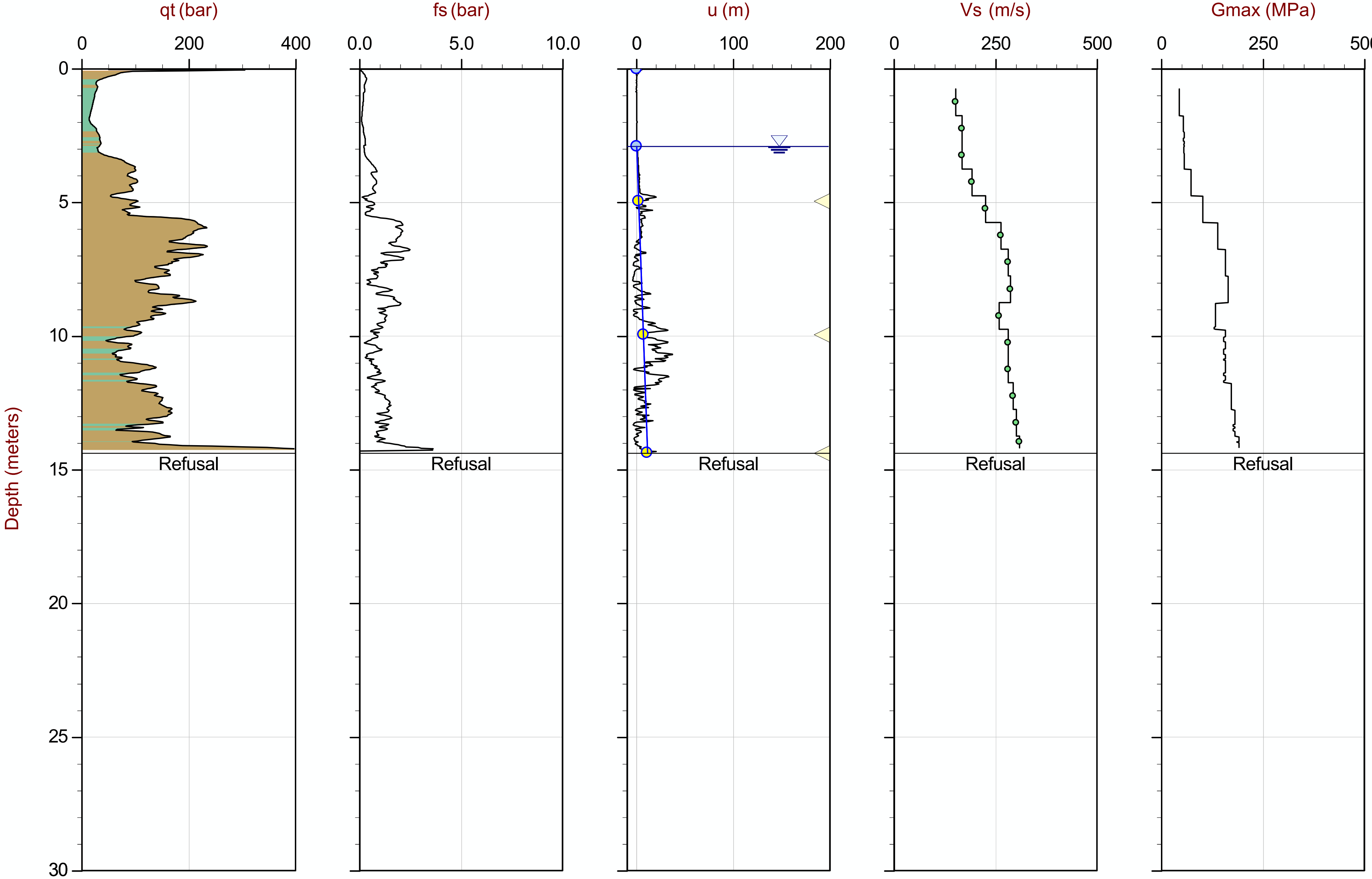
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915006m E:604004m
Sheet No: 1 of 1

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

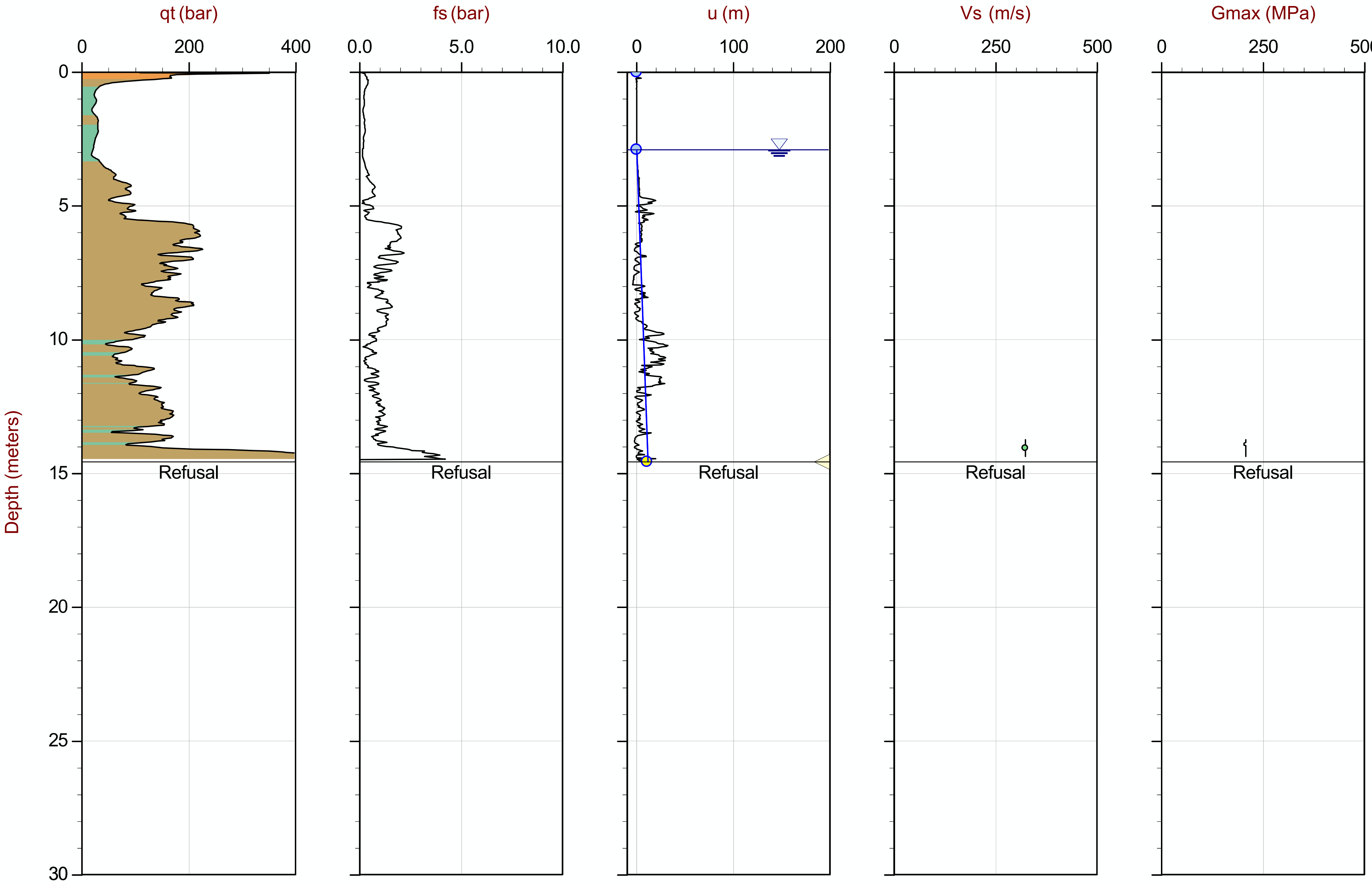
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 11:00
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B
Cone: 824:T1000F10U35



Max Depth: 14.575 m / 47.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4915007m E:604004m
Sheet No: 1 of 1

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

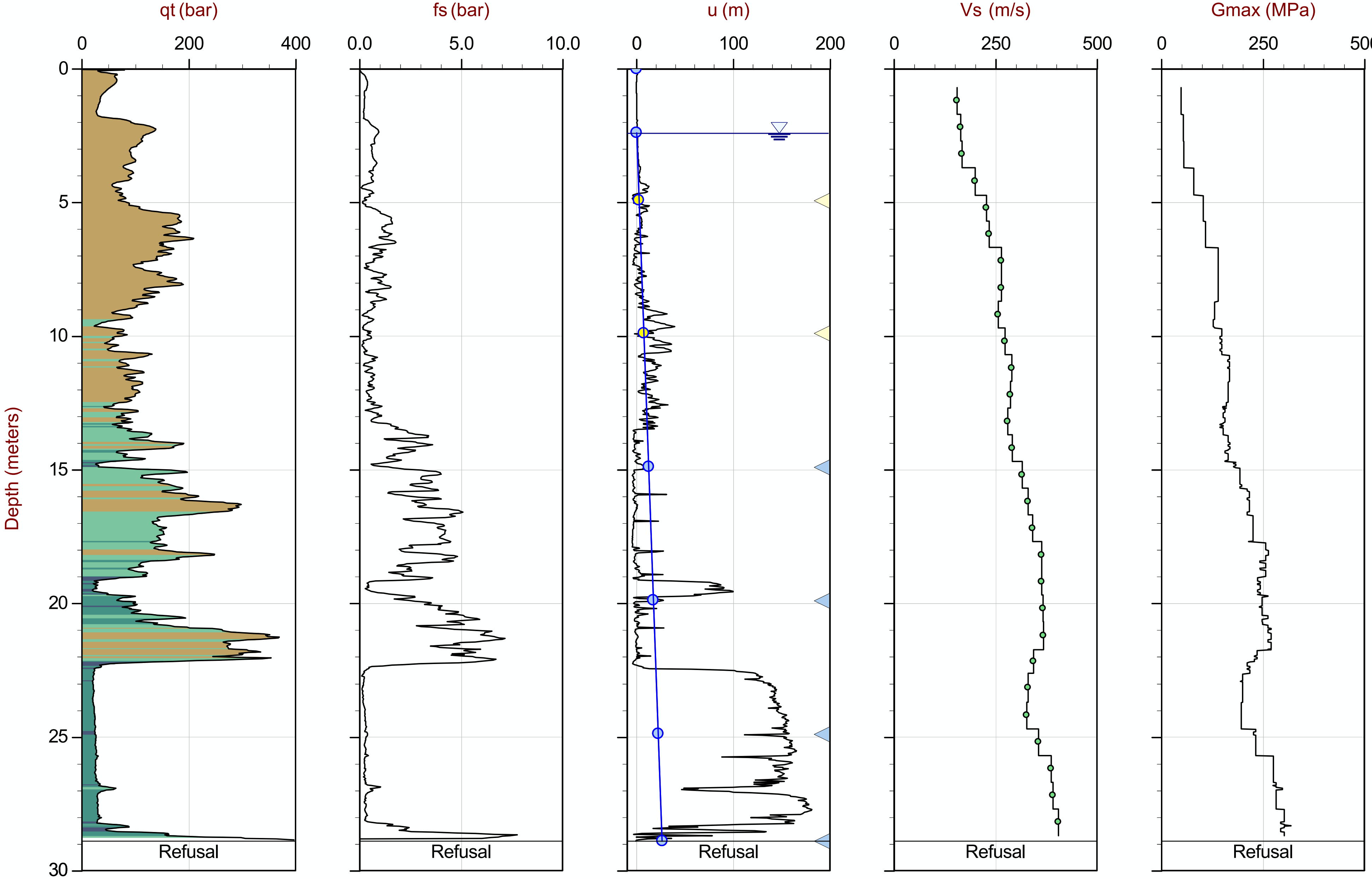
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35



Max Depth: 28.900 m / 94.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25396_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4914953m E:604010m
Sheet No: 1 of 1

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

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

Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results

Job No: 23-05-25396
Client: GHD
Project: 126-140 Bradford Street Barrie
Sounding ID: SCPT23-01
Date: 27-Feb-2023

Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.92	0.72	0.91			
1.90	1.70	1.79	0.88	5.33	165
2.90	2.70	2.76	0.97	5.28	184
3.90	3.70	3.74	0.99	6.93	142
4.90	4.70	4.73	0.99	5.52	180
5.90	5.70	5.73	0.99	4.50	221
6.88	6.68	6.70	0.98	4.31	227
7.90	7.70	7.72	1.02	4.65	219
8.90	8.70	8.72	1.00	4.02	248
9.90	9.70	9.72	1.00	3.87	258
10.90	10.70	10.71	1.00	3.63	275
11.90	11.70	11.71	1.00	3.63	275
12.90	12.70	12.71	1.00	3.54	283
13.90	13.70	13.71	1.00	3.54	283
14.90	14.70	14.71	1.00	3.49	286
15.90	15.70	15.71	1.00	3.34	300
16.90	16.70	16.71	1.00	3.00	333
17.88	17.68	17.69	0.98	2.86	343
18.92	18.72	18.73	1.04	3.05	341
19.88	19.68	19.69	0.96	2.52	381
20.23	20.03	20.04	0.35	0.73	481

Job No: 23-05-25396
Client: GHD
Project: 126-140 Bradford Street Barrie
Sounding ID: SCPT23-02
Date: 27-Feb-2023

Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.78	0.58	0.80			
1.78	1.58	1.67	0.87	4.23	206
2.78	2.58	2.64	0.97	4.74	204
3.78	3.58	3.62	0.98	5.47	180
4.78	4.58	4.61	0.99	5.66	175
5.78	5.58	5.61	0.99	4.42	225
6.78	6.58	6.60	1.00	3.97	251
7.78	7.58	7.60	1.00	4.38	228
8.78	8.58	8.60	1.00	4.09	244
9.78	9.58	9.60	1.00	3.87	258
10.78	10.58	10.59	1.00	3.86	258
11.78	11.58	11.59	1.00	3.55	282
12.78	12.58	12.59	1.00	3.54	282
13.78	13.58	13.59	1.00	3.40	294
14.78	14.58	14.59	1.00	3.32	301
15.75	15.55	15.56	0.97	3.32	293
16.75	16.55	16.56	1.00	3.40	294
17.78	17.58	17.59	1.03	3.18	324
18.78	18.58	18.59	1.00	3.08	325
19.78	19.58	19.59	1.00	3.04	329
20.25	20.05	20.06	0.47	1.20	393

Job No:	23-05-25396
Client:	GHD
Project:	126-140 Bradford Street Barrie
Sounding ID:	SCPT23-03
Date:	27-Feb-2023

Seismic Source:	Beam
Seismic Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.95	0.75	0.93			
1.95	1.75	1.83	0.90	5.96	152
2.95	2.75	2.80	0.97	5.78	168
3.95	3.75	3.79	0.99	5.86	168
4.95	4.75	4.78	0.99	5.16	192
5.95	5.75	5.78	0.99	4.39	227
6.95	6.75	6.77	1.00	3.77	264
7.95	7.75	7.77	1.00	3.54	282
8.95	8.75	8.77	1.00	3.47	288
9.95	9.75	9.77	1.00	3.85	260
10.95	10.75	10.76	1.00	3.55	282
11.95	11.75	11.76	1.00	3.54	282
12.95	12.75	12.76	1.00	3.39	295
13.95	13.75	13.76	1.00	3.31	302
14.38	14.18	14.19	0.43	1.39	310

Job No:	23-05-25396
Client:	GHD
Project:	126-140 Bradford Street Barrie
Sounding ID:	SCPT23-03B
Date:	27-Feb-2023

Seismic Source:	Beam
Seismic Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
13.95	13.75	13.76			
14.58	14.38	14.39	0.63	1.95	324

Job No: 23-05-25396
Client: GHD
Project: 126-140 Bradford Street Barrie
Sounding ID: SCPT23-04
Date: 27-Feb-2023

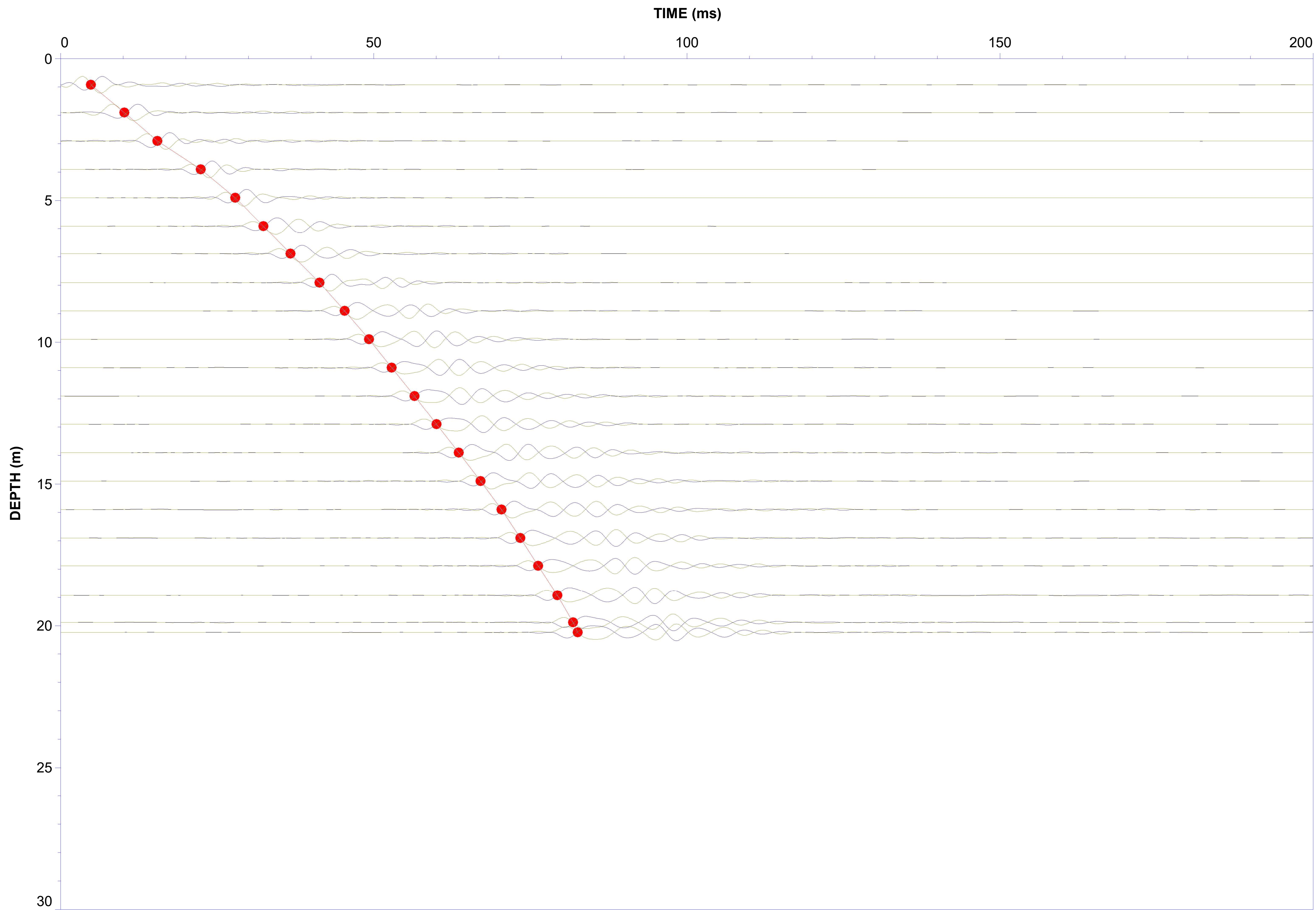
Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.90	0.70	0.89			
1.90	1.70	1.79	0.90	5.77	156
2.90	2.70	2.76	0.97	5.88	165
3.90	3.70	3.74	0.99	5.89	168
4.93	4.73	4.76	1.02	5.09	201
5.90	5.70	5.73	0.96	4.22	228
6.88	6.68	6.70	0.98	4.17	234
7.90	7.70	7.72	1.02	3.84	265
8.90	8.70	8.72	1.00	3.76	265
9.90	9.70	9.72	1.00	3.89	257
10.90	10.70	10.71	1.00	3.65	274
11.90	11.70	11.71	1.00	3.44	291
12.90	12.70	12.71	1.00	3.48	287
13.90	13.70	13.71	1.00	3.56	281
14.90	14.70	14.71	1.00	3.42	292
15.90	15.70	15.71	1.00	3.17	316
16.90	16.70	16.71	1.00	3.02	331
17.90	17.70	17.71	1.00	2.93	342
18.90	18.70	18.71	1.00	2.74	364
19.90	19.70	19.71	1.00	2.74	365
20.90	20.70	20.71	1.00	2.72	368
21.95	21.75	21.76	1.05	2.85	369
22.83	22.63	22.64	0.88	2.55	345
23.90	23.70	23.71	1.07	3.23	331
24.90	24.70	24.71	1.00	3.06	327
25.90	25.70	25.71	1.00	2.81	357
26.90	26.70	26.71	1.00	2.58	388
27.90	27.70	27.71	1.00	2.55	392
28.90	28.70	28.71	1.00	2.46	406

