Geotechnical Investigation Report - 272 Innisfil Street, Barrie, Ontario

2022-04-12

Prepared for:
Ollie Switch Developments

Reference No.: 8707-003

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1.0 INTRODUCTION

Cambium Inc. (Cambium) was retained by Ollie Switch Developments (Client) to complete a supplementary geotechnical investigation in support of the design and construction of a 17-storey residential development located at 272 Innisfil Street in Barrie, Ontario (Site). The development will include four storeys of above-ground parking.

This additional geotechnical investigation consists of two additional boreholes and three cone penetration tests (CPT), with the purpose of supplementing the results of the geotechnical investigation conducted by Cambium in July and August 2019. The additional borehole and CPT locations were situated within the current proposed footprint of the development, which had been adjusted since the initial investigation.

The geotechnical report presenting the results of the borehole investigation conducted in 2019 forms the basis of this report. This report includes the methodology and findings of both geotechnical investigations conducted at the Site, with geotechnical findings and recommendations adjusted as necessary based on the information gained during the supplementary investigation.

1.1 Site Description

The Site is located at the southwest corner of the intersection of Innisfil Street and Jacobs Terrace in Barrie, Ontario and covers an area of approximately 7,000 m^2. Approximately 3,000 m^2 of the parcel area is developed with an existing building consisting of one to three above-ground storeys and one underground storey covering an unknown extent of the existing building's footprint. The underground storey is accessible from the building exterior by a concrete ramp on the south side of the existing development.

The remainder of the property is paved, with driveway access from both Innisfil Street and Jacobs Terrace. The Site is bordered to the west and south by other industrially developed properties, to the east by Innisfil Street and to the north by Jacobs Terrace. The ground surface at the Site is relatively constant throughout the parcel, except for lower elevations in the loading dock areas on the south side of the existing structure.

2.0 METHODOLOGY

2.1 General

Borehole drilling and sampling were completed during both investigations conducted by Cambium at the site using a track-mounted drill rig operating under the supervision of a Cambium Geotechnical Analyst. The boreholes were advanced to the sampling depths by means of continuous flight hollow stem augers with 50 mm O.D. split spoon samplers. Given the encountered soil conditions, mud rotary techniques were utilised in deeper boreholes for stabilizing purposes.

Standard Penetration Test (SPT) N values were recorded for the sampled intervals as the number of blows required to drive a split spoon sampler 305 mm into the soil, using a 63.5 kg drop hammer falling 750 mm, as per ASTM D1586 procedures. The SPT N values are used in this report to assess the consistency of cohesive soils and relative density of non-cohesive materials. Soil samples were collected at approximately 0.75 m intervals in the upper 3.0 m, at 1.5 m intervals between 3 mbgs and 16 mbgs, and at 3 m intervals below 16 mbgs. The encountered soil units were logged in the field using visual and tactile methods, and samples were placed in labelled plastic bags for transport, future reference, possible laboratory testing, and storage.

Open boreholes were checked for groundwater and general stability prior to backfilling. All boreholes were backfilled and sealed in accordance with Ontario Regulation (O.Reg.) 903, as amended, and the property was reinstated as close as reasonably possible to pre-existing conditions.

The borehole logs are provided in Appendix A (2019 investigation) and in Appendix B (2022 investigation). Site soil and groundwater conditions are described, and geotechnical recommendations are discussed in the following sections of this report.

Cone Penetration Tests were conducted at the site by ConeTec Investigations Ltd. The methodology and results are described in detail in the appended report (Presentation of Site Investigation Results), which is provided in Appendix D.

2.2 Borehole and CPT Investigations

2.2.1 Geotechnical Investigation – July and August 2019

Cambium completed a geotechnical investigation at the Site on July 4, 5, and 8, 2019, and August 14, 2019. A total of four boreholes, designated as BH101-19 through BH104-19, were advanced into the subsurface at predetermined locations throughout the Site. BH101-19 was terminated at a depth of 34.0 m below ground surface (mbgs) and BH102-19 at a depth of 31.0 mbgs. BH103-19 and BH104-19 were both terminated at depth of 6.7 mbgs.

Two monitoring wells were installed at the time of investigation, one adjacent to BH101-19 and the other adjacent to BH102-19 to a depth of approximately 9.0 mbgs.

2.2.2 Additional Geotechnical Investigation – February 2022

Cambium completed a supplementary geotechnical investigation at the Site from February 7-10, 2022. A total of two boreholes, designated as BH101-22 and BH102-22, were advanced into the subsurface at predetermined locations throughout the Site. Both boreholes were terminated at a depth of 31.0 mbgs.

In addition to the boreholes, three cone penetration tests (CPT) were advanced throughout the Site. The CPT locations, designated as CPT101-22 through CPT103-22, were situated in sufficiently close proximity to the borehole locations to allow for a comparison of the test results with known soil stratigraphy. The CPT advancement was terminated at depths ranging from 25.2 mbgs to 34.1 mbgs.

The coordinates for all borehole and CPT locations from both investigations were obtained from a handheld GPS unit, while the elevation of all test locations was surveyed utilizing two benchmarks with known elevations (benchmark on the north side of the existing building is the base of the bollard located along the northern edge of the building, benchmark on the south side of the existing building is the centre of the catch basin in the loading dock area).

The geodetic elevation of the bollard base is 229.08 metres above sea level (mASL), and the elevation of the catch basin centre is 228.64 mASL, according to the survey provided by the Client.

A Site Plan including the borehole and CPT locations is appended as Figure 1 of this report. General borehole and CPT information including geodetic ground elevations and termination depths is summarized in Table 1.

Table 1 Borehole and CPT tests

Test ID	Ground Elevation (mASL)	Termination Depth (mbgs)	Termination Elevation (mASL)
Geotechnical Investigation – July and August 2019			
BH101-19	228.98	34.0	194.98
BH102-19	229.87	31.0	198.87
BH103-19	229.34	6.7	222.64
BH104-19	229.80	6.7	223.10
Additional Geotechnical Investigation – February 2022			
BH101-22	229.86	31.0	198.86
BH102-22	229.26	31.0	198.26
CPT101-22 [1]	228.81	27.6	201.21
CPT102-22 [1]	229.83	25.2	204.63
CPT103-22 [1]	229.42	34.1	195.32

BH10x-xx … Borehole with SPT Sampling *CPT10x-xx … Cone Penetration Test*

[1] *Note: CPT locations are denoted in ConeTec investigation report (Appendix D) as follows:*
CPT101-22 → SCPT22-01, CPT102-22 → CPT22-02, CPT103-22 → SCPT22-03

2.3 Physical Laboratory Testing

Physical laboratory testing, including particle size distribution analyses (LS-702,705) and Atterberg limits analyses, was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters.