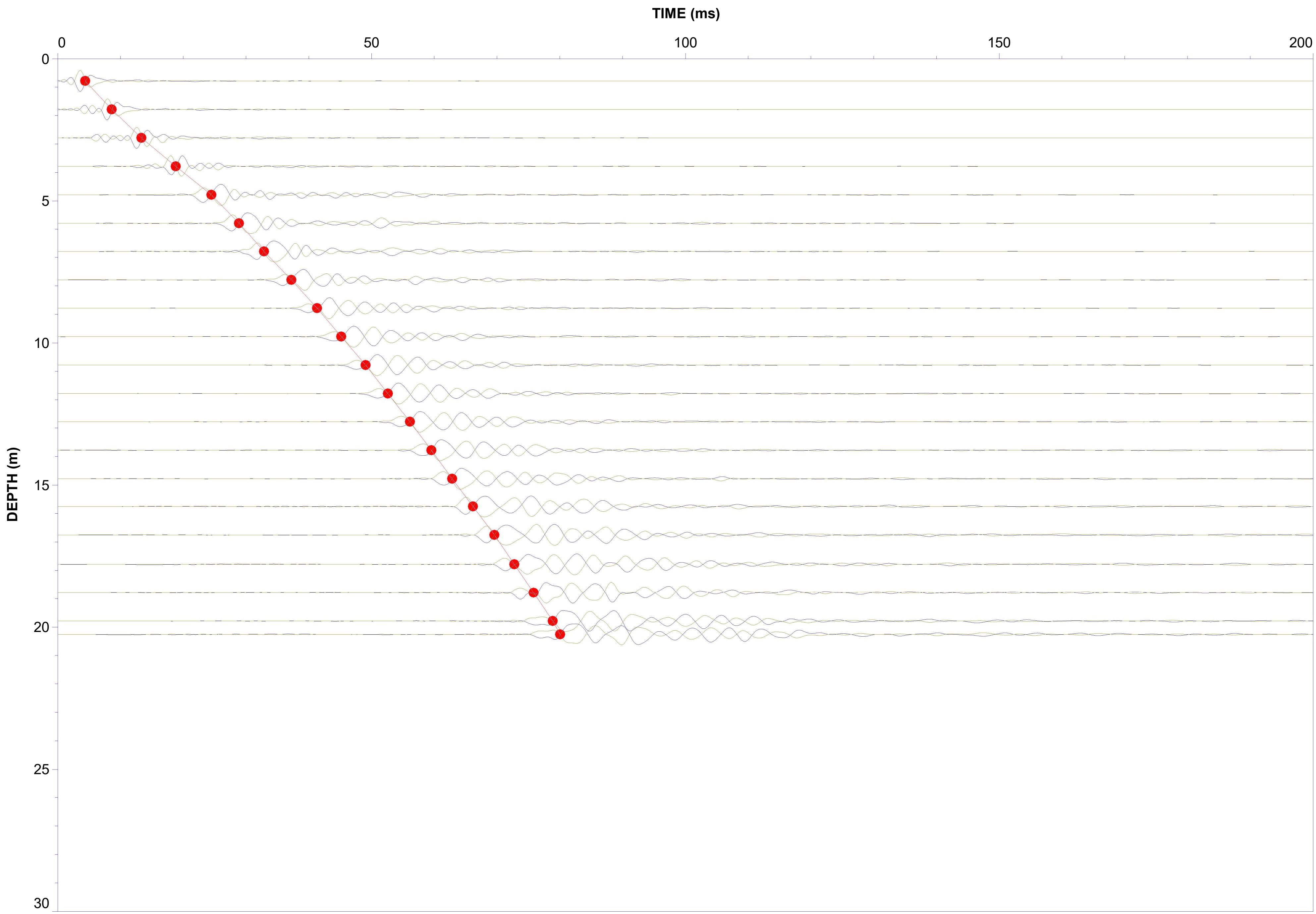
Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 23-05-25396 Client: GHD Project: 126-140 Bradford Street Barrie Filter: 0-250 Hz Sounding: SCPT23-01 Analysis: S Wave - Geo X
Date: 27-Feb-2023

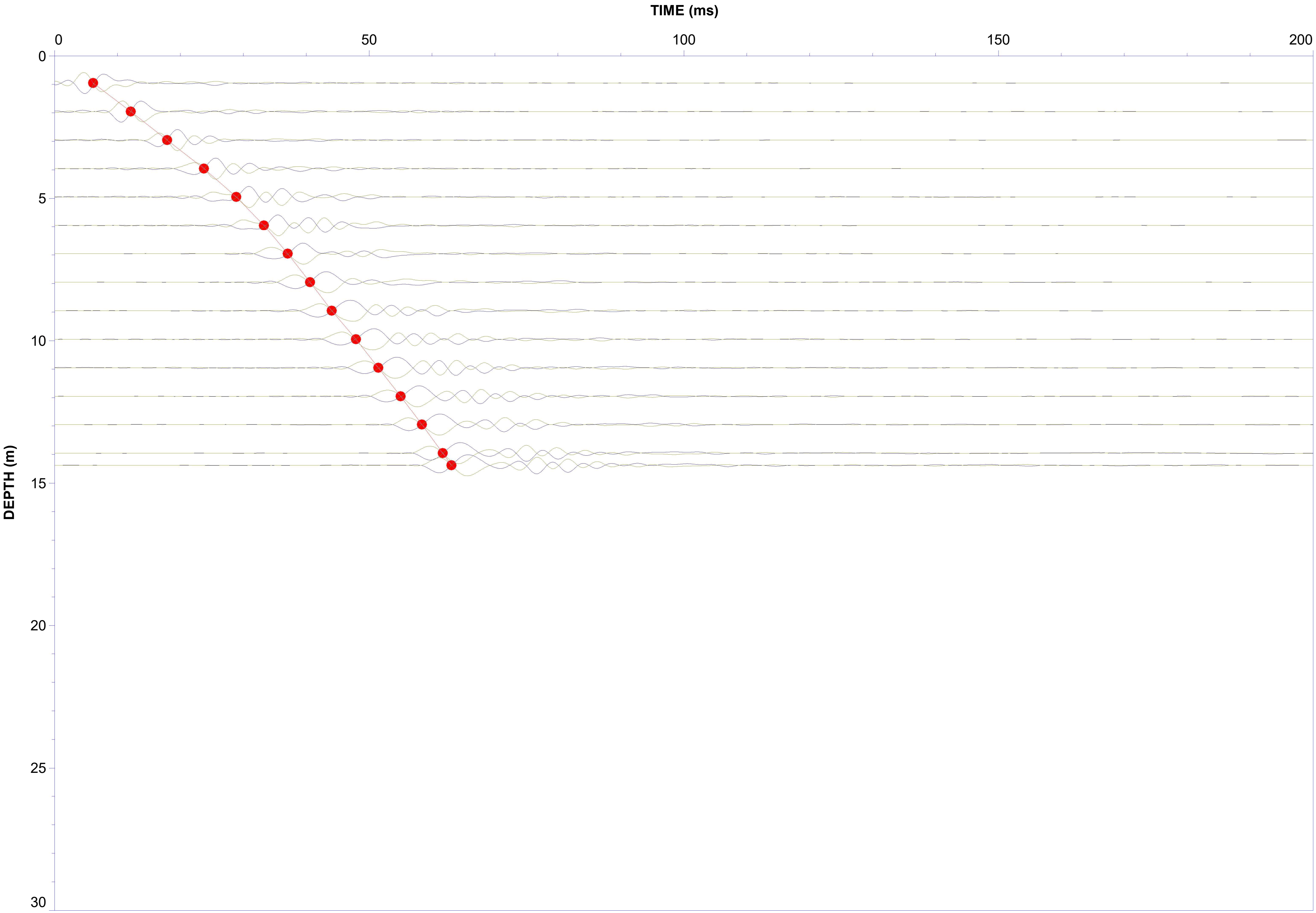


CONETEC

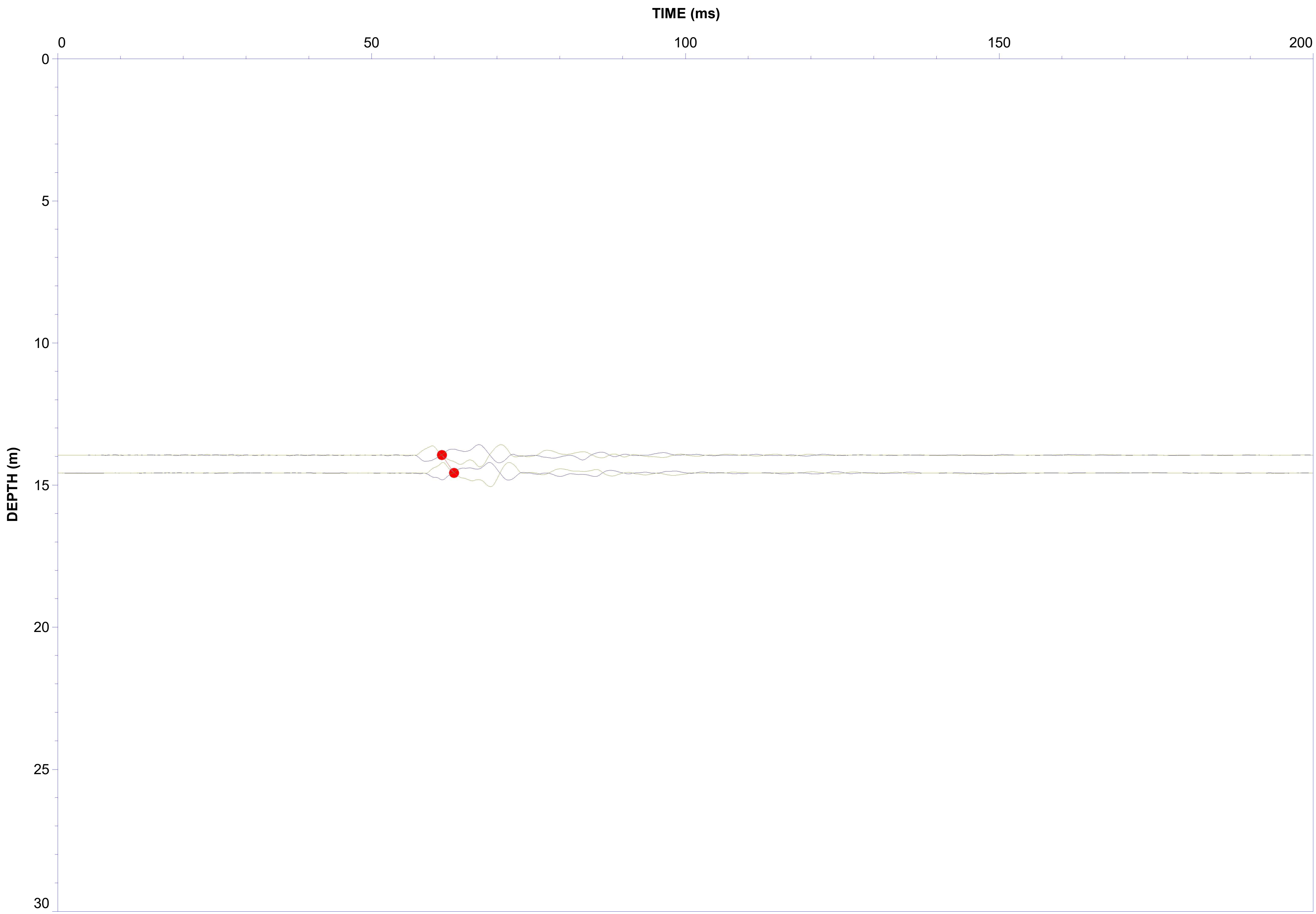
Job No: 23-05-25396
Date: 27-Feb-2023
Client: GHD
Project: 126-140 Bradford Street Barrie
Filter: 0-500 Hz
Sounding: SCPT23-02
Analysis: S Wave - Geo X



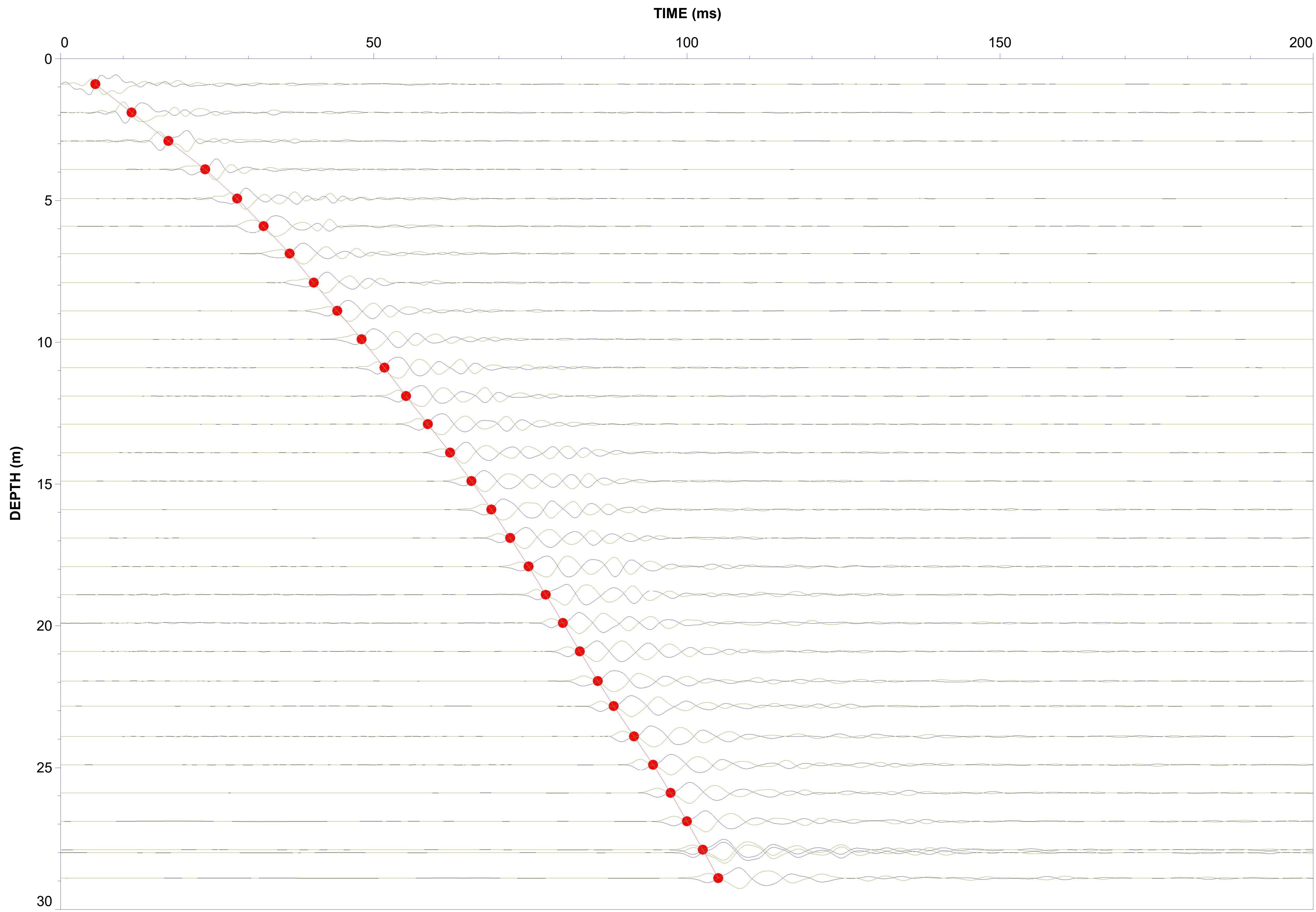
Job No: 23-05-25396
Date: 27-Feb-2023
Client: GHD
Project: 126-140 Bradford Street Barrie
Filter: 0-250 Hz
Sounding: SCPT23-03
Analysis: S Wave - Geo X



Job No: 23-05-25396 Client: GHD Project: 126-140 Bradford Street Barrie Filter: None Sounding: SCPT23-03B Analysis: S Wave - Geo X
Date: 27-Feb-2023



Job No: 23-05-25396 Date: 27-Feb-2023	Client: GHD	Project: 126-140 Bradford Street Barrie	Filter: 0-400 Hz	Sounding: SCPT23-04	Analysis: S Wave - Geo X



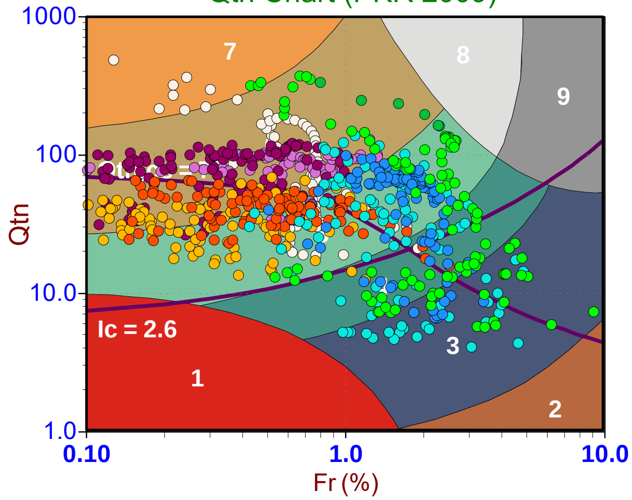
Soil Behaviour Type (SBT) Scatter Plots

GHD

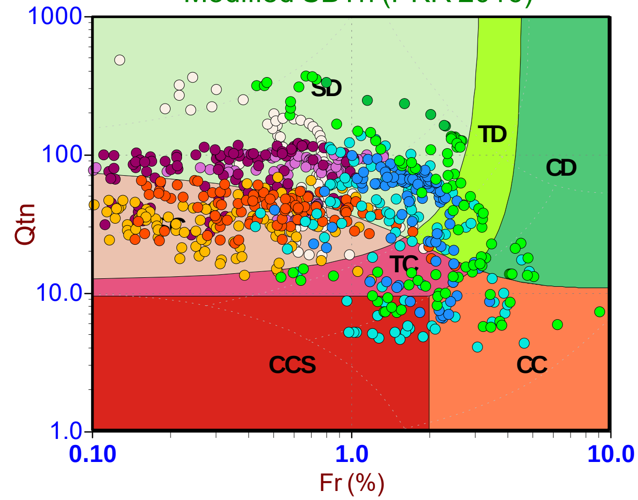
Job No: 23-05-25396
Date: 2023-02-27 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35

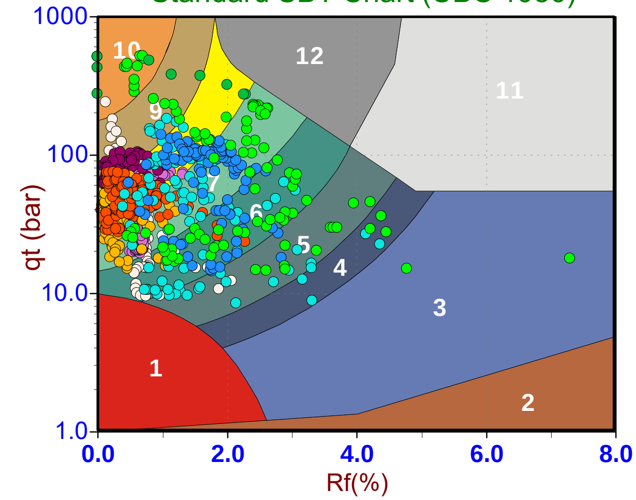
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend

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

Legend

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

Legend

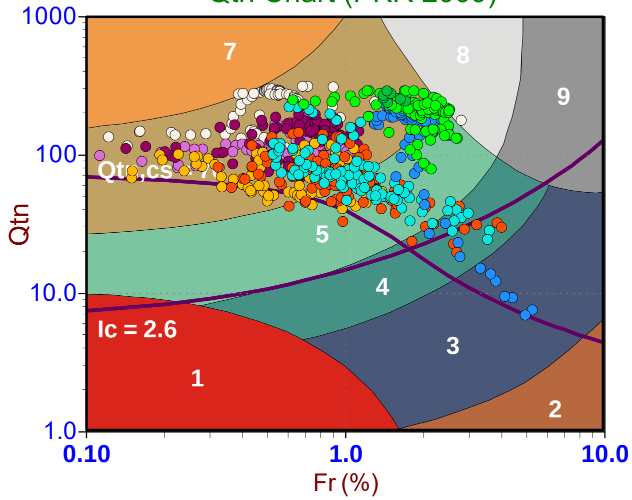
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

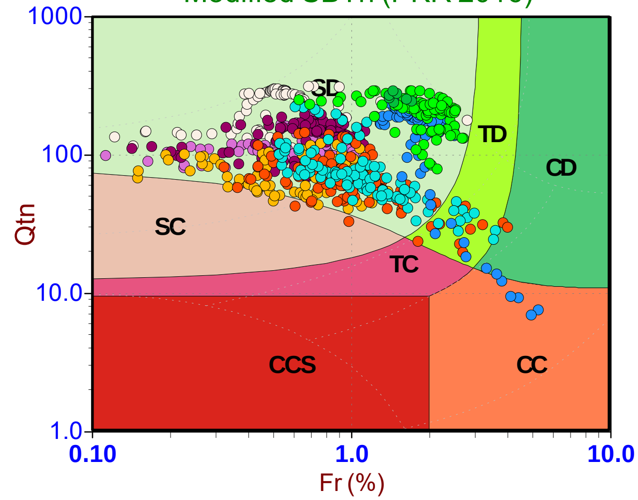
GHD

Job No: 23-05-25396
Date: 2023-02-27 07:41
Site: 126-140 Bradford St, Barrie
Sounding: SCPT23-02
Cone: 824:T1000F10U35

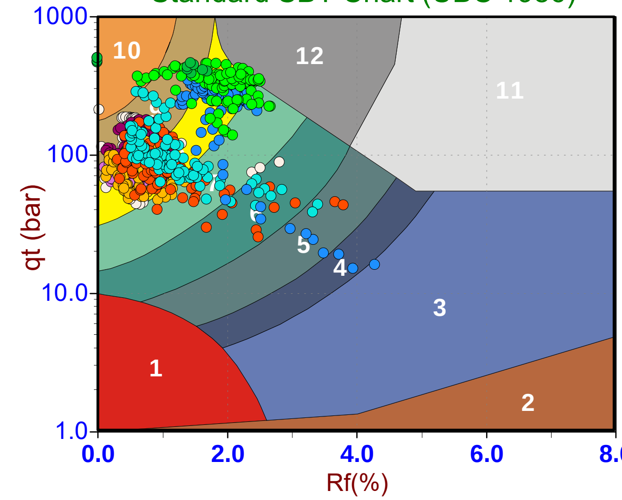
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend

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

Legend

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

Legend

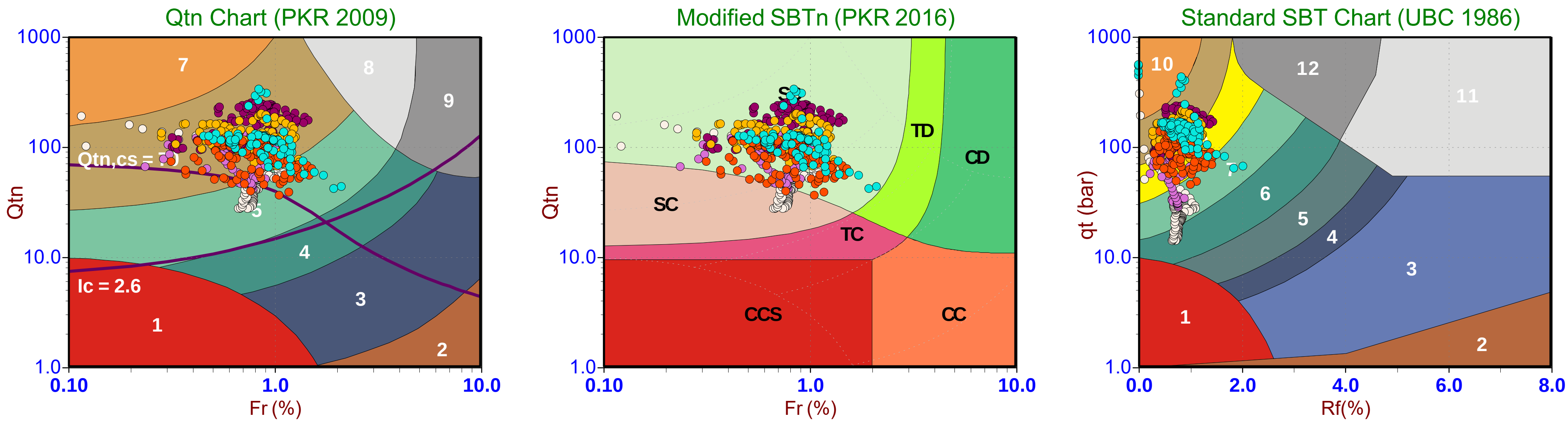
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35



Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

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

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

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

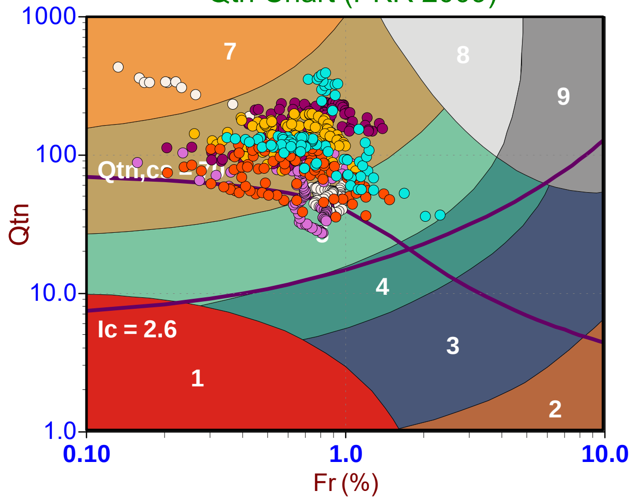
CONETEC

GHD

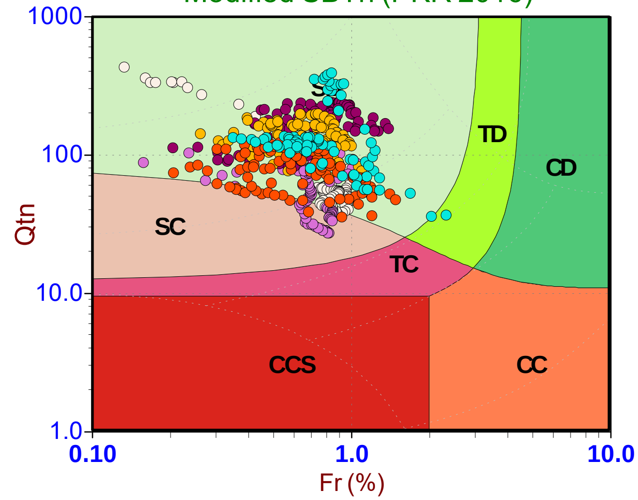
Job No: 23-05-25396
Date: 2023-02-27 11:00
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B
Cone: 824:T1000F10U35

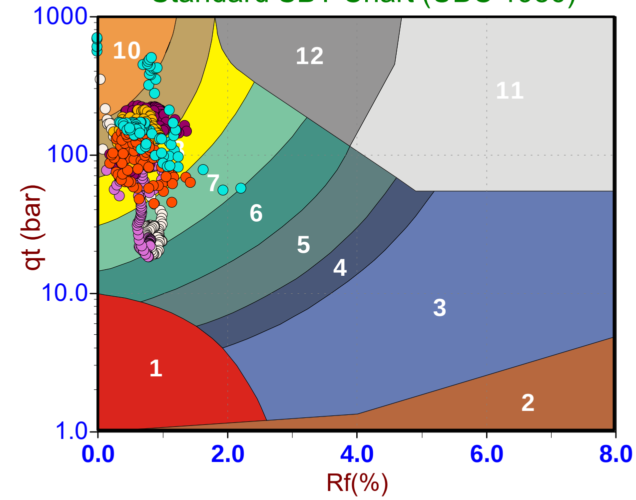
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend

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

Legend

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

Legend

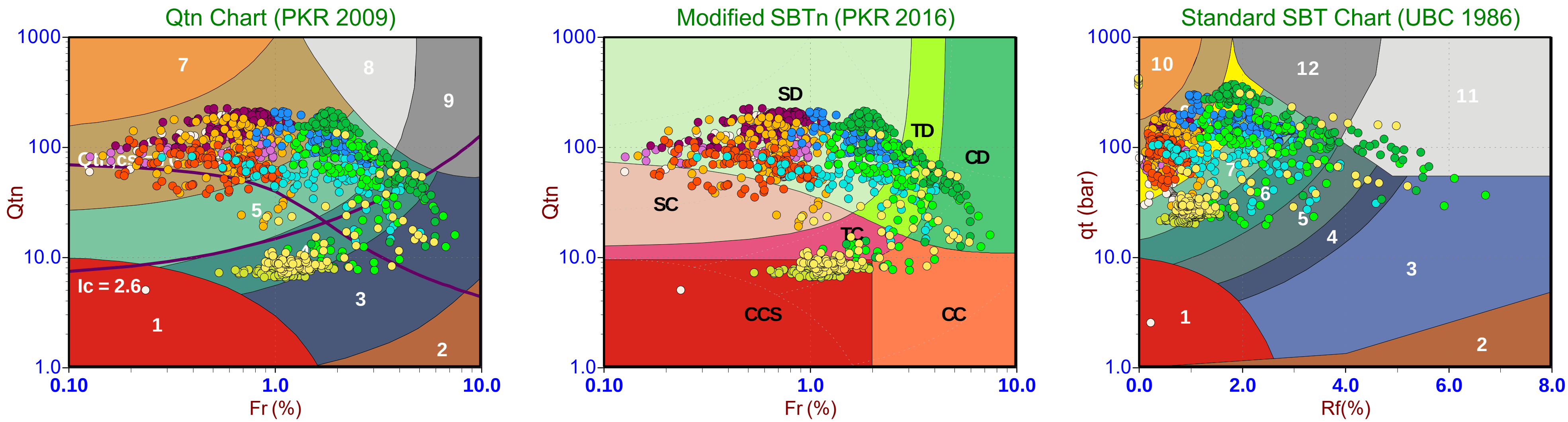
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

GHD

Job No: 23-05-25396
Date: 2023-02-27 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35



Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

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

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

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

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 23-05-25396
Client: GHD Group
Project: 126-140 Bradford Street Barrie
Start Date: 27-Feb-2023
End Date: 27-Feb-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	$U_{Initial}$ (m)	U_{max} (m)	U_{min} (m)	U_{final} (m)	Equilibrium Pore Pressure U_{eq} (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Percent Dissipation (%)	t_{50} $(s)_1$	Assumed Rigidity Index (I_r)	c_h $(cm^2/min)_2$	Refer to Notation Number
SCPT23-01	23-05-25396_SP01	15	500	3.900	3.8	4.3	2.0	2.0	1.9		2.0	99	46			
SCPT23-01	23-05-25396_SP01	15	900	9.900	31.7	33.6	7.9	7.9	7.9		2.0	100	29	100	24.2	
SCPT23-01	23-05-25396_SP01	15	300	16.900	37.3	55.2	23.3	23.3		14.9	2.0	79	101	100	6.9	3
SCPT23-01	23-05-25396_SP01	15	670	20.225	41.1	80.3	2.2	25.7		18.2	2.0	88	75	100	9.4	3
SCPT23-02	23-05-25396_SP02	15	305	2.775	0.8	1.2	0.1	0.5	0.5		2.2	100				
SCPT23-02	23-05-25396_SP02	15	600	7.775	14.6	30.7	5.7	5.7	5.7		2.1	100	31	100	22.4	
SCPT23-02	23-05-25396_SP02	15	300	10.775	32.4	34.2	8.6	8.6	8.5		2.2	100	7			
SCPT23-02	23-05-25396_SP02	15	425	17.775	-3.0	45.7	-6.9	23.6		15.5	2.2	73	160	100	4.4	3
SCPT23-02	23-05-25396_SP02	15	620	20.250	-3.4	55.1	-6.1	31.2		18.0	2.2	64	303	100	2.3	3
SCPT23-03	23-05-25396_SP03	15	300	4.950	3.2	7.6	2.0	2.1	2.1		2.9	100	4			
SCPT23-03	23-05-25396_SP03	15	325	9.950	10.2	25.5	7.0	7.0	6.9		3.0	99	2			
SCPT23-03	23-05-25396_SP03	15	310	14.375	11.4	49.2	9.1	11.2	11.1		3.3	100	4			
SCPT23-03B	23-05-25396_SP03B	15	320	14.575	5.3	12.9	-0.7	11.1	11.1		3.5	99				
SCPT23-04	23-05-25396_SP04	15	400	4.925	1.4	18.1	1.4	2.6	2.5		2.4	99	18			
SCPT23-04	23-05-25396_SP04	15	400	9.900	19.2	24.6	7.7	7.7	7.6		2.3	99	2			
SCPT23-04	23-05-25396_SP04	15	600	14.900	15.2	48.2	14.2	24.9		12.6	2.3	65	348	100	2.0	3
SCPT23-04	23-05-25396_SP04	15	600	19.900	34.1	70.9	23.7	42.2		17.6	2.3	54	498	100	1.4	3
SCPT23-04	23-05-25396_SP04	15	2190	24.900	156.6	156.6	90.9	90.9		22.6	2.3	49				3
SCPT23-04	23-05-25396_SP04	15	300	28.900	-1.2	62.9	-3.2	35.3		26.6	2.3	76	75	100	9.3	3

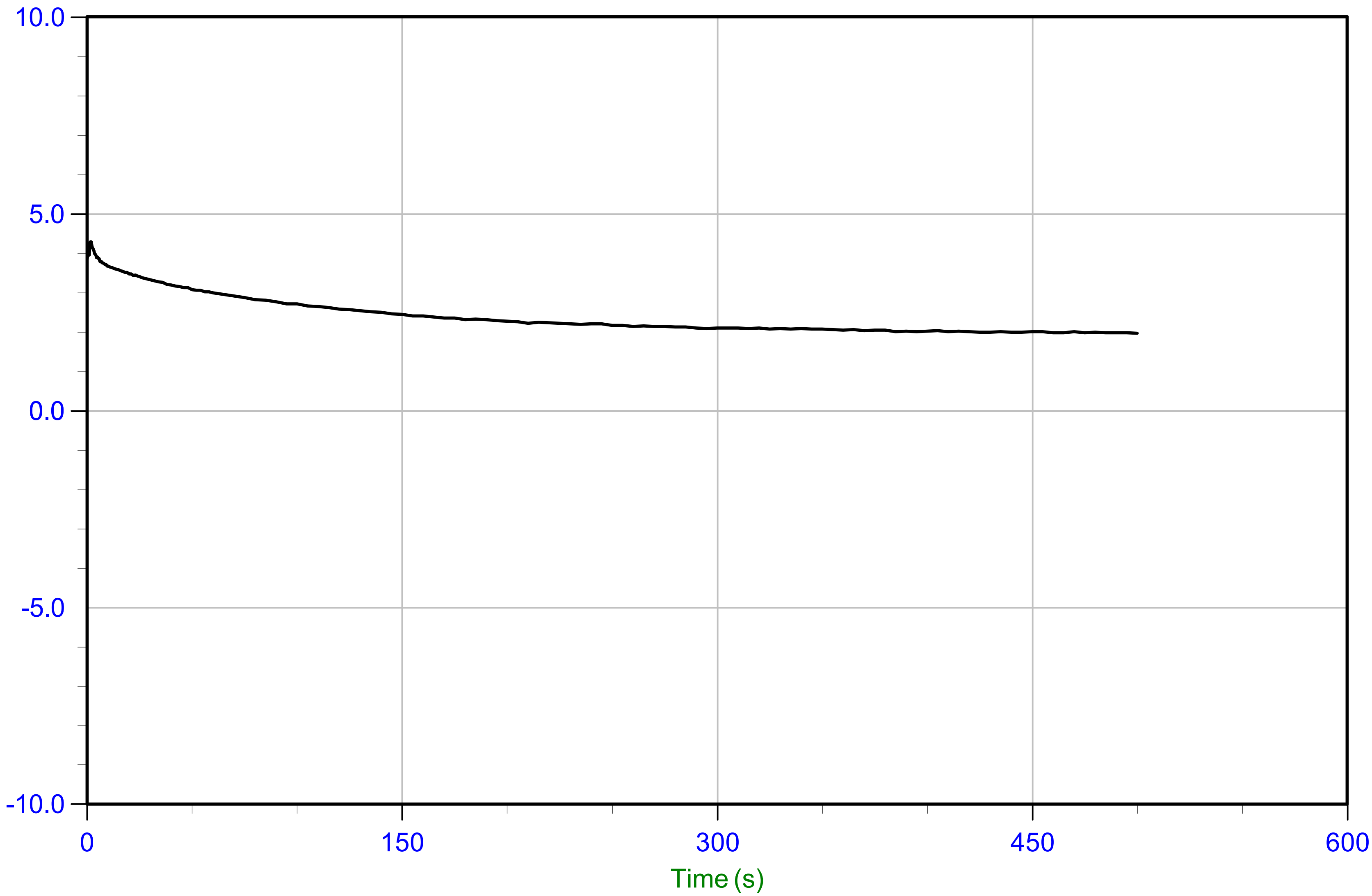
1. Time for 50 percent dissipation based on U_{max}, U_{min}, and the applied U_{eq}. Note the time is relative to where U_{max} occurred.
2. Houlsby and Teh, 1991.
3. Equilibrium pore pressure estimated based on a hydrostatic assumption from the assumed phreatic surface.

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP01.PPF2
Depth: 3.900 m / 12.795 ft
Duration: 500.0 s

u Min: 2.0 m
u Max: 4.3 m
u Final: 2.0 m

WT: 1.953 m / 6.407 ft
Ueq: 1.9 m
U(50): 3.12 m

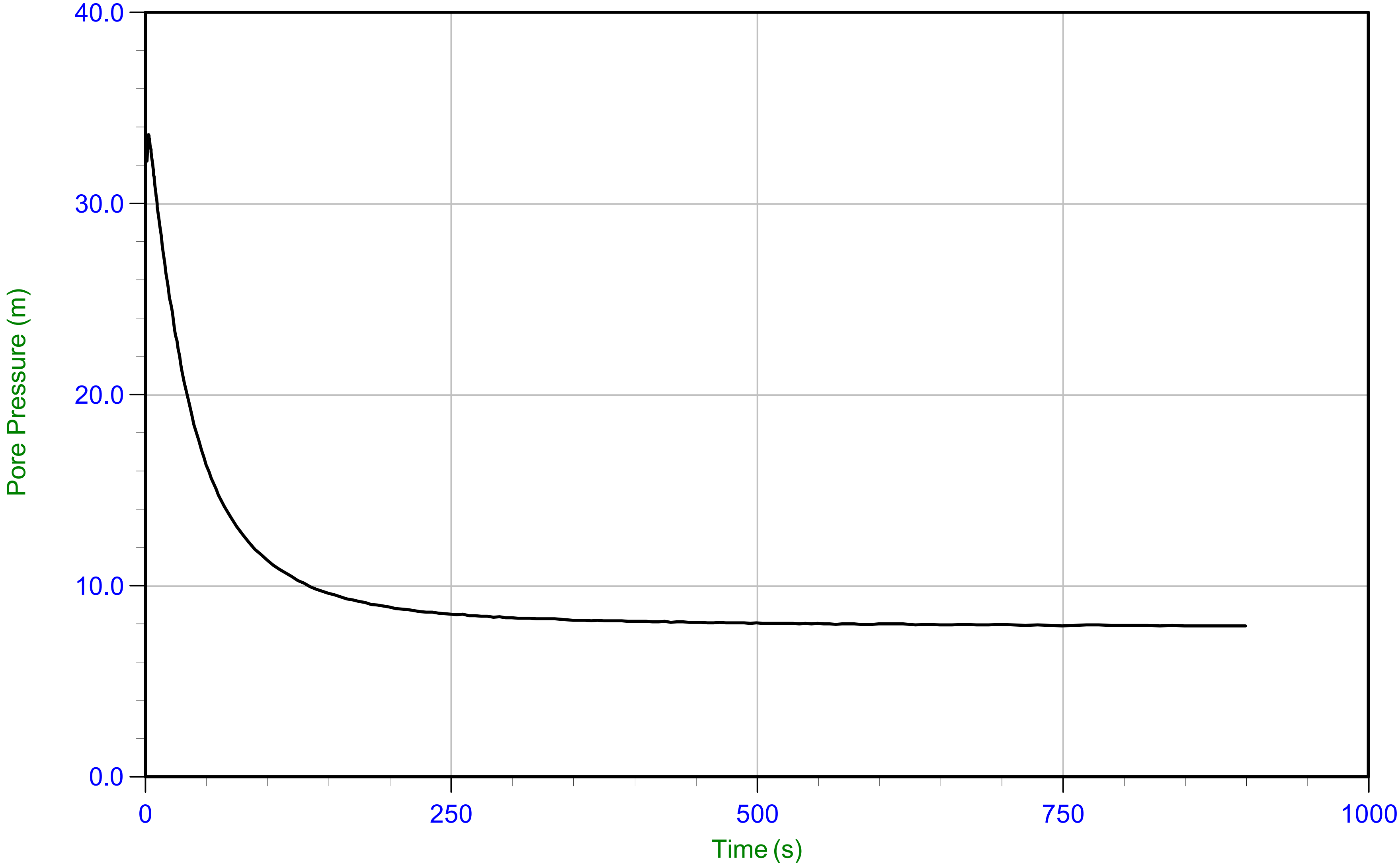
T(50): 46.2 s

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP01.PPF2
Depth: 9.900 m / 32.480 ft
Duration: 900.0 s