Moisture content testing was completed on all soil samples collected from the site during investigations conducted by Cambium. Testing results are presented in Appendix B and are discussed in Section 3.0.

3.0 SUBSURFACE CONDITIONS

The detailed soil profiles encountered in the boreholes are indicated on the attached borehole logs in Appendix A and Appendix B. It should be noted that the conditions indicated on the borehole logs are for specific locations only and can vary between and beyond the borehole locations. The soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change. In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (drilling speed, shaking/grinding of the augers, etc.).

Based on the results of the borehole investigation, subsurface conditions at the Site generally consist of fill material overlying sand formations. At greater depths, clay seams interspersed within the sand formation were encountered. Bedrock was not encountered during the investigations conducted by Cambium at the site.

3.1 Pavement Structure

All of the boreholes were advanced through existing asphaltic concrete pavement, which was noted to be approximately 50 millimetres in thickness. Below the surficial asphalt layer, about 300 to 500 mm of gravelly sand to sand and gravel base/subbase was encountered in some boreholes. It should be noted that the thickness of the asphalt concrete and granular base/subbase may vary across the site.

3.2 Fill

A layer of sand fill containing trace to some gravel, trace silt, trace clay, and occasional asphalt fragments was encountered below the pavement structure at all boreholes. In BH102-19, the encountered fill was determined to be a silty sand with trace amounts of gravel and clay based on laboratory testing.

The fill layer, based on observations recorded during drilling, was inferred to depths of approximately 1.0 to 2.3 mbgs. It is noted that given the heterogeneous nature of the existing development, the depth and composition of fill should be expected to vary significantly throughout the site. In areas adjacent to existing underground levels, backfill depths may be expected to correspond to presumable excavation slopes down to foundation subgrade elevations, which would have been required to construct the basement levels.

Standard penetration tests (SPT) conducted within the inferred fill materials gave N values ranging from 1 to 9 blows per 0.3 metres of penetration, which are reflective of a very loose to loose relative density. Locally, higher blow counts were recorded (N values of 32 and 33 in BH101-22 and BH102-22), which indicate a dense relative density immediately below the asphalt surface.

Table 2 summarizes the inferred fill depths at the borehole locations, including the observations of surficial fill recorded during CPT drillouts, during which the prospective CPT locations were predrilled using hollow stem augers to a depth of approximately 1.5 mbgs.

Table 2 Summary of Inferred Fill Depths

Test ID	Fill Depth (mbgs)	Textural Description
BH101-19	0.05 – 0.40	Brown crushed sand, with gravel, trace silt
	0.40 – 1.20	Brown sand, trace gravel, trace silt
BH102-19	0.05 – 0.30	Brown crushed gravelly sand, trace silt
	0.30 – 2.10	Brown silty sand, trace gravel, trace clay
BH103-19	0.05 – 0.60	Dark brown sand, some gravel, trace silt
	0.60 – 2.30	Brown sand, trace silt, trace gravel
BH104-19	0.05 – 2.30	Brown sand, trace silt, trace gravel
BH101-22	0.05 – 0.90	Brown sand, some gravel
BH102-22	0.05 – 1.50	Dark brown sand, trace gravel
CPT101-22	0.15 – 0.30	Black sand, some gravel, trace silt
	0.30 – 1.50	Brown sand, trace gravel, trace silt
CPT102-22	0.20 – 0.30	Black sand, trace gravel, trace silt
	0.30 – 1.50	Brown sand, trace silt
CPT103-22	0.18 – 0.90	Grey sand, some gravel, trace silt
	0.90 – 1.50	Brown sand, some gravel

A particle size distribution analysis was completed on a sample collected from the inferred fill in BH102-19. The testing results are provided in Appendix C and are summarized in Table 3 based on the USCS.

Table 3 Particle Size Distribution Analysis – Inferred Fill

Borehole/ Sample	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH102-19-SS3	1.5 – 2.1	Silty Sand trace Clay trace Gravel	1	74	22	3	12.9

3.3 Sand Formations

Beneath the fill, the native soils consisted primarily of sand-dominant formations with varying amounts of silt, gravel and clay extending to the borehole termination depths. The silt content of the soil generally increased with depth, with a sand and silt formation encountered between 6.0 mbgs and 12.0 mbgs in BH102-22 and a silty sand formation between approximately 24.5 mbgs and 27.5 mbgs in BH102-19.

Natural moisture contents determined on samples collected from the sand formations by laboratory analysis ranged from 7.2% to 24.0%.

Particle size distribution analyses were completed on a total of five samples collected from the sand formations. The testing results are provided in Appendix C and are summarized in Table 4 based on the USCS.

Table 4 Particle Size Distribution Analysis – Sand Formations

Borehole/ Sample	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH101-19-SS5	3.0 – 3.7	Sand some Silt trace Clay	0	86	12	2	21.8
BH102-19-SS16	24.4 – 24.8	Silty Sand trace Clay	0	63	34	3	19.7
BH104-19-SS4	2.3 – 2.7	Sand some Silt trace Clay	0	81	16	3	8.2
BH101-22-SS11	12.2 – 12.7	Sand trace Silt trace Clay trace Gravel	2	83	8	7	14.7
BH102-22-SS7	6.1 – 6.6	Sand and Silt trace Clay	0	54	42	4	21.9

The SPT N values recorded within the sand formations generally indicated compact to very dense relative densities below depths of approximately 2.3 mbgs to 3.0 mbgs, with blow counts ranging from 12 to over 50 for 150 mm of penetration. Lower blow counts were recorded at shallower depths, indicating very loose to loose relative densities.

It is noted that although the relative density of the soil generally increased with depth, decreases in relative density from dense/very dense to compact were also recorded at greater depths.

In BH101-19, an SPT N value of 5 was recorded at a depth of 30.5 mbgs, indicating a loose relative density, significantly lower than the surrounding compact to very dense sands. It is unknown if this particularly low SPT N value was due to borehole caving, driller error, or a loose/soft stratum of soil such as the seams described in further detail in Section 3.5.

3.4 Gravel Seams

A thin gravel seam consisting of fine gravel was observed during drilling and sampling in BH101-19, BH101-22 and BH102-22 at a depth of approximately 7.6 mbgs, extending to roughly 7.7 mbgs. The gravel seam was also encountered and inferred at all CPT locations, at approximately the same depth.

3.5 Clay Seams

A seam of silty clay was encountered in BH102-22 during drilling and sampling at a depth of approximately 18.3 mbgs, or at an elevation of approximately 211.0 mASL. An SPT N value of 20 was recorded, indicating a stiff consistency.

A confirming observation was made in corresponding CPT103-22, with a drop in base resistance and increase in measured pore water pressures recorded at a depth of approximately 18.0 mbgs, or at an elevation of approximately 211.5 mASL.

Similar observations were made at elevations of 205.0 mASL and 203.5 mASL in CPT103-22, indicating the likely presence of additional clay seams at lower elevations, within depths not covered by SPT sampling intervals. Further, corrected SPT N values using correlations from CPT results at these depths were calculated at approximately 10, indicating possible soft to stiff soil consistencies.

Related observations were also made in CPT102-22 (situated near BH101-22), with a soft clay seam inferred at an elevation of approximately 211.8 mASL.

Based on the observations during SPT sampling and on correlations with CPT results, clay seams can be expected to be interspersed within the sand formations beginning below an elevation of approximately 212.0 mASL (below a depth of approximately 18 mbgs), with the clays exhibiting soft to stiff soil consistency.

An Atterberg Limits analysis was performed on the clay sample collected from BH102-22, with the results summarized in Table 5.

Table 5 Atterberg Limits Analysis – Clay Seams

Borehole/ Sample	Depth (mbgs)	Soil Textural Description	LL	PL	PI	Description	% Moisture Content
BH102-22-SS14	18.3 – 18.7	Silty Clay	24.2	16.7	7.5	CL - ML	19.8

3.6 Bedrock

Bedrock was not encountered in any of the boreholes or CPT tests advanced by Cambium at the site.

3.7 Groundwater

Saturated soils were observed during drilling beginning at depths of 3.0 mbgs and 4.7 mbgs. Boreholes BH101-19, BH102-19, BH101-22 and BH102-22 were stabilized with drilling mud, therefore no significant sloughing (caving) was noted upon terminating the boreholes.

During the site visits and investigation in February 2022, it was noted that BH101-19 and BH102-19, installed during the investigations in July and August 2019, were damaged and no longer functional. Measurements were conducted in all other accessible monitoring wells on site.

It is noted that Cambium recorded monthly groundwater measurements from the installed monitoring wells over a twelve-month period (July 2019 through June 2020). During this monitoring period, groundwater levels at the site were measured between 2.85 mbgs and 4.93 mbgs, corresponding to elevations between 224.51 mASL to 226.29 mASL. A detailed presentation of all recorded groundwater elevations (provided to the Client under a separate cover in July 2020) is provided in Appendix F.

Measurements taken at the functional and/or accessible existing monitoring wells on site during the investigation works in February 2022 are presented, along with observations in the additional boreholes during drilling, in Table 6.

It should be noted that the groundwater levels at the site may fluctuate seasonally and in response to climatic events.

Table 6 Groundwater Levels Observed in Boreholes – February 2022

Borehole ID	Top of Well Elevation (mASL)	Date of Observation	Depth of Groundwater below Top of Well (m)	Elevation of Groundwater (mASL)	Notes
BH101-19	228.87	Feb. 9, 2022	Not functional	Not functional	Monitoring Well
BH102-19	229.80	Feb. 9, 2022	Not functional	Not functional	Monitoring Well
MW101	228.45	Feb. 9, 2022	- 1)	- 1)	Monitoring Well (Installed by Others)
MW102	228.94	Feb. 9, 2022	3.10	225.84	Monitoring Well (Installed by Others)
MW103	229.97	Feb. 9, 2022	dry	dry	Monitoring Well (Installed by Others)
MW104	229.73	Feb. 9, 2022	3.79	225.94	Monitoring Well (Installed by Others)
BH101-22	229.86 2)	Feb. 7, 2022	4.7 2)	225.16	Saturated soils observed during drilling
BH102-22	229.26 2)	Feb. 8, 2022	3.0 2)	226.26	Saturated soils observed during drilling

1) Access unavailable due to snow cover above monitoring well

2) Ground elevation / depth of groundwater below ground surface for borehole not outfitted with well

During the cone penetration tests (CPT) conducted at the site in February 2022, pore water dissipation tests were completed at selected elevations during cone advancement. The assumed phreatic surfaces for each CPT location are summarized in Table 7.

Detailed summaries, analysis and results of the pore water dissipation tests are presented in the appended report in Appendix D.

Table 7 Assumed Phreatic Surface at CPT Locations

CPT ID	Ground Elevation (mASL)	Date of Observation	Assumed Phreatic Surface (mbgs)	Elevation of Phreatic Surface (mASL)	Dissipation Test Depths (mbgs)
CPT101-22	228.81	February 10, 2022	3.5	225.31	6.00 / 20.00 / 27.45
CPT102-22	229.83	February 10, 2022	4.1	225.73	5.25 / 15.50 / 18.075 / 20.50 / 25.20
CPT103-22	229.42	February 10, 2022	3.0	226.42	10.85 / 12.85 / 18.50 / 24.50 / 27.825 / 34.125

4.0 GEOTECHNICAL CONSIDERATIONS

The following recommendations are based on the borehole information and are intended to assist designers. Recommendations should not be construed as providing instructions to contractors, who should form their own opinions about site conditions. It is possible that subsurface conditions beyond the borehole locations may vary from those observed. If significant variations are found before or during construction, Cambium should be contacted so that we can reassess our findings, if necessary.

4.1 General Site Preparation

Any existing fill material and organic materials encountered should be excavated and removed from beneath the proposed building footprints; additionally, this material should be excavated and removed to a minimum distance of 3 m around the building footprints. The fill material may potentially be left in place beneath any proposed parking, driving or landscaped areas. The fill material includes but is not limited to the fill materials identified in this report. Any topsoil and materials with significant quantities of organics and deleterious materials (i.e., construction debris, asphalt etc.) are not appropriate for use as fill below parking and driving areas.

The exposed subgrade should be proof-rolled and inspected by a qualified geotechnical engineer prior to placement of any granular fill. Any loose/soft soils identified at the time of proof-rolling that are unable to uniformly be compacted should be sub-excavated and removed. The excavations created through the removal of these materials should be backfilled with approved engineered fill consistent with the recommendations provided below.

The near surface sand and silty sand soils can be very unstable if they are wet or saturated. Such conditions are common in the spring and late fall. Under these conditions, temporary use of granular fill, and possible reinforcing geotextiles, may be required to prevent severe rutting on construction access routes.

4.2 Excavations

The bulk excavation for construction of the foundation of the proposed structure is expected to extend to a maximum depth of approximately 3.0 mbgs, with accordingly deeper excavations for elevator shafts likely required within the excavation footprint. The excavations will extend through surficial shallow fill materials, into the predominately sandy/silty formations described above. Based on the observed water levels during drilling and subsequent site visits to measure water elevations in the monitoring wells, the sandy/silty deposits are expected to be water-bearing below a depth of approximately 3.0 mbgs or below an elevation of approximately 226.5 mASL (these values represent the maximum water levels observed during the excavation and may be expected to vary based on external influences such as precipitation and snowmelt).