u Min: 7.9 m
u Max: 33.6 m
u Final: 7.9 m

WT: 2.005 m / 6.578 ft
Ueq: 7.9 m
U(50): 20.75 m

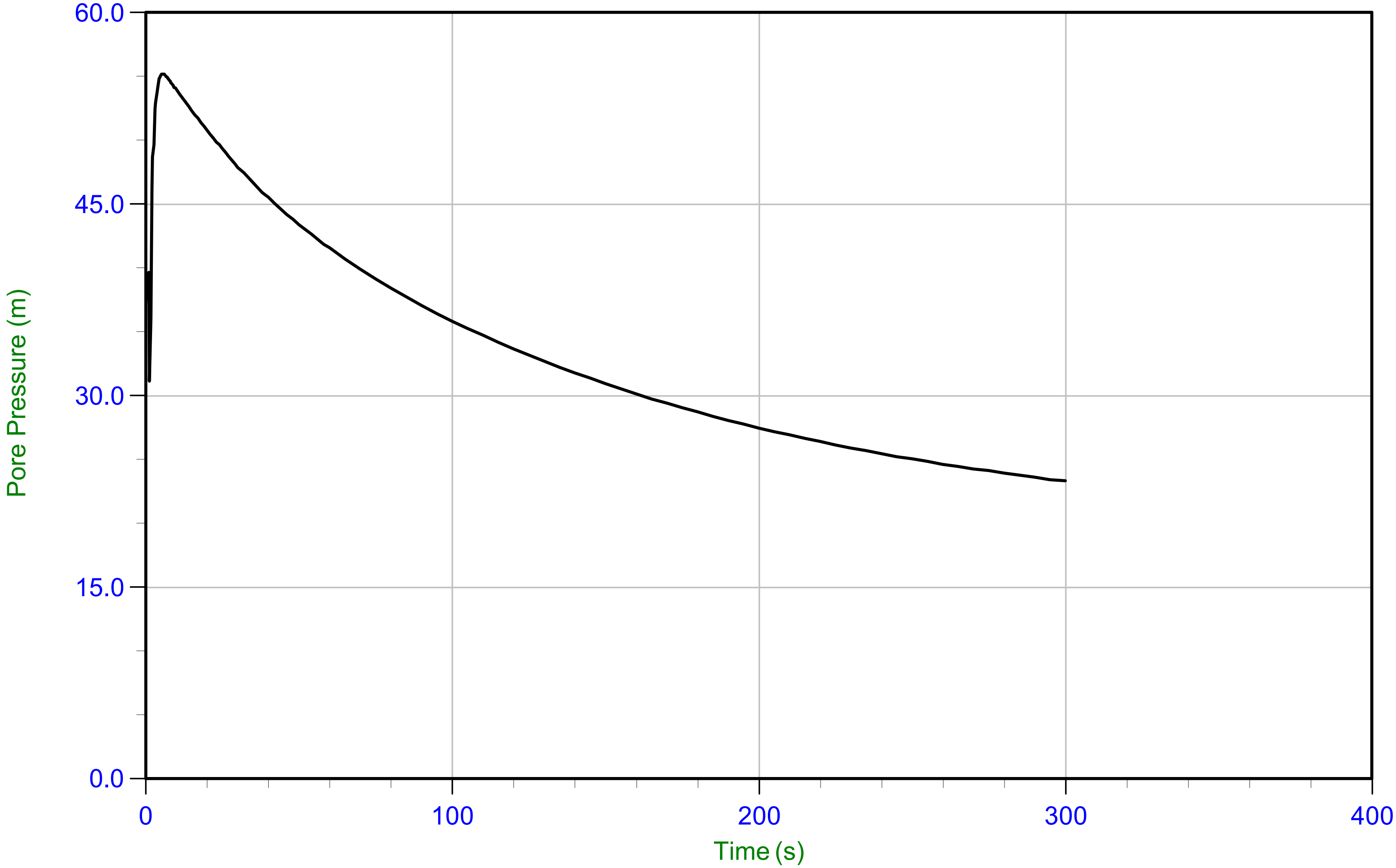
T(50): 29.0 s
Ir: 100
Ch: 24.2 cm²/min

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP01.PPF2
Depth: 16.900 m / 55.446 ft
Duration: 300.0 s

u Min: 23.3 m
u Max: 55.2 m
u Final: 23.3 m

WT: 2.005 m / 6.578 ft
Ueq: 14.9 m
U(50): 35.05 m

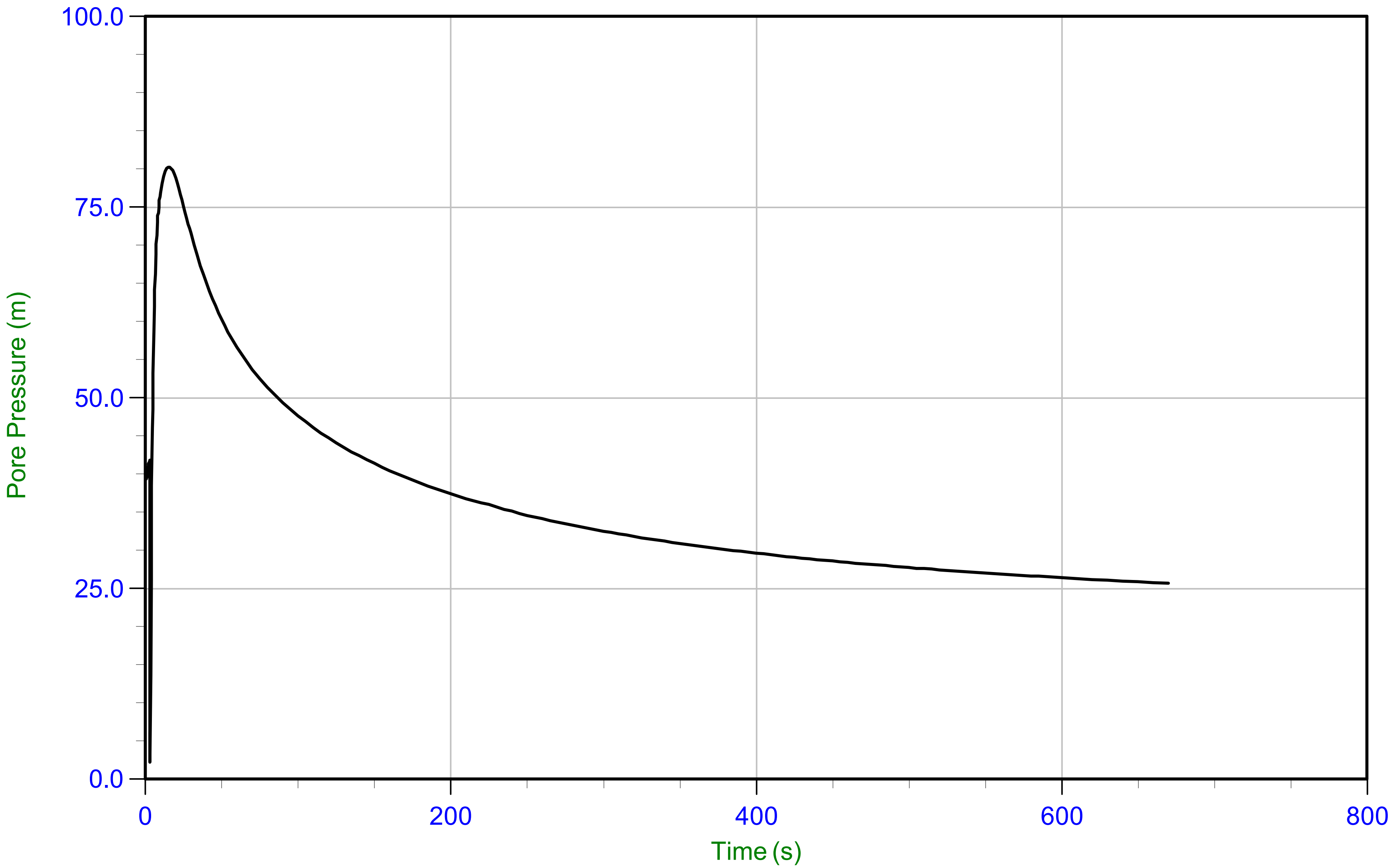
T(50): 101.3 s
Ir: 100
Ch: 6.9 cm²/min

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 05:56
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-01
Cone: 824:T1000F10U35 Area=15 cm²



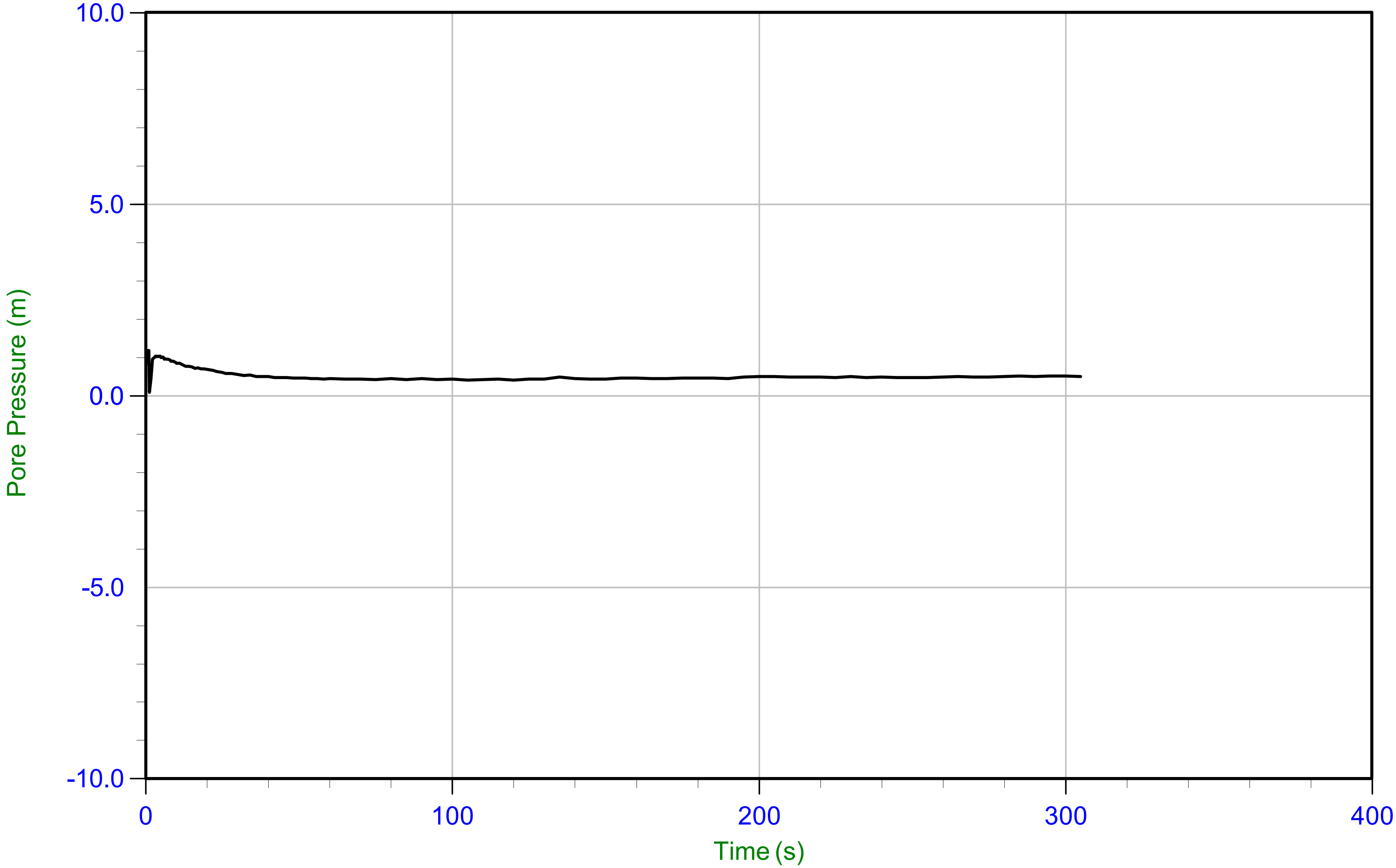
Trace Summary:	Filename: 23-05-25396_SP01.PPF2	u Min: 2.2 m	WT: 2.005 m / 6.578 ft	T(50): 74.9 s
	Depth: 20.225 m / 66.354 ft	u Max: 80.3 m	Ueq: 18.2 m	Ir: 100
	Duration: 670.0 s	u Final: 25.7 m	U(50): 49.24 m	Ch: 9.4 cm²/min

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP02.PPF2
Depth: 2.775 m / 9.104 ft
Duration: 305.0 s

u Min: 0.1 m
u Max: 1.2 m
u Final: 0.5 m

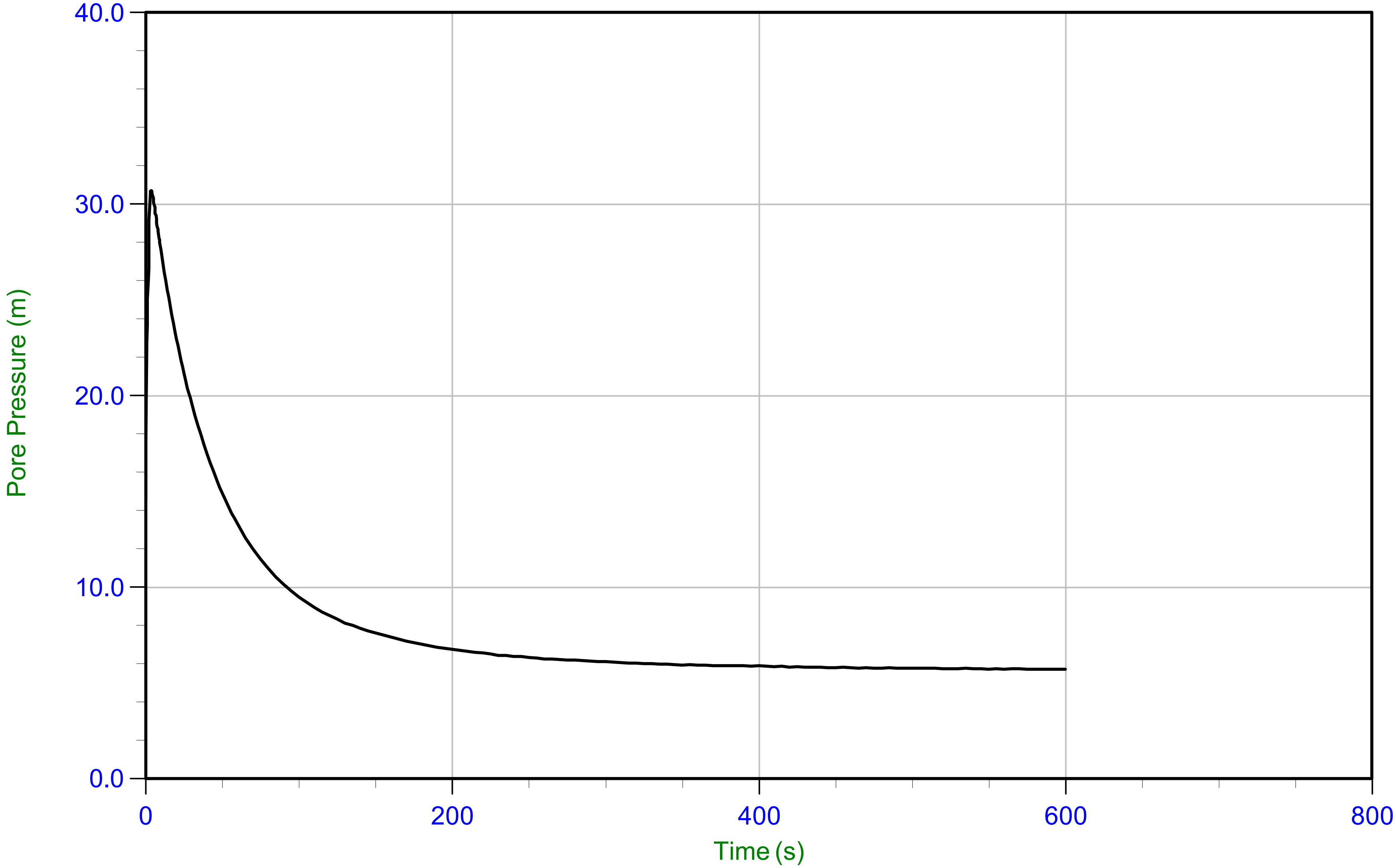
WT: 2.249 m / 7.379 ft
Ueq: 0.5 m

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP02.PPF2
Depth: 7.775 m / 25.508 ft
Duration: 600.0 s

u Min: 5.7 m
u Max: 30.7 m
u Final: 5.7 m

WT: 2.091 m / 6.860 ft
Ueq: 5.7 m
U(50): 18.19 m

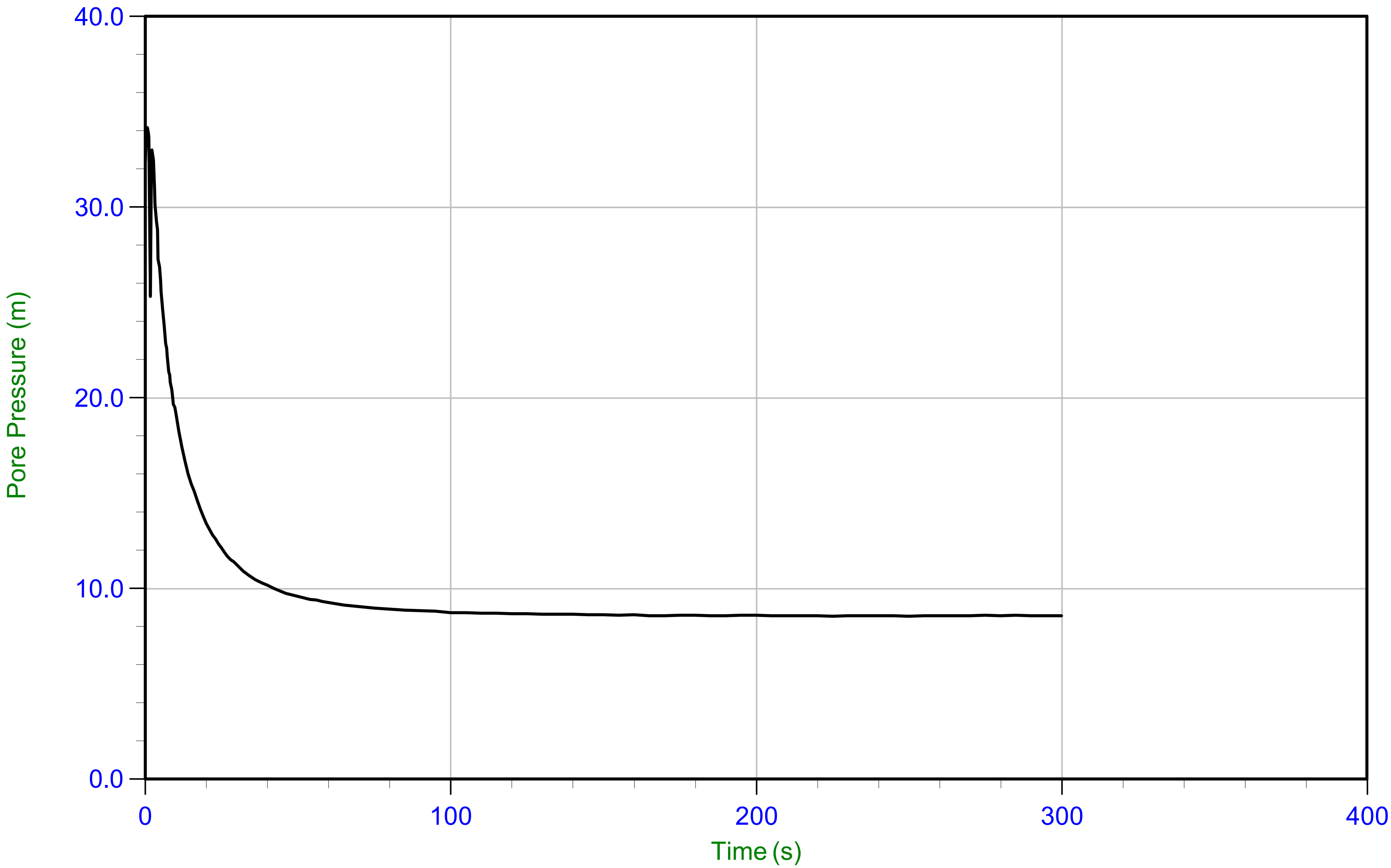
T(50): 31.3 s
Ir: 100
Ch: 22.4 cm²/min

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

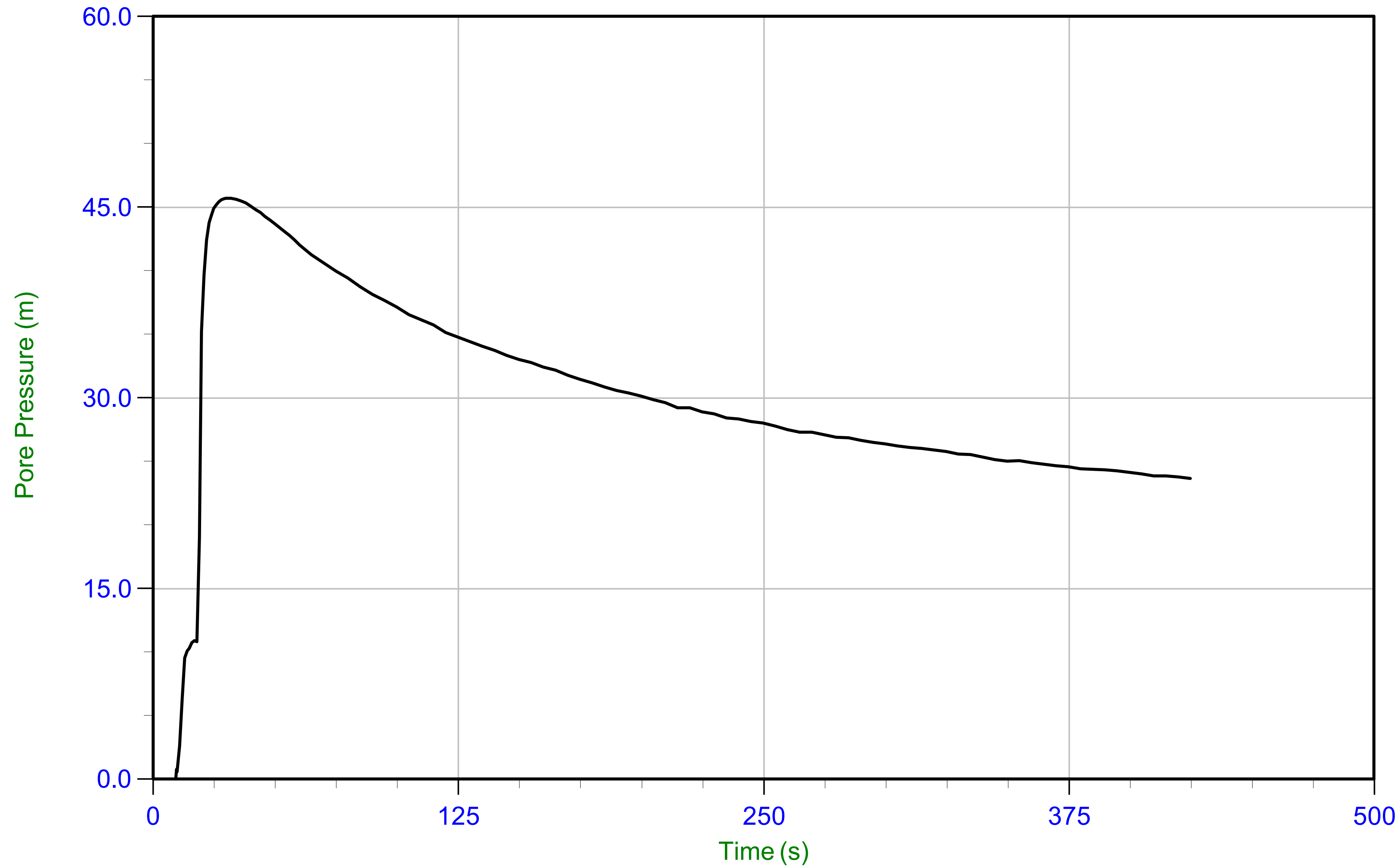
Filename: 23-05-25396_SP02.PPF2
Depth: 10.775 m / 35.351 ft
Duration: 300.0 s

u Min: 8.6 m
u Max: 34.2 m
u Final: 8.6 m

WT: 2.249 m / 7.379 ft
Ueq: 8.5 m
U(50): 21.36 m

T(50): 7.0 s

GHD	Job No: 23-05-25396 Date: 02/27/2023 07:41 Site: 126-140 Bradford St, Barrie	Sounding: SCPT23-02 Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

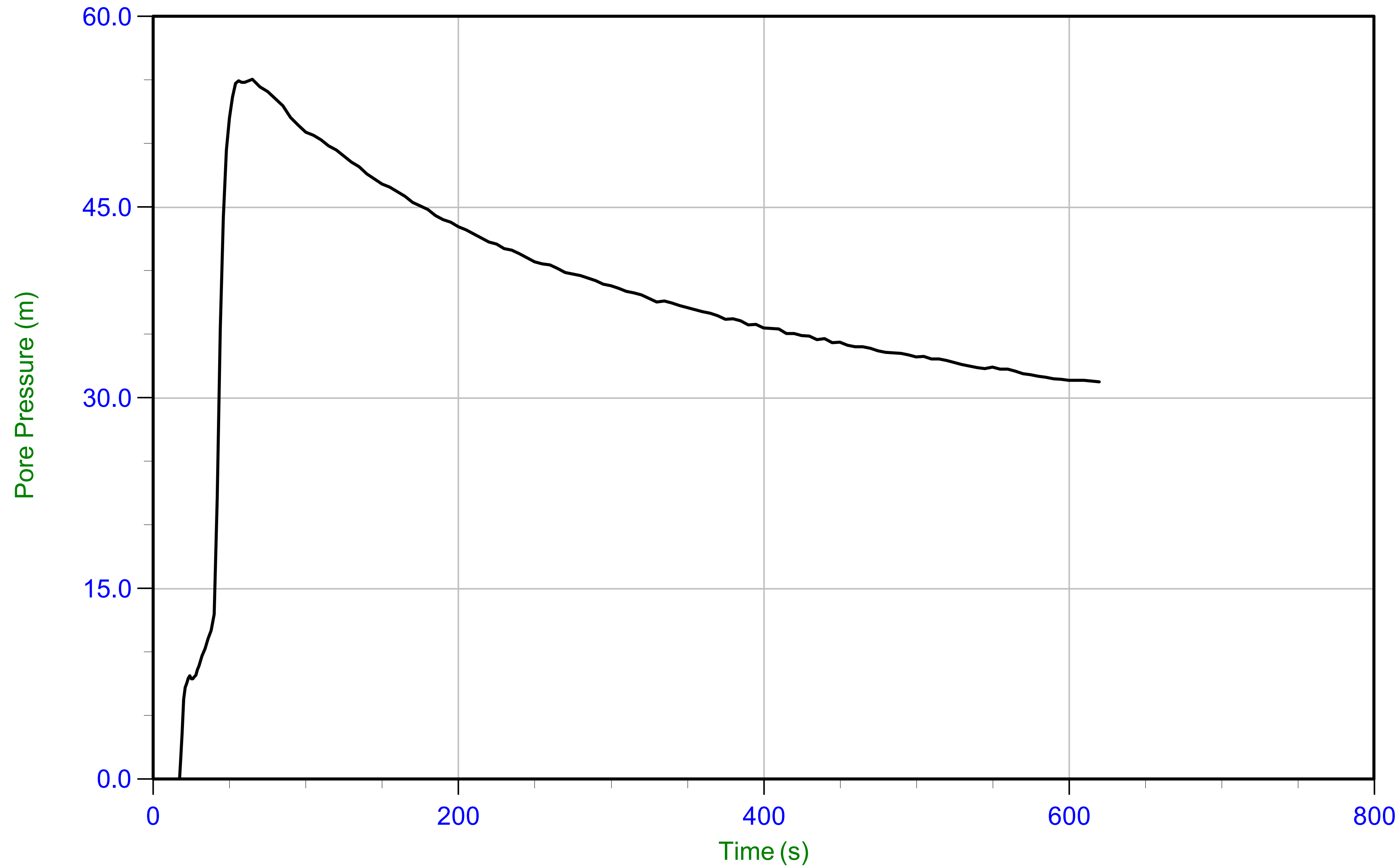
Filename: 23-05-25396_SP02.PPF2	u Min: -6.9 m	WT: 2.249 m / 7.379 ft	T(50): 159.6 s
Depth: 17.775 m / 58.316 ft	u Max: 45.7 m	Ueq: 15.5 m	Ir: 100
Duration: 425.0 s	u Final: 23.6 m	U(50): 30.62 m	Ch: 4.4 cm²/min

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 07:41
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-02
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP02.PPF2
Depth: 20.250 m / 66.436 ft
Duration: 620.0 s

u Min: -6.1 m
u Max: 55.1 m
u Final: 31.2 m

WT: 2.249 m / 7.379 ft
Ueq: 18.0 m
U(50): 36.53 m

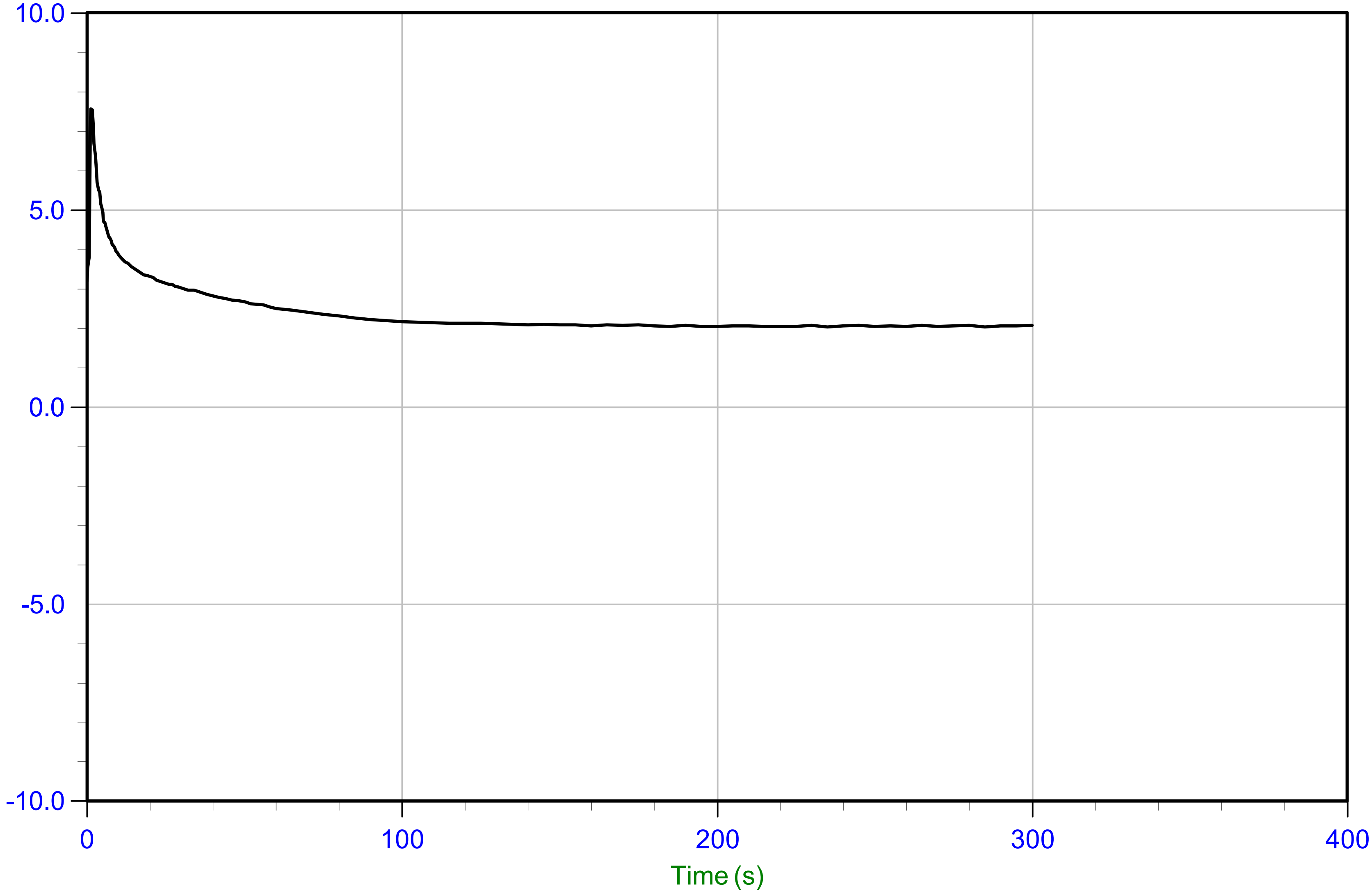
T(50): 303.4 s
Ir: 100
Ch: 2.3 cm²/min

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP03.PPF2
Depth: 4.950 m / 16.240 ft
Duration: 300.0 s

u Min: 2.0 m
u Max: 7.6 m
u Final: 2.1 m

WT: 2.897 m / 9.504 ft
Ueq: 2.1 m
U(50): 4.81 m

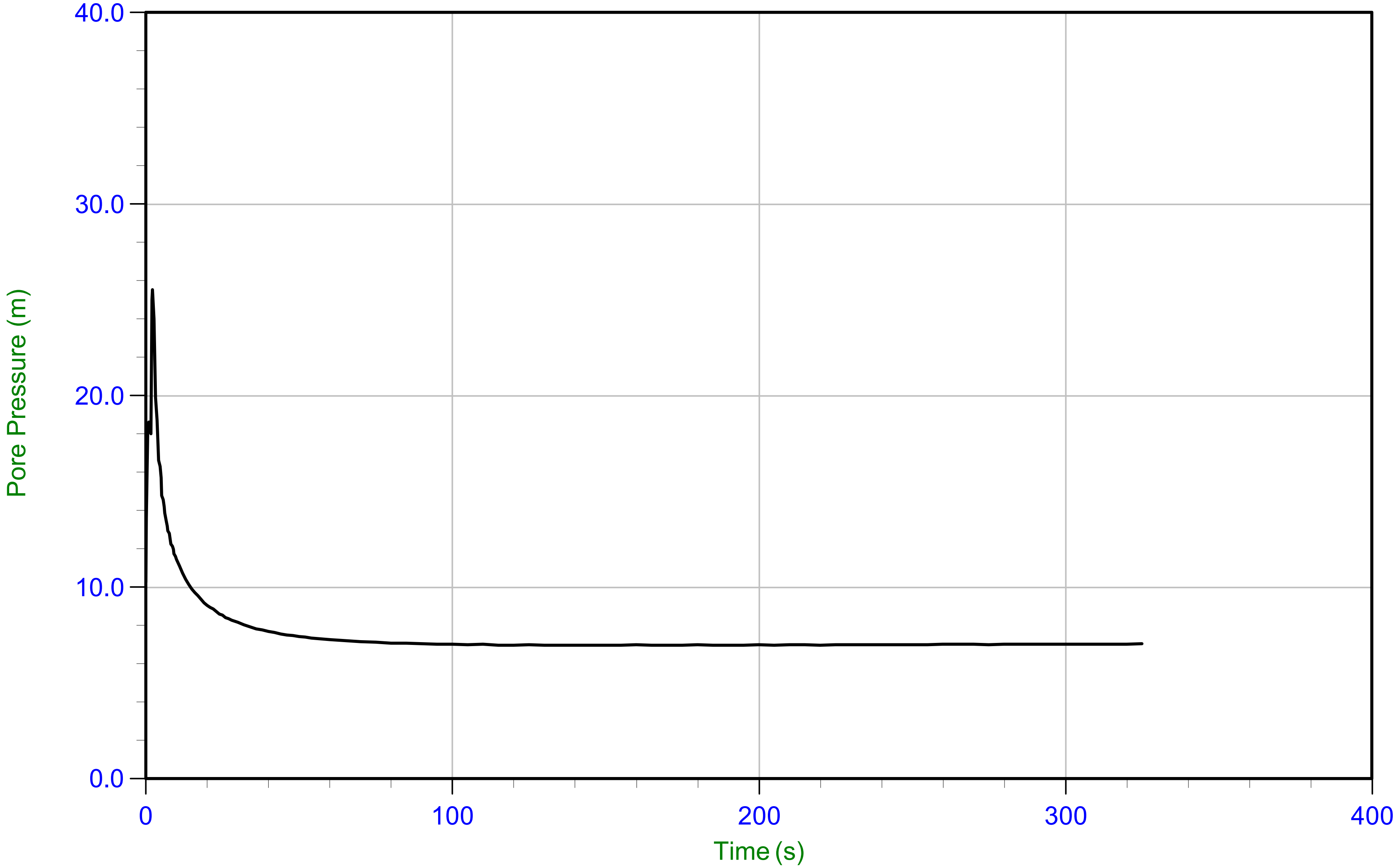
T(50): 3.9 s

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 09:36
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

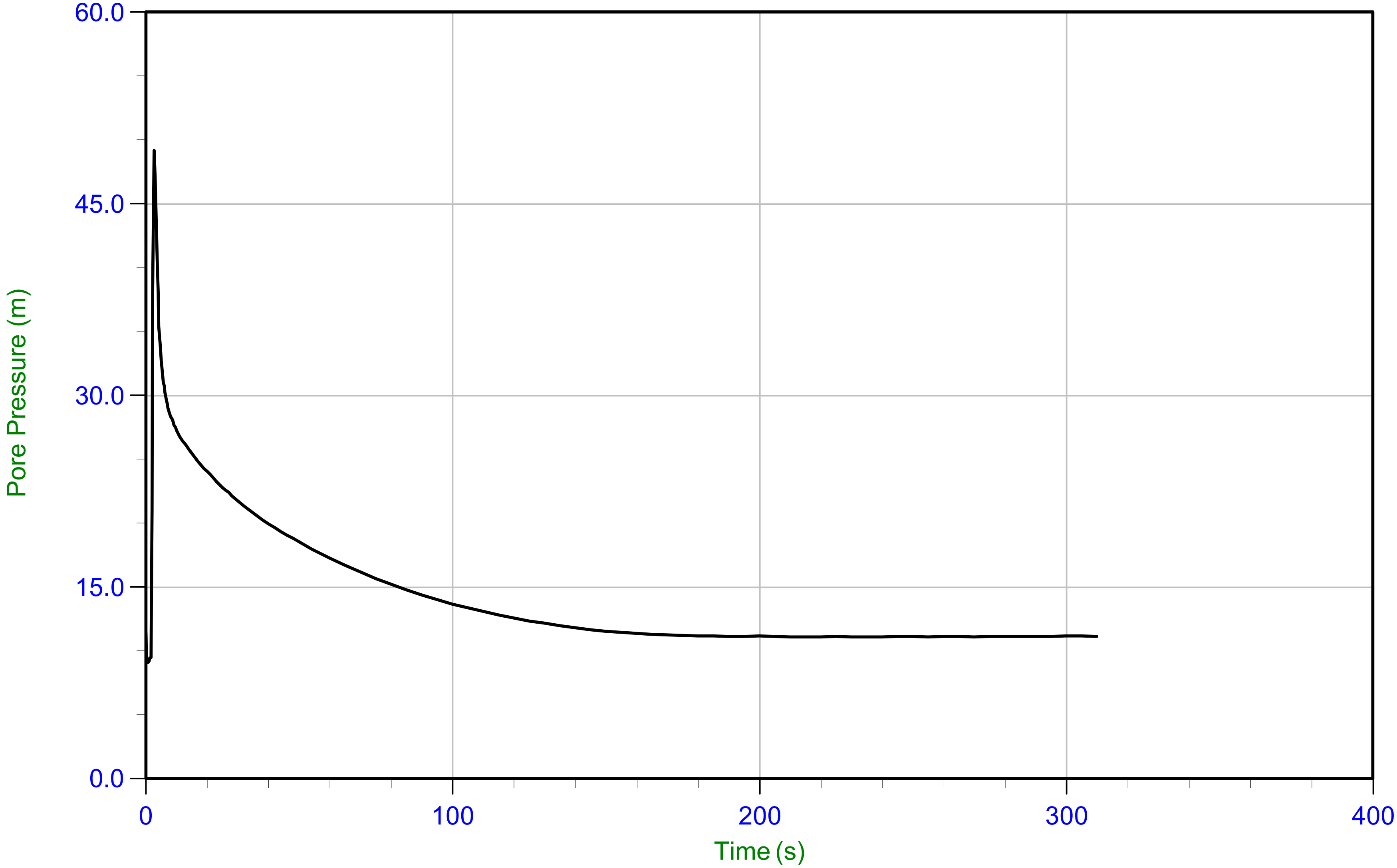
Filename: 23-05-25396_SP03.PPF2
Depth: 9.950 m / 32.644 ft
Duration: 325.0 s

u Min: 7.0 m
u Max: 25.5 m
u Final: 7.0 m

WT: 3.003 m / 9.852 ft
Ueq: 6.9 m
U(50): 16.24 m

T(50): 2.4 s

GHD	Job No: 23-05-25396	Sounding: SCPT23-03	
	Date: 02/27/2023 09:36	Cone: 824:T1000F10U35	Area=15 cm²
	Site: 126-140 Bradford St, Barrie		



Trace Summary:

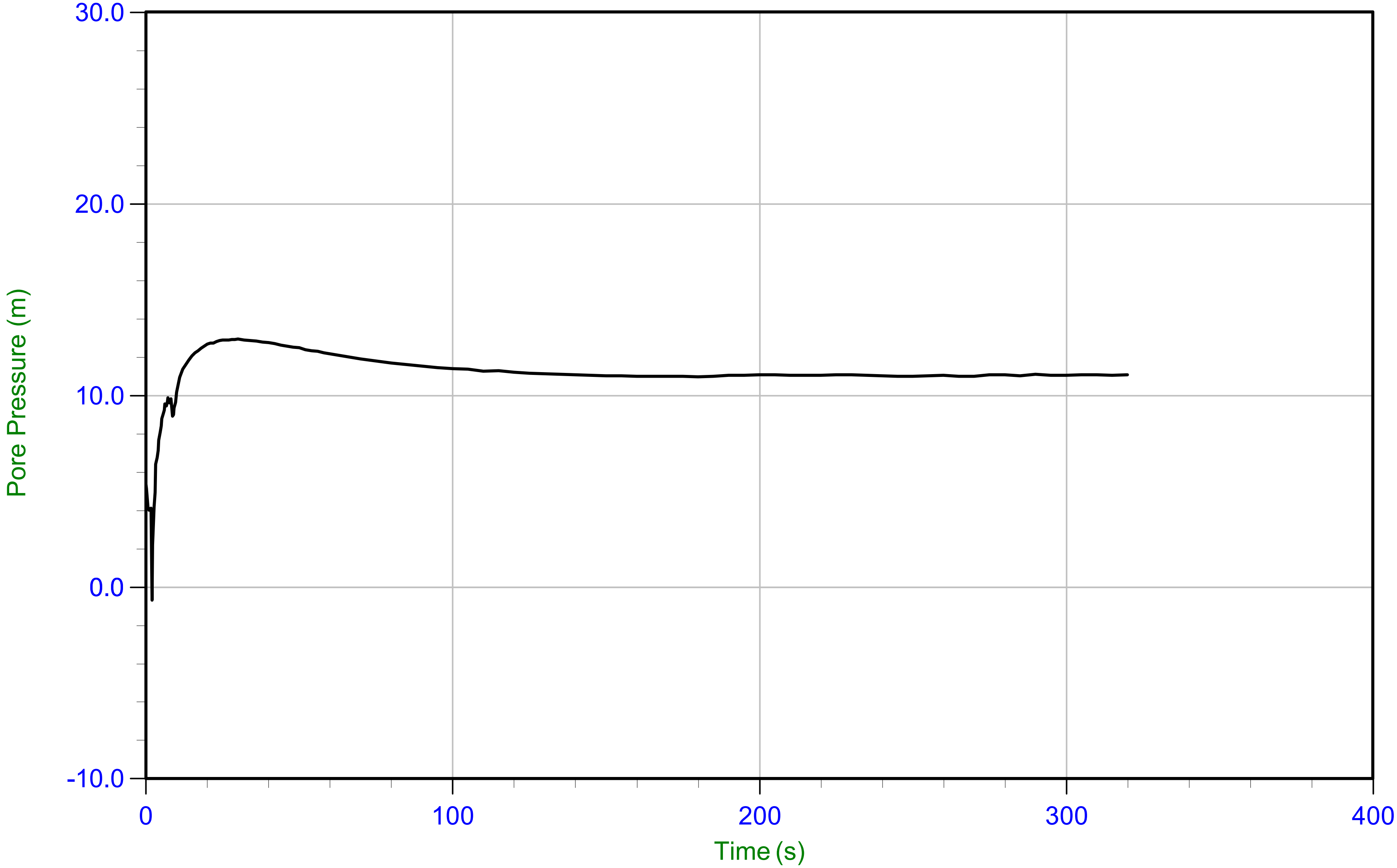
Filename: 23-05-25396_SP03.PPF2	u Min: 9.1 m	WT: 3.322 m / 10.899 ft	T(50): 3.7 s
Depth: 14.375 m / 47.162 ft	u Max: 49.2 m	Ueq: 11.1 m	
Duration: 310.0 s	u Final: 11.2 m	U(50): 30.13 m	

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 11:00
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-03B
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP03B.PPF2
Depth: 14.575 m / 47.818 ft
Duration: 320.0 s

u Min: -0.7 m
u Max: 12.9 m
u Final: 11.1 m

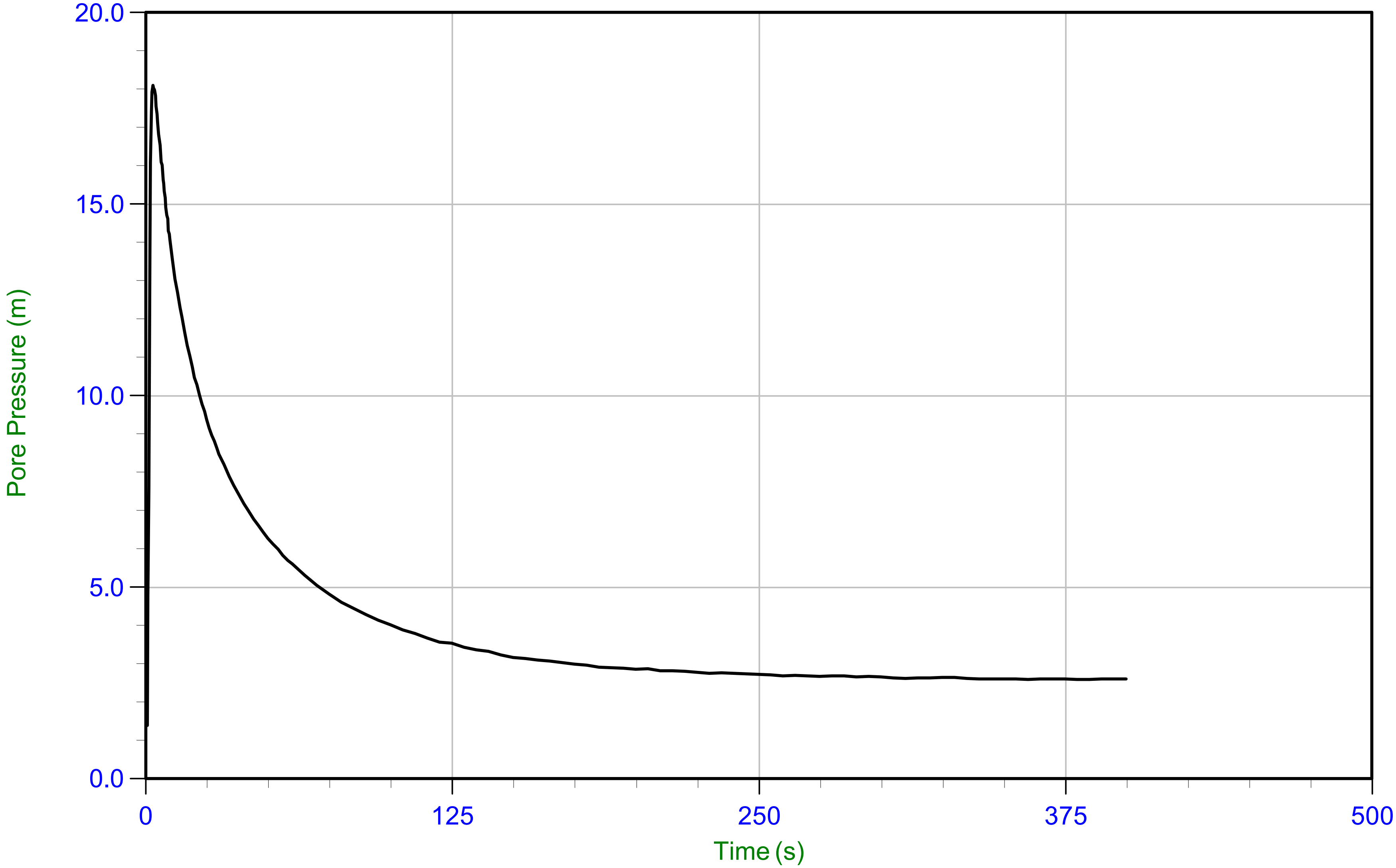
WT: 3.522 m / 11.555 ft
Ueq: 11.1 m

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 4.925 m / 16.158 ft
Duration: 400.0 s

u Min: 1.4 m
u Max: 18.1 m
u Final: 2.6 m

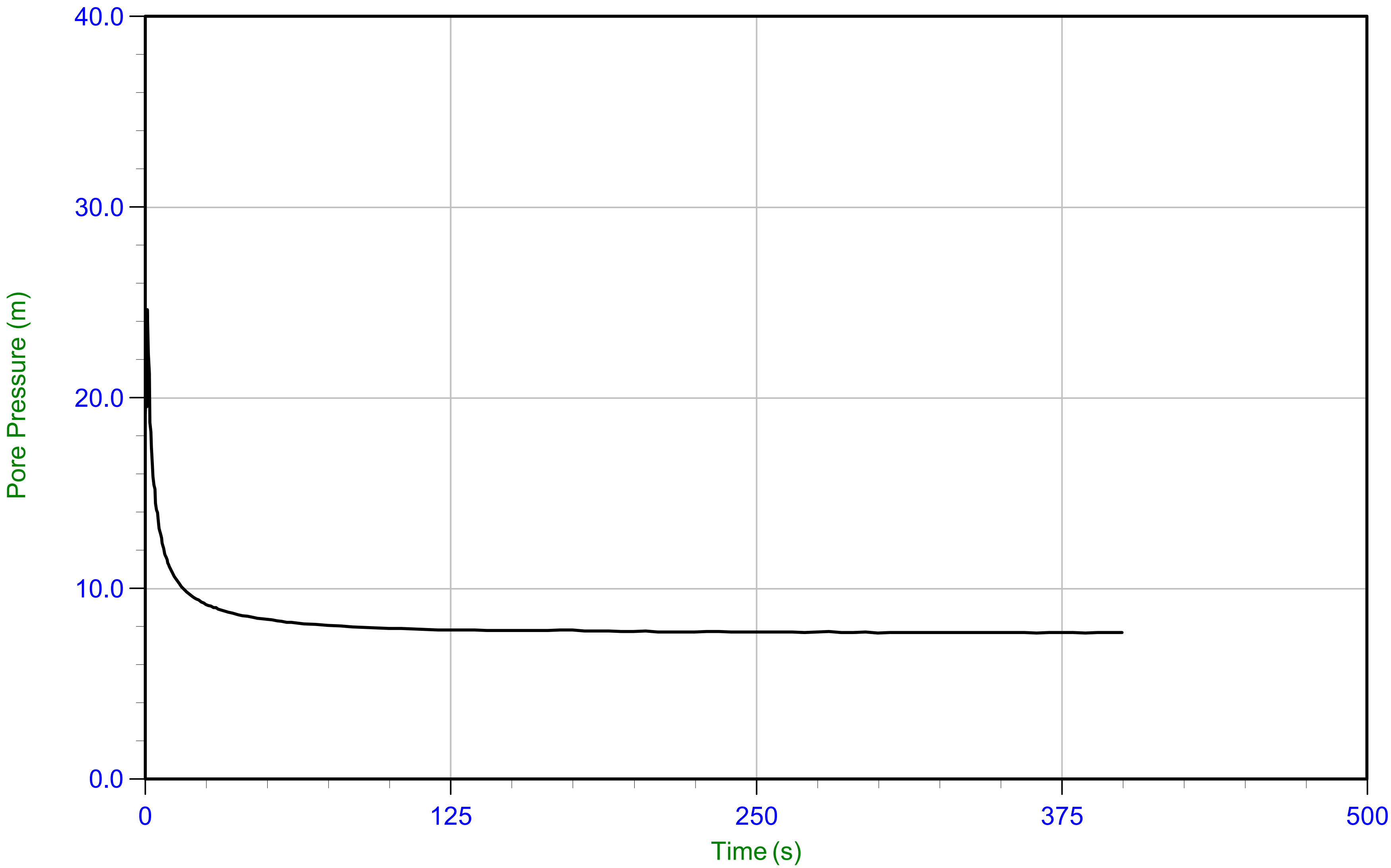
WT: 2.399 m / 7.871 ft
Ueq: 2.5 m
U(50): 10.31 m

T(50): 17.8 s

GHD

Job No: 23-05-25396
Date: 02/27/2023 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 9.900 m / 32.480 ft
Duration: 400.0 s

u Min: 7.7 m
u Max: 24.6 m
u Final: 7.7 m

WT: 2.321 m / 7.615 ft
Ueq: 7.6 m
U(50): 16.10 m

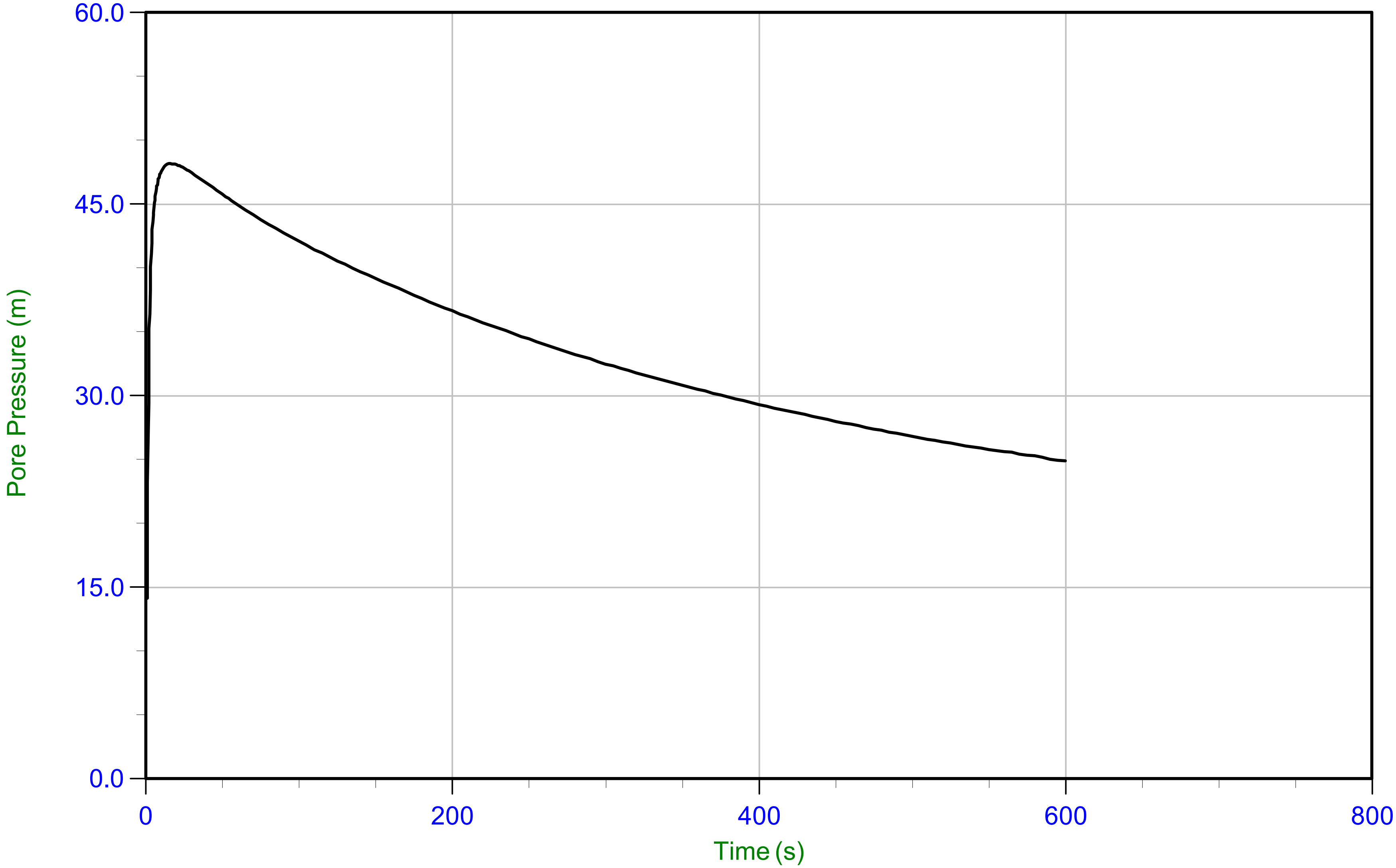
T(50): 2.2 s

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 14.900 m / 48.884 ft
Duration: 600.0 s

u Min: 14.2 m
u Max: 48.2 m
u Final: 24.9 m

WT: 2.321 m / 7.615 ft
Ueq: 12.6 m
U(50): 30.38 m

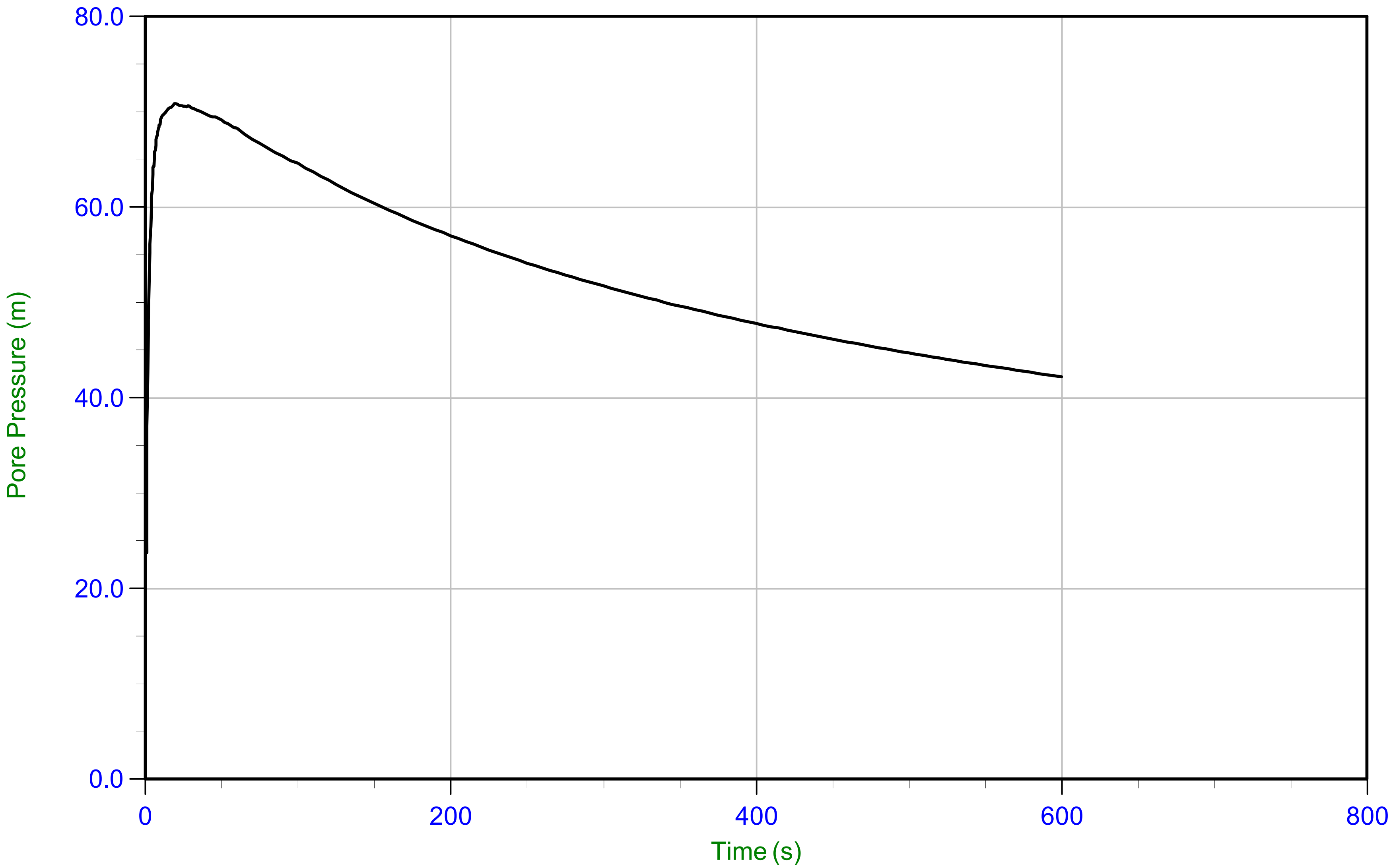
T(50): 348.4 s
Ir: 100
Ch: 2.0 cm²/min

CONETEC

GHD

Job No: 23-05-25396
Date: 02/27/2023 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 19.900 m / 65.288 ft
Duration: 600.0 s