Temporary excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). Above the groundwater table and within continually dewatered depths, the soils at this site would generally be classified as Type 3 soils in accordance with OHSA, with permissible unsupported side slopes no steeper than 1H:1V extending to the bottom of the excavation.

Excavation side slopes should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavations or if wet conditions are encountered, the side slopes should be flattened as required to maintain safe working conditions.

If required due to excavation depths and/or spatial restrictions (i.e., proximity of outer excavation perimeter to property lines), the excavation sidewalls should be fully supported (shored). The excavation support system must be designed to resist the lateral earth pressures of the soils, hydrostatic pressures and any surcharges while limiting ground movements to tolerable levels. It is a common practice for a specialist contractor to design and install the excavation support system. Considering the soils encountered at the site, possible excavation support systems include but are not limited to sheet pile walls, secant/tangent pile walls, and depending on the excavation depth in relation to the groundwater level also soldier pile walls.

Since the proposed design founding elevations and the need for a shoring have not been determined at the time this report was prepared, the geotechnical parameters to be used in the shoring design, if required, should be determined in consultation with Cambium during the detailed design stage once the excavation schemes are known.

4.3 Dewatering

As discussed in the previous section, groundwater was observed in monitoring well at variable depths below approximately 3.0 mbgs. It should be noted that the groundwater table is influenced by seasonal fluctuations and major precipitation events.

Based on the groundwater conditions measured in the monitoring wells and the anticipated founding elevations (assumed for the purposes of this report to be at a maximum depth of 3.0 mbgs), the foundation excavations may be approximately at the groundwater level or in times of the year with seasonally high groundwater levels possibly slightly below the local groundwater table within the sandy/silty formations. It is noted that any sub-excavations related to special foundation works, ground improvement methods, etc., may extend below the groundwater table.

The sandy/silty deposits encountered in the boreholes are highly susceptible to disturbance by groundwater seepage. In this regard, depending on the time of year the construction takes place, localized pro-active dewatering of the groundwater levels to at least 1 m below the maximum excavation depth elevations using a well-point system may be required to maintain the integrity of the excavation. If possible, it is recommended that construction excavations during the wet/spring seasons be avoided to reduce the need for pro-active dewatering.

Overall, it is anticipated that some groundwater seepage is likely to occur where excavations are made below the groundwater level. It should be possible to handle the groundwater inflow from this deposit by pumping from well-filtered sumps in the floor of the excavation, using suitably sized pumps. A dewatering system such as well-points may also be used to depress the groundwater level below the excavation bases.

If a well-point dewatering system is used, registration on the Environmental Activity and Sector Registry (EASR) or a Permit to Take Water (PTTW) may be required from the Ministry of the Environment Conservation and Parks (MOECP) as pumping could exceed 50,000 L/day or 400,000 L/day respectively.

For further information regarding dewatering, it is recommended to consult the hydrogeological report provided by Cambium under a separate cover.

4.4 Foundation Design

It is our understanding that the proposed development at the subject property would consist of a primarily residential building with up to 17 storeys, including four levels of above-ground parking. Based on current floor plans, the majority of the structure’s footprint will contain 12 above-ground storeys, with the 17-storey section situated within the western portion of the structure.

The existing sand and silty sand deposits encountered at shallow depths (less than 3.0 mbgs) are generally to be considered unsuitable for supporting conventional strip/spread foundations, given the high loads expected for the proposed structure. From a geotechnical perspective, two feasible and practical foundation systems for the proposed multi storey tower building will be discussed in the following sections of this report: the construction of a raft/mat foundation (likely in conjunction with ground improvement methods), and alternatively, drilled piles/caissons.

4.4.1 Raft / Mat Foundations

A viable option for the foundation of the proposed structure is the use of a reinforced concrete raft/mat foundation system, which would be positioned at or above 3.0 mbgs, given that no basements are being constructed. The bearing stratum below 3.0 mbgs will likely consist of compact to very dense (silty) sands. The founding level should ideally be uniform across the footprint to limit differential settlements. To make sure the building loads are distributed uniformly over the entire building footprint, a raft foundation would need to be designed with sufficient rigidity.

The design of raft foundations is generally governed by settlement considerations rather than bearing capacity since the design bearing pressure is generally less than the allowable bearing capacity. The thickness and reinforcement for the raft foundation should be designed by the project structural engineer.

At the proposed foundation level, the raft foundation may be designed using a maximum bearing capacity of 275 kPa at SLS (Serviceability Limit State) and 350 kPa at ULS (Ultimate Limit State) on undisturbed native soils. If the bearing stresses under the raft were to reach the SLS bearing resistance provided here (i.e., if the full structural load were to equal the full available SLS bearing resistance), the calculated total settlement of the raft foundation can be expected to be in the order of 40 mm to 50 mm. The post-construction total and differential settlements will likely occur in a short period of time after applying the building loads, given the predominately granular soil composition.

Given the complex nature of the proposed structure, with different areas of the footprint likely to experience significantly different loading, it is recommended to undertake a settlement analysis using actual loading and using appropriate deformation parameters for the encountered soil stratigraphy. The results can be used to calibrate the concrete design of the raft (reinforcement requirements), using an iterative process to determine actual moduli of subgrade reaction as necessary. The settlement analysis can also be used to design the required dimensions/depths of ground improvement methods, as necessary.

As a preliminary recommendation, a differential settlement of up to 80% of the total settlement can be expected. However, this differential settlement will also depend greatly on the stiffness of the raft; even for ideally uniform ground conditions, the settlement of the edge of the raft would typically be less than that of the centre. It should be noted that the SLS bearing resistance specified above for design of a raft foundation should not be interpreted as the same for the design of conventional spread/strip foundations, which is given in a separate section below.

If the actual foundation loading exceeds maximum bearing capacities for the native soil stratigraphy or if the estimated settlement values are not acceptable for the proposed structure, the raft foundation may be combined with ground improvement methods to increase the bearing capacity of the soils below the foundation subgrade elevation.

Possible ground improvement methods include, but are not limited to, rammed aggregate piers (RAP), Controlled Modulus Columns (CMC), and Dynamic Compaction. The improved bearing capacity values will depend on the type of application used and related parameters. Allowances should be made to overlay the ground improvement elements with an engineered fill pad, extending from the work surface for ground improvement up to the desired subgrade elevation of the raft foundation.

Ground improvement systems are generally proprietary in nature, as such it is recommended to consult with specialty design-build contractors regarding the system best suited to this site, feasibility, design bearing capacity improvements and cost estimates.

If ground improvement alone is determined insufficient to provide for the required bearing capacities (i.e., for portions of the proposed building with 17 storeys), consideration could be given to a piled raft foundation (combination of a raft foundation with drilled concrete piles/caissons).

The structural analysis of the raft slab should involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until these two are consistent with each other. For initial analyses, the modulus of subgrade reaction may be assumed to be in the range of 20 MPa/m to 100 MPa/m. This range reflects both uncertainties of the size of the footing as well as variability in the properties of the subgrade soils and should be refined as actual loads and foundation dimensions become known.

Hydrostatic pressure acting on the foundation elements, if applicable, must be considered in the structural design of the foundations. For these purposes, it is recommended to assume a maximum water level of 227.0 mASL (corresponds to the maximum water level observed during groundwater monitoring events from the previously installed monitoring well of approximately 226.50 mASL, with an additional allowance of 500 mm). For watertight raft foundations, this elevation may be used to determine uplift loading.

4.4.2 Strip/Spread Foundations

Shallow strip foundations being used for different elements throughout the structure (i.e., walls or columns) and bearing on compact native sands below the frost penetration depth may be designed using a maximum bearing capacity of 90 kPa at SLS and 140 kPa at ULS. For spread foundations bearing on compact native soils below the frost penetration depth, these values can be increased to 110 kPa at SLS and 160 kPa. It is noted that the soil bearing capacity at deeper depths may be subject to change and that the depth of fill varies across the site. Should shallow footings be considered, Cambium would recommend reviewing the foundation drawings so we can provide an updated recommendation based on the known information.

These bearing capacities may be increased further where foundations are constructed on improved ground or on an engineered fill pad extending to competent soils. Engineered fill pads must be constructed according to the requirements for backfill and compaction given below, and are required to extend a minimum of 1 m from the outer edges of the foundation prior to sloping down the approved subgrade at a 1H:1V slope. If engineered fill pads are considered, Cambium should be notified to provide further recommendations.

Improved bearing capacity values depend on the engineered fill pad thickness and/or ground improvement, and can be given by Cambium once these parameters are known.

4.4.3 Drilled Piles/Caissons

Cast-in-place concrete piles may be installed as conventional drilled caissons or as continuous flight auger (CFA) piles. The table below provides parameters for calculating the unfactored geotechnical shaft skin friction at ULS. The parameters are given in accordance with the Canadian Foundation Engineering Manual’s (CFEM) provided correlations for CPT test results (Formula 18.15b and Table 18.4, with assumptions made for compact granular soils) and also using discretion based on experience with the encountered soil types.

Given the soft to stiff cohesive soil layers encountered at greater depths (below approximately 212.0 mASL), it is recommended to neglect any pile base/tip resistance in the design of drilled piles/caissons.

Table 8 Unfactored Skin Friction for Cast-in-Place Concrete Piles

Soil Strata	Elevation (mASL)	Maximum Skin Friction (kPa)
Loose to compact sand	> 226.0 *(approx. 0.0 – 3.0 mbgs)*	-
Compact to dense (silty) sand	226.0 – 221.2 *(approx. 3.0 – 7.8 mbgs)*	60
Dense to very dense (silty) sand	< 221.2 *(approx. 7.8 mbgs and below)*	80

The associated resistance factor = 0.40 should be applied according to CFEM criteria for deep foundations without load test data. Further consideration should be given to reassessing/confirming the capacity by Osterberg Cell (O-Cell) tests and the subject site to determine design values.

Based on the findings of the investigation, the soils at this site are cohesionless with poor gradation, permeable and sufficiently wet such that augered holes made into these soils will be unstable. It will be necessary to control the stability of the bases of any augered holes below groundwater to protect them against basal disturbance caused by the ingress of groundwater and to prevent loss of ground. Contractors must be advised to use special procedures, including the use of slurry (polymer drilling fluid) to maintain the stability of the shafts, end cleaning using the air-lifting or the one-eyed bucket technique and tremie pour. All augered holes shall have testing for shaft verticality, volume and diameter measurements in-situ prior to reinforcement or concrete placement.

To avoid additional settlement of caissons caused by group effect, the individual caissons must have a spacing of 3 times the diameter or more. Caisson foundations at different elevations must be provided with a 10H:7V vertical separation.

Piling operations should be inspected on a full-time basis by geotechnical personnel.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction (lateral spring parameters) and ultimate lateral resistance may be calculated from the following expression if the soil is primarily cohesionless at the site:

$$k_{s,h} = n_h \cdot (z/d) \text{ in [kPa/m]}$$

$$p_{ult} = 3 \cdot \gamma' \cdot z \, K_p \text{ [kPa]}$$

Where $k_{s,h}$ = Coefficient of horizontal subgrade reaction in [kPa/m]

n_h = Constant of horizontal subgrade reaction in [kPa/m]

d = Pile width in [m]

z = Depth in [m]

γ' = effective unit weight in [kN/m^3]

p_{ult} = Horizontal Passive Pressure in [kPa]

Table 9 below summarizes the values of n_h for the soil encountered in the borehole investigation.

Table 9 Design Parameters for Lateral Resistance

Soil Strata	Elevation (mASL)	Bulk Unit Weight (kN/m^3)	Passive Earth Pressure Coefficient K_p (-)	n_h (kPa/m)
Loose to compact sand	> 226.0 *(approx. 0.0 – 3.0 mbgs)*	19.0	2.8	-
Compact to dense (silty) sand	226.0 – 221.2 *(approx. 3.0 – 7.8 mbgs)*	19.5	3.0	3,000
Dense to very dense (silty) sand	< 221.2 *(approx. 7.8 mbgs and below)*	20.0	3.2	10,000

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 8d. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading using a reduction factor, R, as follows in Table 10:

Table 10 Reduction Factor Due to Pile Spacing

Pile Spacing d = Pile Diameter or Width	Subgrade Reaction Reduction Factor, R
8d	1.00
4d	0.75
2d	0.40
d	0.25

4.5 Seismic Site Classification

The structures should be designed to withstand forces caused by seismic activity in accordance with the Ontario Building Code (OBC).

To determine the site classification, shear wave velocity measurements were conducted during CPT advancement. A v_{s30} value was determined using results from CPT103-22, which was advanced to a depth of 34.1 mbgs. The determined value of v_{s30} = 373 m/s allows for a categorization of the site into Site Class C as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. These earthquake/seismic design parameters should be reviewed in detail by the structural engineer and incorporated into the design as required.

Peak ground acceleration and spectral acceleration (period of 0.2 seconds) for the site are calculated to be 0.064g and 0.108g respectively using the 2015 National Building Code Seismic Hazard Calculation. A detailed report of the calculation and its results can be found in Appendix E.