u Min: 23.7 m
u Max: 70.9 m
u Final: 42.2 m

WT: 2.321 m / 7.615 ft
Ueq: 17.6 m
U(50): 44.22 m

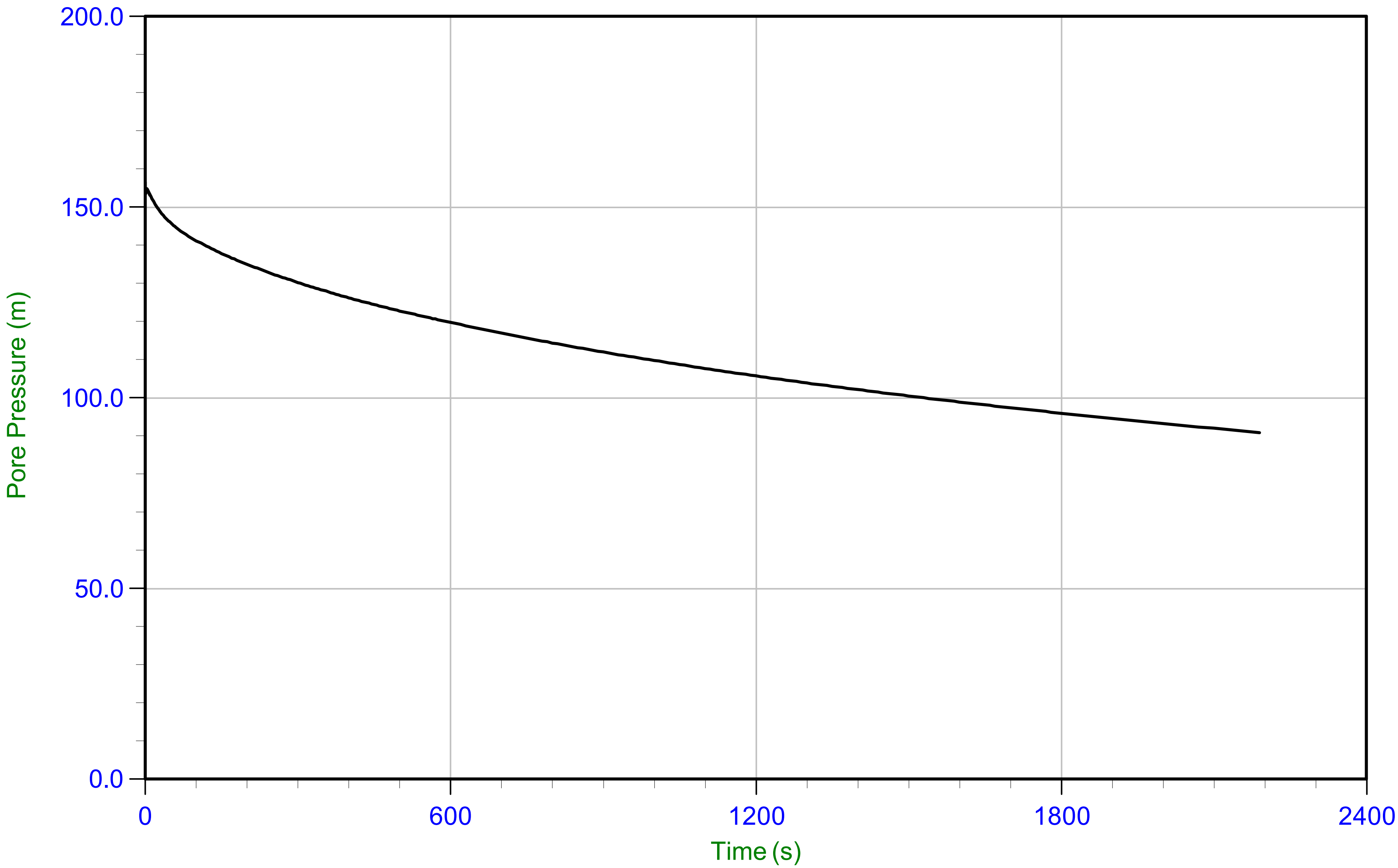
T(50): 498.3 s
Ir: 100
Ch: 1.4 cm²/min

ConeTec

GHD

Job No: 23-05-25396
Date: 02/27/2023 12:10
Site: 126-140 Bradford St, Barrie

Sounding: SCPT23-04
Cone: 824:T1000F10U35 Area=15 cm²



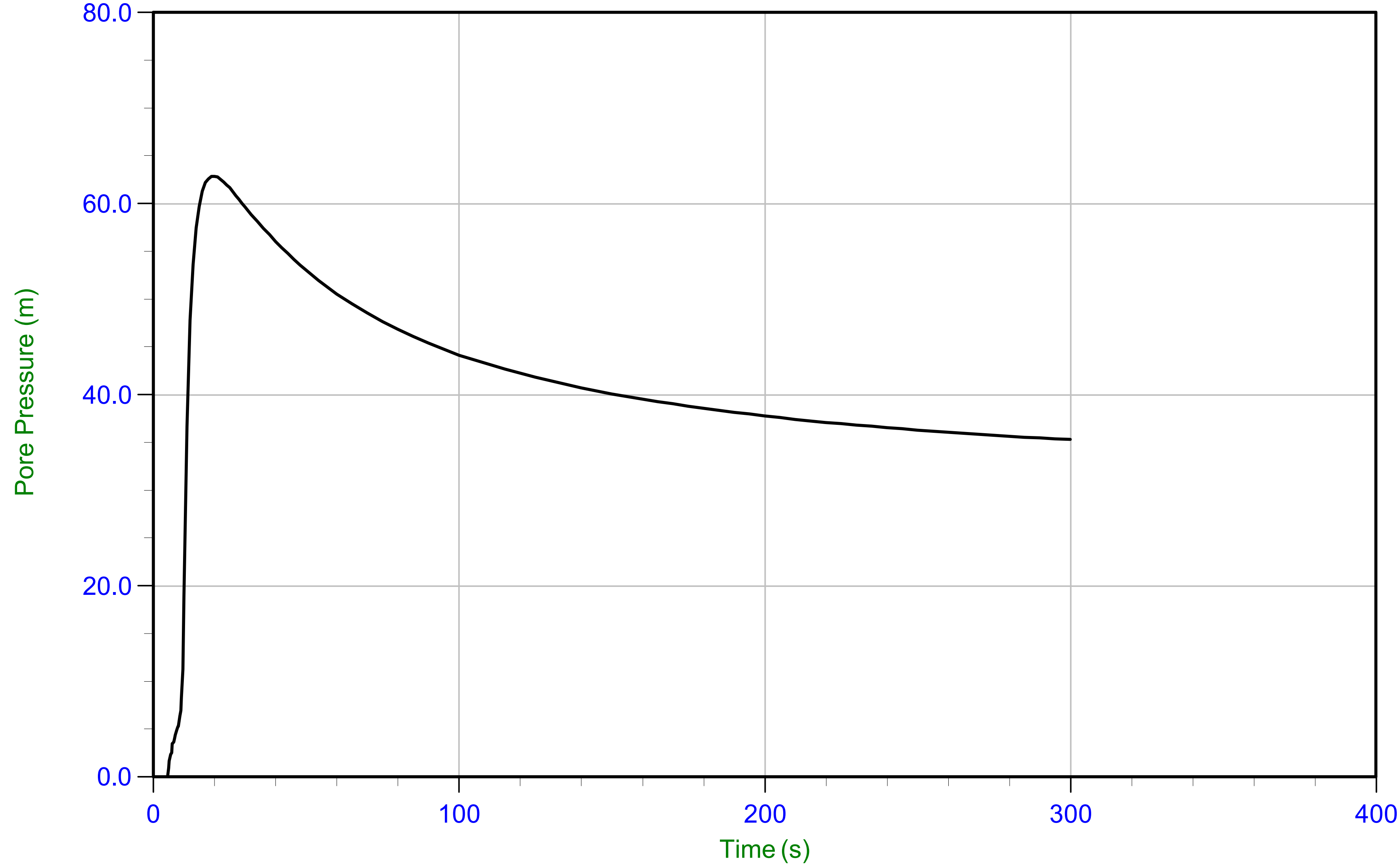
Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 24.900 m / 81.692 ft
Duration: 2190.0 s

u Min: 90.9 m
u Max: 156.6 m
u Final: 90.9 m

WT: 2.321 m / 7.615 ft
Ueq: 22.6 m

GHD	Job No: 23-05-25396 Date: 02/27/2023 12:10 Site: 126-140 Bradford St, Barrie	Sounding: SCPT23-04 Cone: 824:T1000F10U35 Area=15 cm²	



Trace Summary:

Filename: 23-05-25396_SP04.PPF2
Depth: 28.900 m / 94.815 ft
Duration: 300.0 s

u Min: -3.2 m
u Max: 62.9 m
u Final: 35.3 m

WT: 2.321 m / 7.615 ft
Ueq: 26.6 m
U(50): 44.73 m

T(50): 75.4 s
Ir: 100
Ch: 9.3 cm²/min

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 14

Revised November 26, 2019

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

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

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

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

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

The tip correction is: $q_t = q_c + (1\text{-}a) \bullet u_2$ (consistent units are implied)

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

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

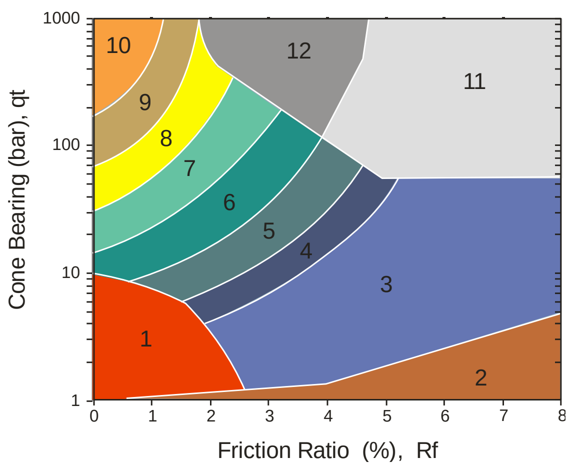
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the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

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

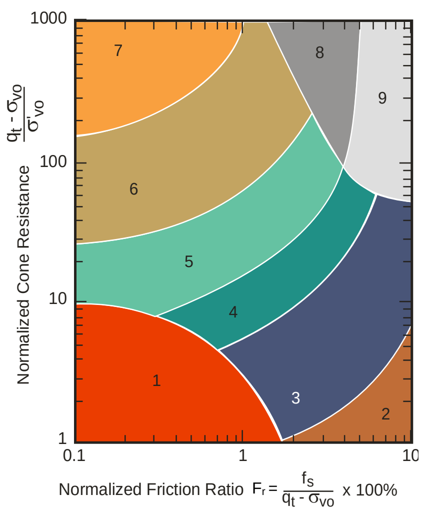


Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

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

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



Zone	Normalized Soil Behavior Type
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 sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

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

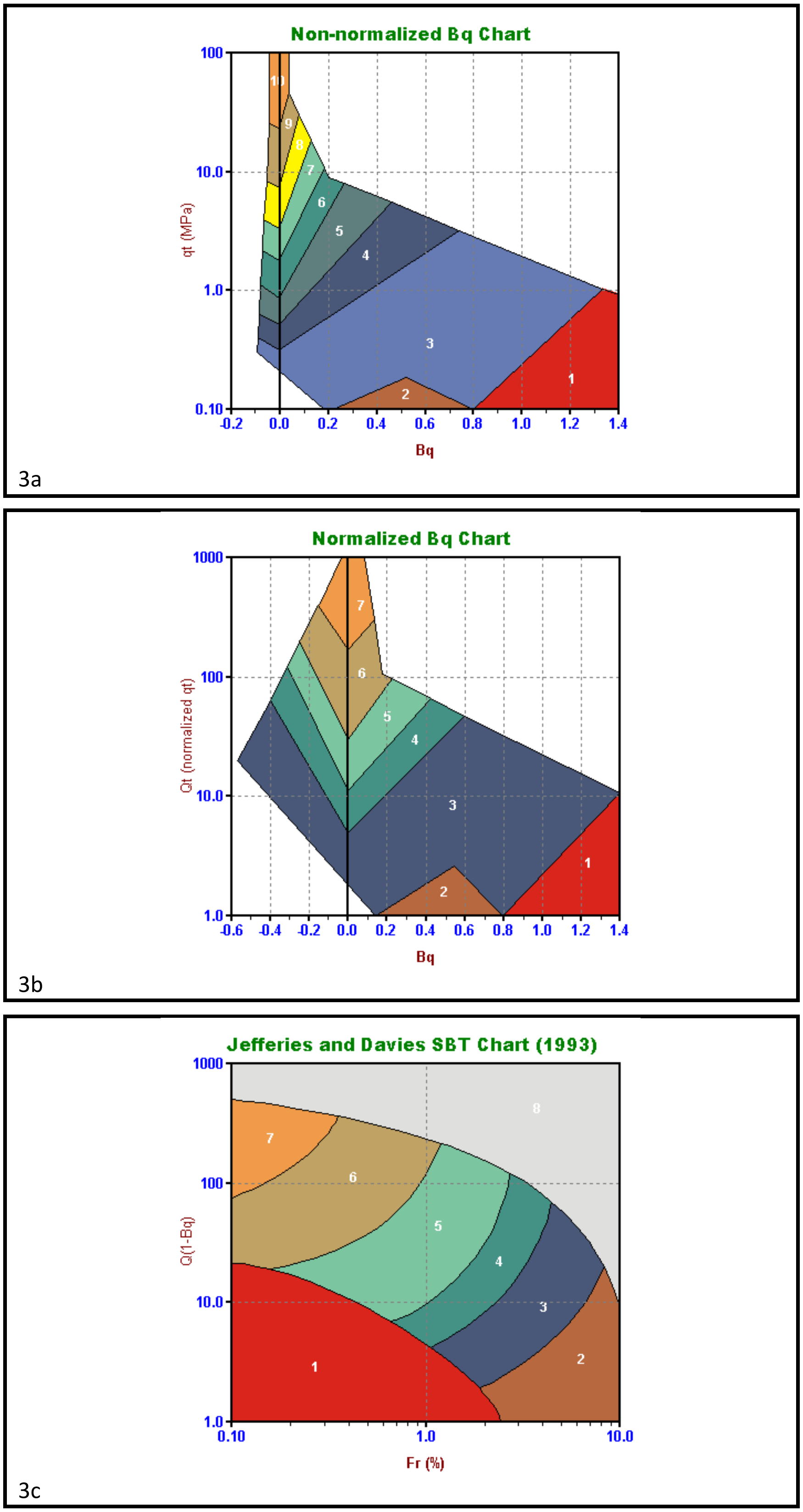


Figure 3. Alternate Soil Behavior Type Charts

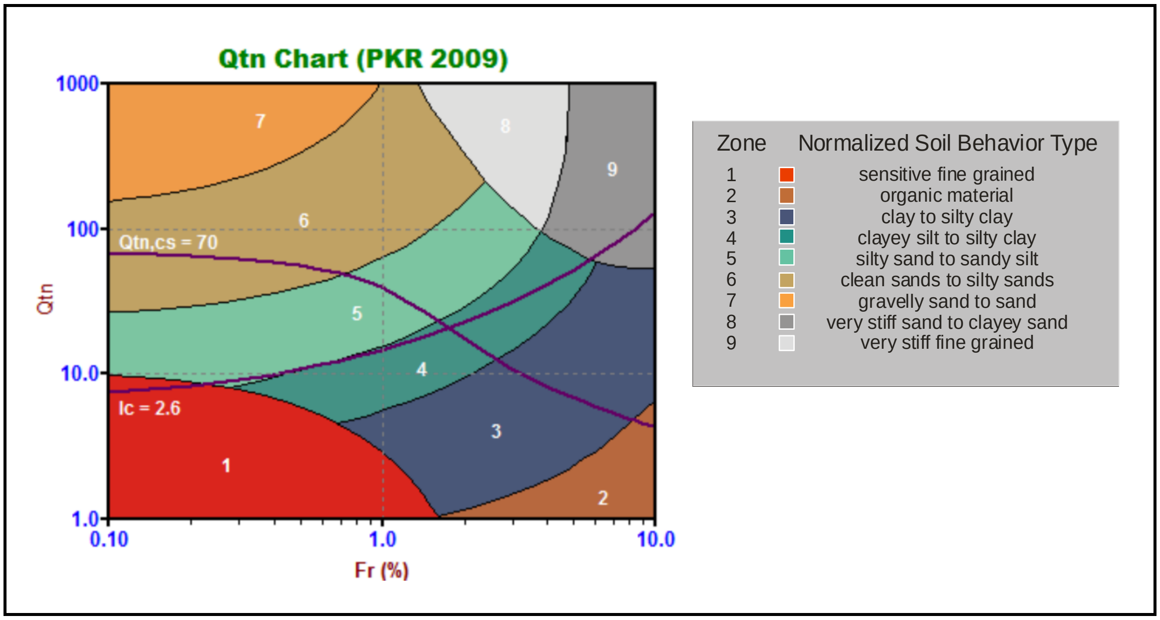


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

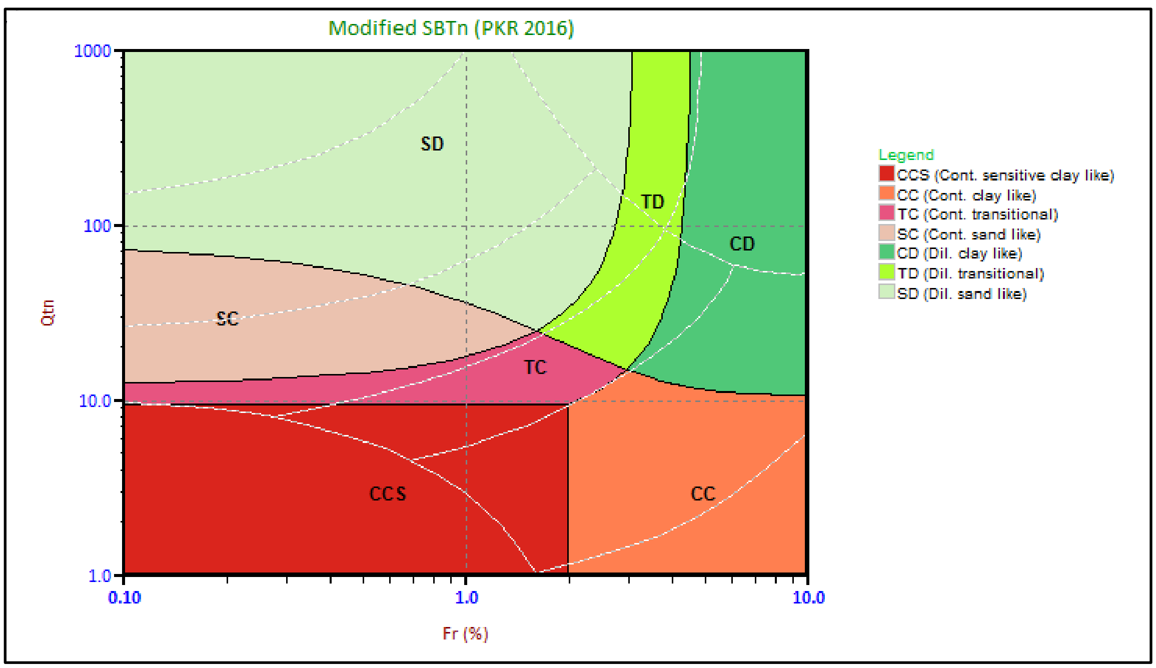


Figure 5. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.* *Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0	$K_0 = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a/\sigma_v')^{0.5}$ where *$0.0 < C_n < 2.0$ (user adjustable, typically 1.7)* *P_a is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)*	3, 12

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/\ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/\ N_{60} = 10^{\ (1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\,I_c)$ 2) $q_{c1n}/\ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/\ (N_1)_{60}I_c\ = 10^{\ (1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	$n\ (PKR\ 2009) = 0.381\ (I_c) + 0.05\ (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	 19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	 a) requires a user defined value for NC Su/Pc′ ratio b through f) *based on yield stresses* g) $OCR = 0.25\cdot(Qt)^{1.25}$	 9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}\left(\sigma'_v + \sigma'_h + \sigma'_h\right)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su\ (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\,(1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta = 1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta = 1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\dfrac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\,[(N_1)_{60}]^{4.79}$ $qc1$ is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\,[q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\,[q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

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No.	Reference
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Appendix E

MASW Test Results

MASW Investigation – Rev1

126-140 Bradford Street, Barrie, Ontario

Crown Communities Development Inc.

5 May 2023, Revised 4 December 2023

The Power of Commitment

Contents

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Appendices

1. Introduction

GHD was retained by Crown (Bradford) Developments Inc. (Crown) to conduct a Multichannel Analysis of Surface Waves (MASW) testing as part of the preliminary geotechnical investigation for the proposed development at 126-140 Bradford Street, Barrie, Ontario (Site).

As per email received from Crown dated November 27, 2023, GHD understands that the proposed development concept plan contains one (1) mixed-use building with two residential towers, each forty-five (45) storeys in height and a shared six (6) storey podium with commercial at-grade. As a whole, the proposal will result in a total gross floor area (GFA) of approximately 51,383 m^2, comprised of 51,049 m^2 of residential GFA and 333 m2 of retail/commercial GFA.

It is noted that no underground structure, such as underground parking levels, is planned for the development. The Site is currently utilized for multiple commercial operations, and occupied by the following businesses: NAPA Autocare Centre, Barrie Foot Clinic and Get Moving Physio. The Site location is shown on **Figure 1.**

It is expected that the proposed buildings will be surrounded by pavement structures. Further details regarding the development plans at this property have not been provided to GHD at the time of writing this report.

Multichannel Analysis of Surface Waves (MASW) is a geophysical testing method that uses surface wave (Rayleigh wave) propagation to determine the subsurface profile. The purpose of the MASW survey was to assist with the seismic site class determination by measuring the average shear wave velocity approximately within the upper 30 m of the soil/rock profile below the founding elevation of the proposed structure at the Site. The shear wave velocity measurements were carried out along two MASW survey lines assumed to be representative of the Site. The location of investigation lines is shown in the attached **Figure 1**.

Based on the geotechnical investigation borehole logs provided in **Appendix A** of GHD (2023) preliminary geotechnical report, the reported soil profile in the advanced boreholes near the proposed development and the MASW lines generally consists of a loose to compact cohesionless fill layer of sand. The fill layer extends to a depth of approximately between 2.3 m below ground surface (Elevation 221.3). Underneath the fill, a generally very loose to very dense silt/sandy silt/silty sand/sand and silt/sand deposit was encountered with interbedded cohesive silty clay layers. The cohesionless soil deposit extended to the termination depth of investigation in all boreholes. All investigative boreholes were advanced to about 28 m below ground surface (BH5-23 and BH6-23 and shown in **Figure 1**) and the deepest SCPT was advanced to a depth of 28 m below ground surface (SCPT23-04 as shown in **Figure 1**). No bedrock was encountered in GHD (2023) preliminary geotechnical investigation. However, based on a review of geologic mapping, the bedrock topography at the Site as indicated on the map titled Ontario Division of Mines, Preliminary Map No. P.979 Bedrock Topography Series Barrie Area Southern Ontario, it is our understanding that the bedrock elevation at this Site is anticipated to be approximately 100 m below ground surface. The described relative density/consistency terms and soil classification in this section are based on the recorded SPT “N” values and soil descriptions provided on the GHD (2023) geotechnical borehole logs.