4.6 Frost Penetration

Based on the Ontario Provincial Standard Drawing (OPSD) 3090.101, the typical frost penetration depth for the proposed structure is expected to be approximately 1.5 mbgs. Footings for the proposed structure should be situated at or below this depth for frost penetration or should be protected. If construction is carried out during the winter months, all footing locations where excavated soils become exposed to potential frost penetration should be poured within the same day to prevent potential future settlements. If the footings cannot be poured within the same day the soils are excavated, the surface should be covered with thermal insulation equivalent to 1.5 m of soil cover to prevent potential freezing of the frost susceptible soils at the Site.

It is assumed that any pavement structure thickness will be less than 1.5 m; therefore, grading and drainage are important for good pavement performance and life expectancy. Any utilities should be located below this depth or be appropriately insulated.

4.7 Basement Floor Slabs & Subdrains

For both piled and raft foundation options, it is considered that conventional construction may be used for floor slabs for the proposed structure. Any soft, loose, wet, and/or disturbed material should be removed and replaced with quality granular material compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD).

The guidelines below are based on the assumptions that a “drained” foundation system will be provided. These guidelines should be revisited if it is decided that the construction of the basement would be water tight.

To prevent hydrostatic pressure building up beneath the floor and potential groundwater infiltration, it is suggested that the granular base for the floor be drained. Provision should be made for at least 300 mm of 600 mm clear crushed stone to underlie the floor. A vapour barrier beneath the slab should be provided. Perimeter and under floor drainage systems would be also required for the proposed building. The perforated pipes should discharge to a positive outlet such as a storm water sewer or a sump from which the water is pumped. Basement floor slabs and foundation walls should also be waterproofed.

4.8 Backfill and Compaction

Excavated non-organic fill and native sand soils from the site may be appropriate for use as fill below grading and parking areas, provided that the actual or adjusted moisture content at the time of construction is within a range that permits compaction to required densities. Some moisture content adjustments may be required depending on seasonal conditions. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality.

Engineered fill for foundations should consist of free-draining granular material meeting the specifications of OPSS 1010 Granular B or an approved equivalent and should be placed in maximum 200 mm thick lifts compacted to a minimum of 100% of SPMDD as confirmed by full time nuclear densometer testing and inspection.

Foundation wall backfill should consist of imported free-draining granular material meeting the specifications for OPSS Granular B, or an approved equivalent, compacted to 95% of SPMDD, taking care to keep heavy compaction equipment from damaging the walls.

The backfill material, if any, in the upper 300 mm below the pavement subgrade elevation should be compacted to 100% of SPMDD in all areas.

4.9 Buried Utilities

Trench excavations should generally consider Type 3 soil conditions which require side slopes no steeper than 1H:1V to the bottom of the excavation. Where very loose or very soft to soft soils are encountered during excavations, trench slopes should generally consider Type 4 soil conditions which require side slopes no steeper than 3H:1V to the bottom of the excavation.

The bedding and cover material for any buried utilities should consist of OPSS 1010 Granular A or B Type II, placed in accordance with pertinent Ontario Provincial Standard Drawings (OPSD 802.013). The bedding and cover material shall be placed in maximum 200 mm thick lifts and should be compacted to at least 98% of SPMDD. The cover material shall be a minimum of 300 mm over the top of the pipe and compacted to 98% of SPMDD, taking care not to damage the utility pipes during compaction.

If wet or saturated conditions exist within any utility excavation, consideration should be given to using 19 mm diameter crushed clear stone wrapped in a geotextile filter fabric as pipe bedding.

4.10 Preliminary Pavement Design

The performance of the pavement is dependent upon proper subgrade preparation. All topsoil and organic materials should be removed down to native material and backfilled with approved engineered fill or native material, compacted to 100% of SPMDD. The subgrade should be compacted, proof rolled, and inspected by a Geotechnical Engineer. Any areas where rutting or appreciable deflection is noted should be sub-excavated and replaced with suitable fill. The fill should be compacted to at least 100% of SPMDD.

The recommended minimum pavement structure design has been developed for two (2) traffic loading scenarios, light duty and heavy duty. The light duty pavement structure is intended for private roads, parking areas, driveways which will not see frequent heavy traffic from trucks, buses, etc. while the heavy-duty pavement structure is appropriate for private roads where heavy loads such as trucks, buses are anticipated or for designated fire routes. The recommended minimum pavement structure is provided in Table 11.

Table 11 Recommended Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty (Roads to be assumed by the City of Barrie)	Light Duty (parking lot)
Surface Course Asphalt	OPSS 310	40 mm HL3 or HL4	40 mm HL3 or HL4
Binder Course Asphalt	OPSS 310	70 mm HL8	50 mm HL8
Granular Base	100% SPMDD (ASTM-D698)	150 mm OPSS 1010 Granular A	150 mm OPSS 1010 Granular A
Granular Subbase	100% SPMDD (ASTM-D698)	450 mm OPSS 1010 Granular B	300 mm OPSS 1010 Granular B

Material and thickness substitutions must be approved by the Design Engineer.

The thickness of the subbase layer could be increased at the discretion of the Engineer, to accommodate site conditions at the time of construction, including soft or weak subgrade soil replacement.

Compaction of the subgrade should be verified by the Engineer prior to placing the granular fill. Granular layers should be placed in 200 mm maximum loose lifts and compacted to at least 100% of SPMDD. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing. Asphalt materials should be rolled and compacted as per OPSS 310.

The final asphalt surface should be sloped at a minimum of 2% to shed runoff. Abutting pavements should be saw cut to provide clean vertical joints with new pavement areas.

4.11 Infiltration testing

In order to help determine the infiltration rate of site soils a particle size distribution test (sieve and hydrometer analyses) was completed on two samples collected close to the ground surface. In order to determine the rate at which water will be absorbed into the soil (T time), the soil was classified according to the USCS and the T Time was interpolated based on the USCS gradation charts for a particle size distribution test (hydrometer analyses).

Hydraulic conductivity values were calculated using Hazen's equation. Based on these test results, a percolation time of 10 min/cm is appropriate for the native silty sand layer.

Percolation rates for two samples are provided in Table 12 and results are attached in Appendix C.

Table 12 Infiltration Test Results

Borehole	Depth (mbgs)	Soil	Percolation Time (min/cm)	Hydraulic Conductivity, K_{fs} (cm/s)
BH102-19	1.5 – 2.1	Silty Sand	10	2.5×10^{-3}
BH104-19	2.3 – 2.7	Sand Some Silt	10	2.7×10^{-3}

4.12 Design Review and Inspections

Cambium should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed.

Further Cambium should be retained to complete testing and inspections during construction operations to examine and approve subgrade conditions, placement and compaction of fill materials, granular base courses, and asphaltic concrete.

5.0 CLOSING

Please note that this report is governed by the attached qualifications and limitations. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at (705) 719-0700.