2. MASW Procedure

To carry out the MASW test, 24 transducers (geophones) are deployed along a line at certain distances from a seismic source. The length of the geophone array determines the deepest investigation depth that can be obtained from the measurements. The source should produce enough seismic energy over the desired test frequency range to allow for detection of Rayleigh waves above background noise (Park et al., 1999[1]). A common seismic source is either a sledgehammer or a drop weight hitting a metallic or rubber base plate set at ground surface. The existing traffic

[1] Park, C. B., Miller, R. D., & Xia, J. (1999). Multichannel analysis of surface waves. Geophysics, 64(3), 800-808.

noise or the noise generated by heavy machinery travelling close to the survey line can also be utilized as a source for investigating deep soil layers. For this site, only active seismic source is used. **Figure 2.1** shows a typical MASW setup.

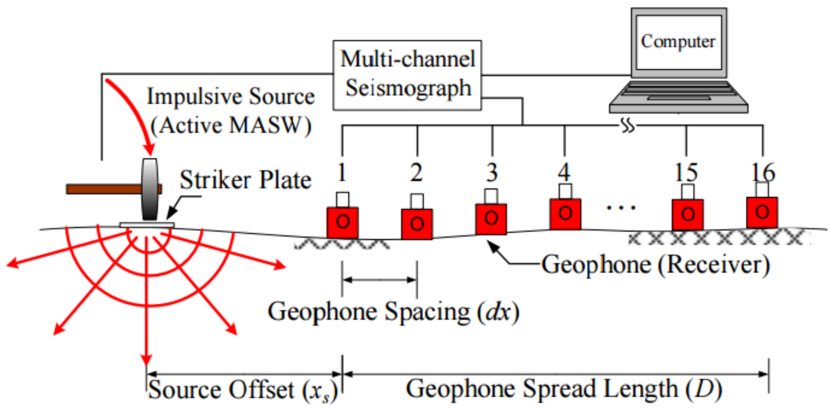


Figure 2.1: Schematic Layout of MASW Test Setup (Sahadewa et al., 2012[2])

3. Fieldwork

The fieldwork for this MASW investigation program was carried out on March 2nd, 2022, by GHD professionals. The field data was collected using a 24-channel seismograph (Geometrics Geode 24 console #3389), twenty-four 4.5 Hz geophones, and one 24 take-out cable with 5 m spacing. A Panasonic Toughbook© laptop was used in the field to record and collect the seismic data utilizing Geometrics single geode OS controller version 9.14.0.0.

The survey was carried out along two survey lines in the footprint of the proposed development as shown on **Figure 1** attached to this report. For all survey lines, the geophones were installed 75 mm into the ground by manually pushing them into position.

A multi geometry approach was utilized for data collection along all lines. The active data sets were collected using a 4.5 kg sledgehammer hitting the ground surface at three different offset distances (distance between the source and first geophone) along each survey line. The following table summarizes the geometry for each investigation line.

Table 1 MASW Lines Geometry

Line No.	Designation	Geophone Spacing (m)	Array Length (m)	Offset Distances (m)
Line 1	Long	2.0	46.0	30.0, 20.0, 10.0
Line 1	Short	1.0	23.0	15.0, 10.0, 5.0
Line 2	Long	2.0	46.0	30.0, 20.0, 10.0
Line 2	Short	1.0	23.0	15.0, 10.0, 5.0

Three sets of data files (active) were collected for each array location/set up. For the active survey measurements, the ground vibrations were recorded for four seconds with one sample per 0.25 ms.

[2] Sahadewa, A., Zekkos, D., & Woods, R. D. (2012). Observations from the implementation of a combined active and passive surface wave based methodology. In GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering (pp. 2786-2795).

4. Data Interpretation

MASW method utilizes the frequency-dependent properties of Rayleigh surface waves in order to develop the profile of shear wave velocity versus with depth. This method includes three stages as shown in **Figure 4.1**. In this project, generation of dispersion curves, inversion of the obtained dispersion curves and development of the 1D shear wave velocity profiles were carried out using SurfSeis© version 6.0. The dispersion curves were calculated at the middle stations along each line. At each investigation line, the dispersion images obtained from active data at different offsets were stacked to obtain a combined dispersion curve. The data inversion was carried out using a 10-layer soil velocity numerical model to obtain 1D shear wave velocity profiles at the location of each mid station. The calculated 1D velocity profile along the investigation lines is shown on the attached Shear Wave Velocity Profile. **Figure 2** shows the obtained results at the location of the proposed development. As it can be seen in this figure, values of shear wave velocity for Line 1 and Line 2 are relatively consistent in depth. The data obtained from the advanced boreholes also confirms a consistent subsurface soil profile in the vicinity of the MASW lines. The stratigraphy borehole logs are provided in **Appendix A** of the GHD (2023) preliminary geotechnical investigation report. For all investigation lines, the shear wave velocity increases with depth indicating values higher than 360 m/s below approximate depths of 15 m.

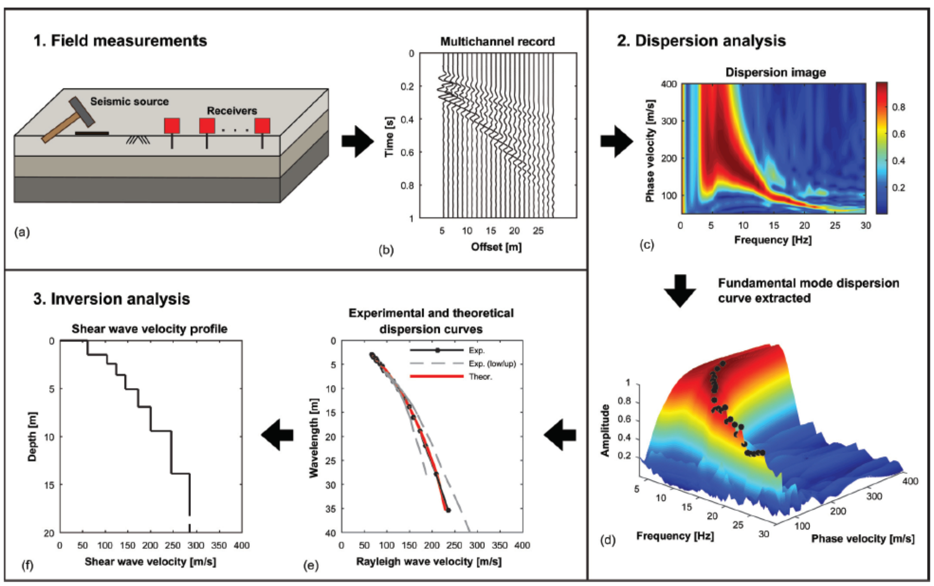


Figure 4.1: Overview of MASW method (Olafsdottir et al., 2018[3])

In accordance with the requirements of Ontario Building Code (OBC 2012) and National Building Code of Canada (NBC 2020), the variation of the measured shear wave velocity versus depth up to 30 m below the proposed founding level of the buildings (assumed to be 0 m below existing ground surface for this project) was obtained along each line and is shown in **Table 1-A** and **Table 1-B** attached to this report. The average shear wave velocity within the upper 30 m of the soil/rock profile (Vs_{30}) immediately below the founding level of the building (assumed to be at ground surface) were obtained utilizing the averaging scheme introduced in Sentence 4.1.8.4 (2) of NBC (2020) User's Guide.

Based on the calculations presented in **Table 1** attached to this report, the average shear wave velocity (from 0.0 m below ground surface to 30.0 m below ground surface) along the two investigation lines is **327 m/s**. Therefore, in accordance with Table 4.1.8.4.A of the OBC 2012 (**Table 2** attached to this report) and based on the measured average shear wave velocity, the Site can be classified as **Class 'D'** for the seismic load calculations.

[3] Olafsdottir, E. A., Erlingsson, S., & Bessason, B. (2018). Tool for analysis of multichannel analysis of surface waves (MASW) field data and evaluation of shear wave velocity profiles of soils. Canadian Geotechnical Journal, 55(2), 217-233.

Based on available geotechnical information from the advanced boreholes in the Site, the deepest investigative borehole was advanced to approximately 28 m below ground surface (BH5-23 and BH6-23 as shown on **Figure 1**) and no bedrock was encountered on boreholes advanced by GHD (2022). As per OBC 2012, Site Classes A and B can be considered where there is no more than 3.0 m of overburden between the bedrock and the underside of footing.

In addition, based on the average shear wave velocity provided in **Table 1** and in accordance with Table 4.1.8.4.A and Section 4.1.8.4.(2) of the NBC 2020, site designation is determined using the average shear wave velocity Vs30, calculated from in situ measurements of shear wave velocity. For ground profile which contains more than 3.0 m of softer materials between rock and underside of footing or mat foundation, the site designation shall be Xv, where V is the value of Vs30. As a result, a **Site Designation of X_{327}** can be assigned for seismic load calculations.

The recommended site class is only applicable if site conditions for Site Class F (liquefiable soil/soft soil layers more than 3.0 m thick) are not application. Else, Site Class and Site Designation should be revised accordingly.

The seismic site classification provided in this report is based solely on the shear wave velocity values derived from the MASW method and that it can be superseded by other geotechnical information as per requirement from NBC (2020).

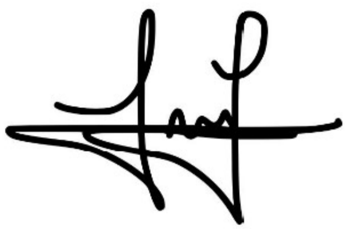
The seismic hazards for the site as obtained from the 2020 National Building Code of Canada Seismic Hazard Tool website are provided as **Appendix A** to this correspondence.

5. Closure

It is important to emphasize that the results and conclusions of the MASW analysis are based on the available geotechnical information and the survey conducted along the two investigation lines. Should any conditions at the Site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations.

All of Which is Respectfully Submitted,

GHD



Brice Zanne, M.Eng., EIT

Ali Ghassemi, Ph.D., P.Eng

Figures

LEGEND

BOREHOLE LOCATION

CPT LOCATION

MONITORING WELL LOCATION (GHD, NOVEMBER 2022)

MASW LINE

0 5 10 15m

1:500

Coordinate System:
NAD 83 UTM ZONE 17
METRES

N

GHD

CROWN (BRADFORD) DEVELOPMENTS INC.
126 AND 140 BRADFORD STREET,
BARRIE, ONTARIO

BOREHOLE / CPT / MASW LINE
LOCATION PLAN

Project No. 12593831
Date December 2023

FIGURE 1

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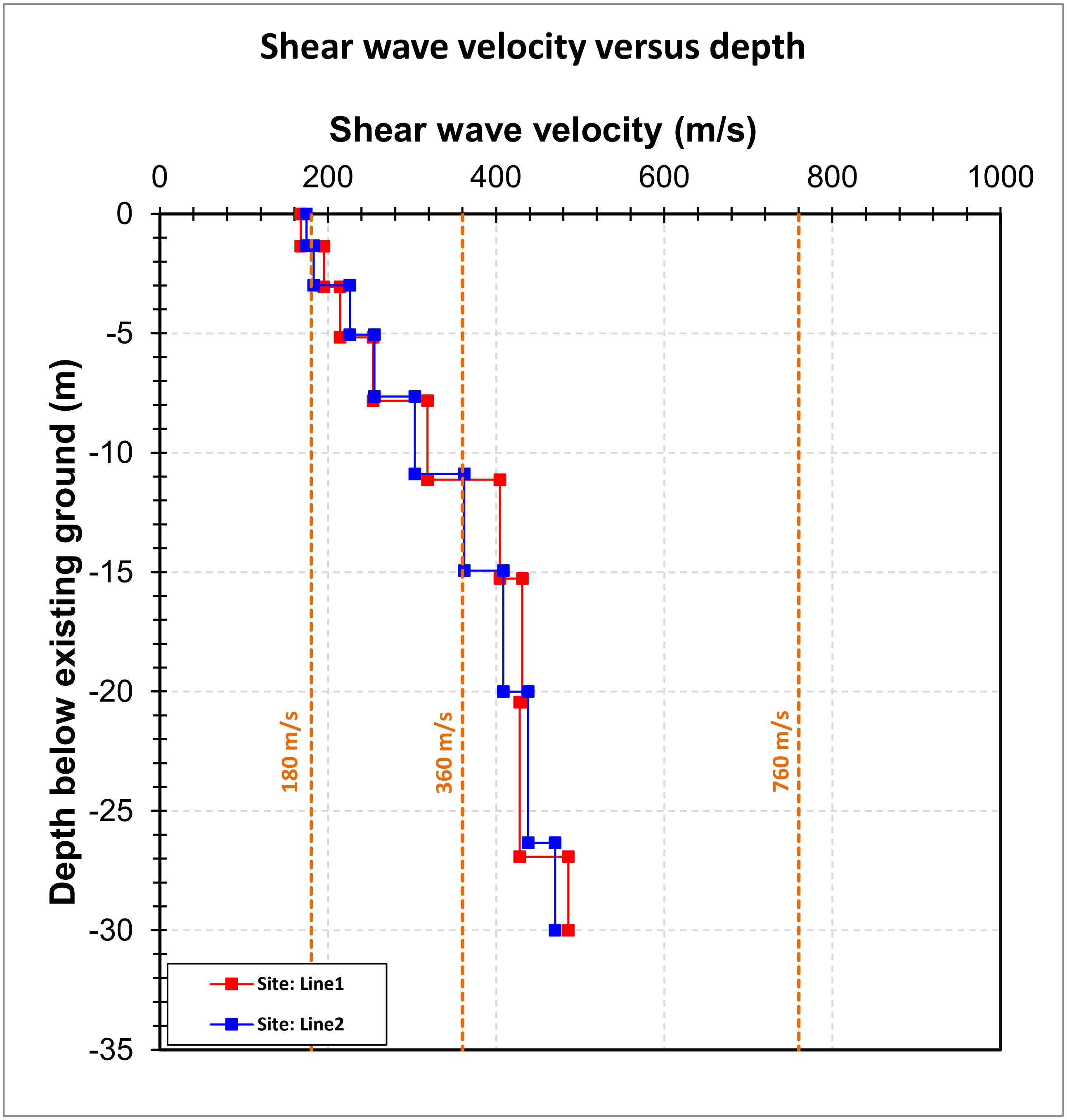
Data Source: Image ©2022 Google (Imagery Date 7/24/2019).



Crown (Bradford) Developments Inc.
Preliminary Geotechnical Invesitigation
126-140 Bradford street, Barrie, Ontario

SHEAR WAVE VELOCITY VS DEPTH

PROJECT NO.
12593831
DATE
04-Dec-23

FIGURE NO. 2

Tables

Table 1
Summary of Shear Wave Velocity Measurements
Seismic Site Class Determination
Preliminary Geotechnical Investigation
Crown (Bradford) Developments Inc.
126-140 Bradford street, Barrie, Ontario

Table 1-A: Average Shear Wave Velocity (VS_{30})
(Assumed foundation below existing ground surface)
Line 1

Layer No.	Depth (m bgs) From	Depth (m bgs) To	Thickness m	V_s m/s	d_i/V_{si}
1	0.0	1.4	1.4	167	0.0081
2	1.4	3.1	1.7	195	0.0087
3	3.1	5.2	2.1	214	0.0099
4	5.2	7.8	2.7	254	0.0104
5	7.8	11.1	3.3	318	0.0104
6	11.1	15.3	4.1	404	0.0102
7	15.3	20.5	5.2	431	0.0120
8	20.5	26.9	6.5	428	0.0151
9	26.9	30.0	3.1	486	0.0063
Total			30.0		0.0912
Average Shear Wave Velocity Along the Line (m/s)					***329***

Table 1-B: Average Shear Wave Velocity (VS_{30})
(Assumed foundation below existing ground surface)
Line 2

Layer No.	Depth (m bgs) From	Depth (m bgs) To	Thickness m	V_s m/s	d_i/V_{si}
1	0.0	1.3	1.3	174	0.0076
2	1.3	3.0	1.7	183	0.0091
3	3.0	5.1	2.1	226	0.0092
4	5.1	7.7	2.6	255	0.0102
5	7.7	10.9	3.2	303	0.0107
6	10.9	14.9	4.1	362	0.0112
7	14.9	20.0	5.1	409	0.0124
8	20.0	26.3	6.3	438	0.0144
9	26.3	30.0	3.7	470	0.0078
Total			30.0		0.0925
Average Shear Wave Velocity Along the Line (m/s)					***324***

Average VS_{30} = 327 m/s

Recommended Minimal Site Designation (NBCC 2020) : **X_{327}** Subjected to Code requirements

Notes:
1 - The Seismic Site designation is recommended in accordance to Table 4.1.8.4.A of the National Building code of Canada 2020 (NBCC 2020), section 4.1.8.4 (2) and based on the measured average shear wave velocity measured along the investigated lines.
2 - VS30 is the average shear wave velocity in top 30 m below the proposed founding elevation calculated from in situ measurements.
3 - Ground profile which contains no more than 3 m of softer materials between rock and underside of footing or mat foundation, the site designation shall be Xv, where V is the value of VS30.

Recommended Minimal Site Class (OBC 2012) : **D** Subjected to Code requirements

Notes:
1 - The Seismic Site class is recommended in accordance to Table 4.1.8.4.A of the Ontario Building Code (OBC 2012, O.Reg 332/12) and based on the measured average shear wave velocity measured along the investigated lines.
2 - VS30 is the average shear wave velocity in top 30 m below the proposed founding elevation calculated from in situ measurements.
3 - Site Classes A and B are only applicable if footings are founded on bedrock or there is no more than 3.0 m of soil between founding elevation and bedrock.
4 - The recommended site class is only applicable if site conditions for Site Class F (liquefiable soil/soft soil layers more than 3.0 m thick) are not applicable.

Table 2

Site Classification for Seismic Site Response
Forming Part of Sentences 4.1.8.4. (1) to (3)

	Ground Profile Name	Average Properties in Top 30 m		
		Average Shear Wave Velocity, $\bar{V}_s$ (m/s)	Average Standard Penetration Resistance, $\bar{N}_{60}$	Soil Undrained Shear Strength, s_u
A	Hard rock	$\bar{V}_s > 1500$	N/A	N/A
B	Rock	$760 < \bar{V}_s \leq 1500$	N/A	N/A
C	Very dense soil and soft rock	$360 < \bar{V}_s < 760$	$\bar{N}_{60} > 50$	$s_u > 100$ kPa
D	Stiff soil	$180 < \bar{V}_s < 360$	$15 \leq \bar{N}_{60} \leq 50$	50 kPa $< s_u \leq$ 100 kPa
E	Soft soil	$\bar{V}_s < 180$	$\bar{N}_{60} \leq 15$	$s_u < 50$ kPa
		Any profile with more than 3m of soil with the following characteristics: plasticity index: PI > 20 moisture content w ≥ 40%, and undrained shear strength: $s_u < 25$ kPa		
F	Other soils	Site-specific evaluation required		

Reference: *2012 Ontario Building Code Compendium, Division B – Part 4, Section 4.1.8.4.*

Appendices

Appendix A

NBC Seismic Hazard and Site Classification for Seismic Site Response

Government of Canada Gouvernement du Canada

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_V	X_{327}
Latitude (°)	44.38
Longitude (°)	-79.694

Please select one of the tabs below.

NBC 2020 | Additional Values | Plots | API | Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_{327})$	$S_a(0.5, X_{327})$	$S_a(1.0, X_{327})$	$S_a(2.0, X_{327})$	$S_a(5.0, X_{327})$	$S_a(10.0, X_{327})$	$PGA(X_{327})$	$PGV(X_{327})$
0.206	0.159	0.0925	0.0452	0.0123	0.00425	0.0964	0.103

The log-log interpolated 2%/50 year $S_a(4.0, X_{327})$ value is : **0.0169**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_{327})$	$S_a(0.5, X_{327})$	$S_a(1.0, X_{327})$	$S_a(2.0, X_{327})$	$S_a(5.0, X_{327})$	$S_a(10.0, X_{327})$	$PGA(X_{327})$	$PGV(X_{327})$
0.125	0.0969	0.0548	0.026	0.00657	0.0023	0.058	0.0584

The log-log interpolated 5%/50 year $S_a(4.0, X_{327})$ value is : **0.0092**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_{327})$	$S_a(0.5, X_{327})$	$S_a(1.0, X_{327})$	$S_a(2.0, X_{327})$	$S_a(5.0, X_{327})$	$S_a(10.0, X_{327})$	$PGA(X_{327})$	$PGV(X_{327})$

$S_a(0.2, X_{327})$	$S_a(0.5, X_{327})$	$S_a(1.0, X_{327})$	$S_a(2.0, X_{327})$	$S_a(5.0, X_{327})$	$S_a(10.0, X_{327})$	$PGA(X_{327})$	$PGV(X_{327})$
0.0812	0.0625	0.0344	0.0157	0.0037	0.0013	0.0373	0.0354

The log-log interpolated 10%/50 year $S_a(4.0, X_{327})$ value is : **0.0053**

Download CSV

Go back to the seismic hazard calculator form

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