Cambium Inc.

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100230770
April 12, 2022

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Project Manager – Geotechnical

R. L. GETHIN
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April 12, 2022

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SEB/ZL/rlg/js

Qualifications and Limitations

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Appended Figures

GEOTECHNICAL INVESTIGATION

TYPHON GROUP LTD.
272 Innisfil Street,
Barrie, Ontario

LEGEND

🟥	Borehole Location (Cambium Inc, February 2022)
●	CPT Location (Cambium inc, Feruary 2022)
🟧	Borehole Location (Cambium Inc, August 2019)
🟩	Borehole Location (HLV2K Engineering Limited, August 2018)
▭	Proposed Building
🟥	Site (approximate)

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2019 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

CAMBIUM

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BOREHOLE LOCATION PLAN

Project No.:	8707-003	Date:	February 2021
		Rev.:	
Scale:	1:750	Projection:	NAD 1983 UTM Zone 17N

Created by: MAT	Checked by: RLG	Figure: 1

O:\GIS\MXDs\8700 to 8799\8707-003 Ollie Switch Developments - Updated Geo & HydroG - 272 Innisfil St. Barrie\2022-02-18 FIG 1 - Borehole Location Plan.mxd

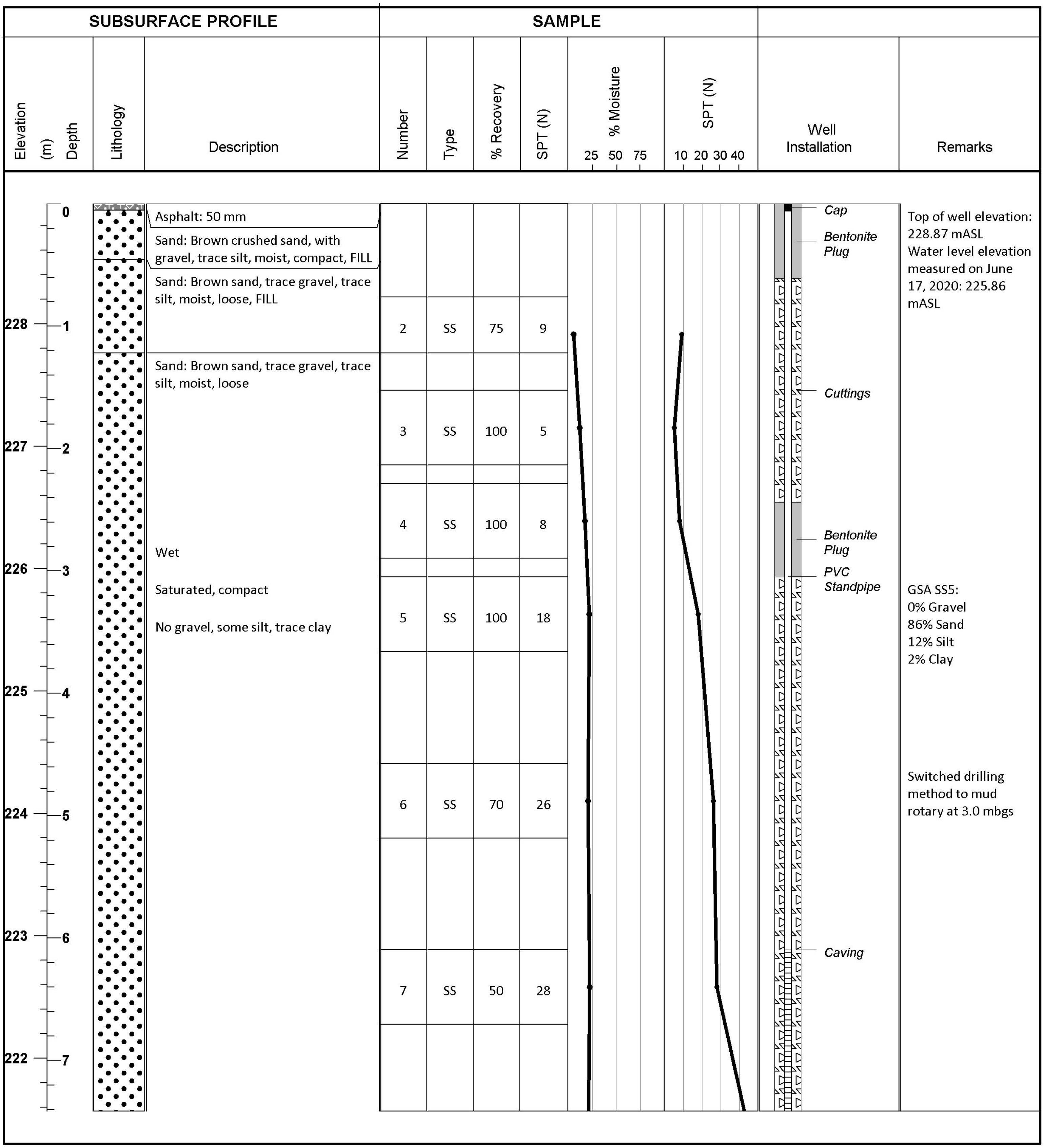
Appendix A

Borehole Logs – 2019 Investigation

Log of Borehole: BH101-19

Page 1 of 5

Client:	Innovative Planning Solution	***Project Name:***	Geotech Investigation - 272 Innisfil St	***Project No.:***	8707-001
Contractor:	Walker Drilling	***Method:***	Hollow Stem Augers + Mud Rotary	***Date Completed:***	July 4-5, 2019
Location:	272 Innisfil Street, Barrie, ON	***UTM:***	17T 604026, 4914100	***Elevation:***	228.98 mASL



Asphalt: 50 mm

Sand: Brown crushed sand, with gravel, trace silt, moist, compact, FILL

Sand: Brown sand, trace gravel, trace silt, moist, loose, FILL

Sand: Brown sand, trace gravel, trace silt, moist, loose

Wet

Saturated, compact

No gravel, some silt, trace clay

Number	Type	% Recovery	SPT (N)
2	SS	75	9
3	SS	100	5
4	SS	100	8
5	SS	100	18
6	SS	70	26
7	SS	50	28

Top of well elevation: 228.87 mASL
Water level elevation measured on June 17, 2020: 225.86 mASL

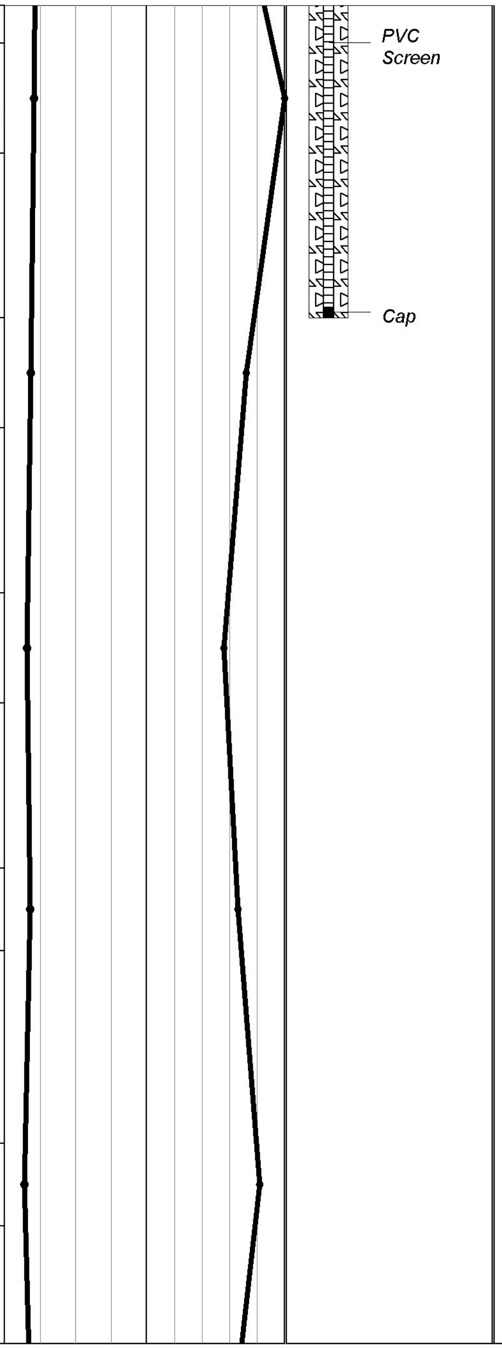
GSA SS5:
0% Gravel
86% Sand
12% Silt
2% Clay

Switched drilling method to mud rotary at 3.0 mbgs

Logged By: JB ***Input By:*** CM

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
Project No.: 8707-001
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: July 4-5, 2019
Location: 272 Innisfil Street, Barrie, ON
UTM: 17T 604026, 4914100
Elevation: 228.98 mASL

Elevation (m)	Depth	Description	Number	Type	% Recovery	SPT (N)
		Very dense, increased silt content, gravel seam				
221	8		8	SS	55	50/ 255mm
220	9	Dense				
			9	SS	80	36
219	10					
		Compact				
218	11		10	SS	70	28
217	12	Dense				
			11	SS	65	33
216	13					
215	14		12	SS	55	41



Logged By: JB **Input By:** CM

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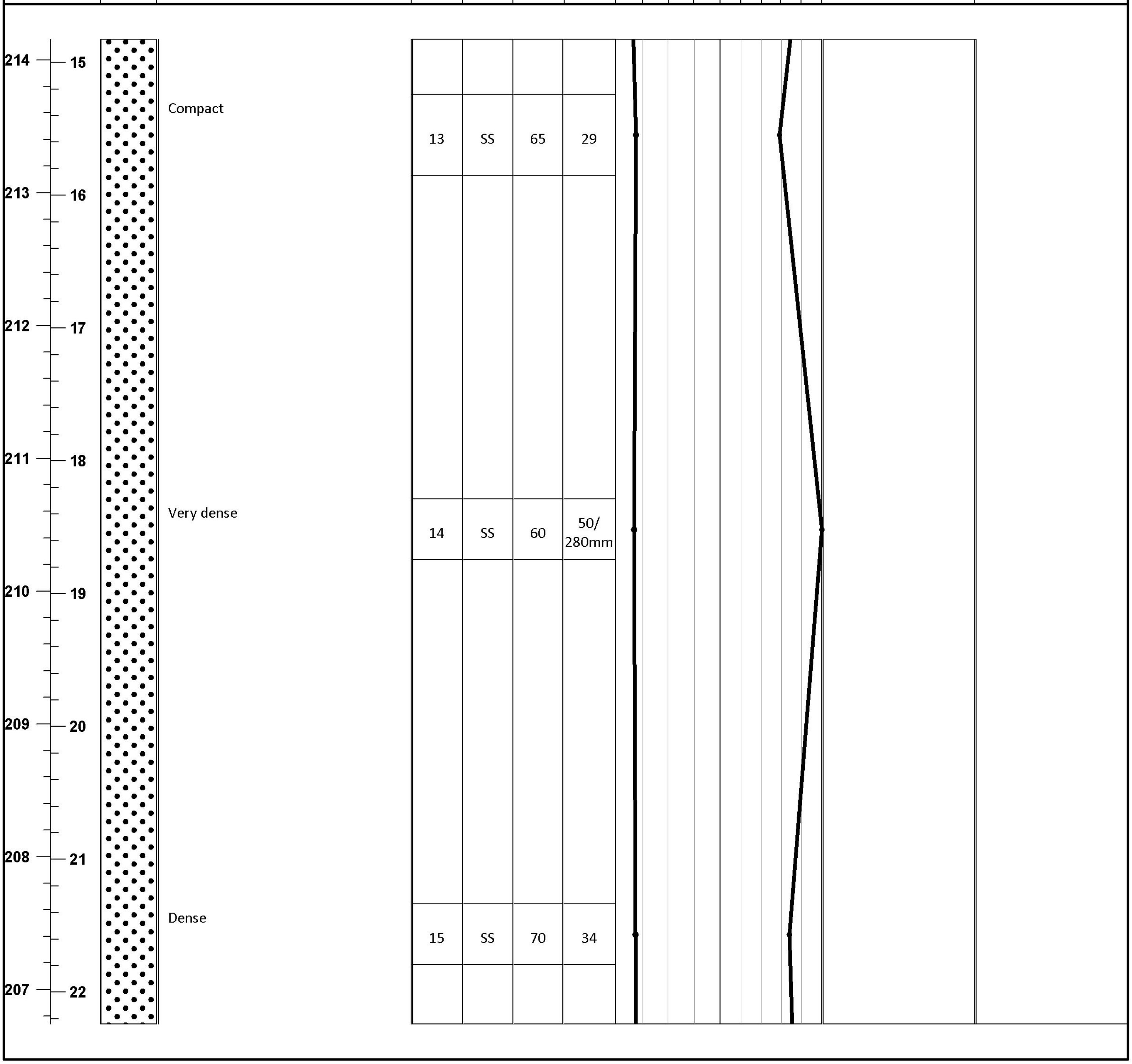
Log of Borehole: BH101-19

Page 3 of 5

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
Project No.: 8707-001
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: July 4-5, 2019
Location: 272 Innisfil Street, Barrie, ON
UTM: 17T 604026, 4914100
Elevation: 228.98 mASL

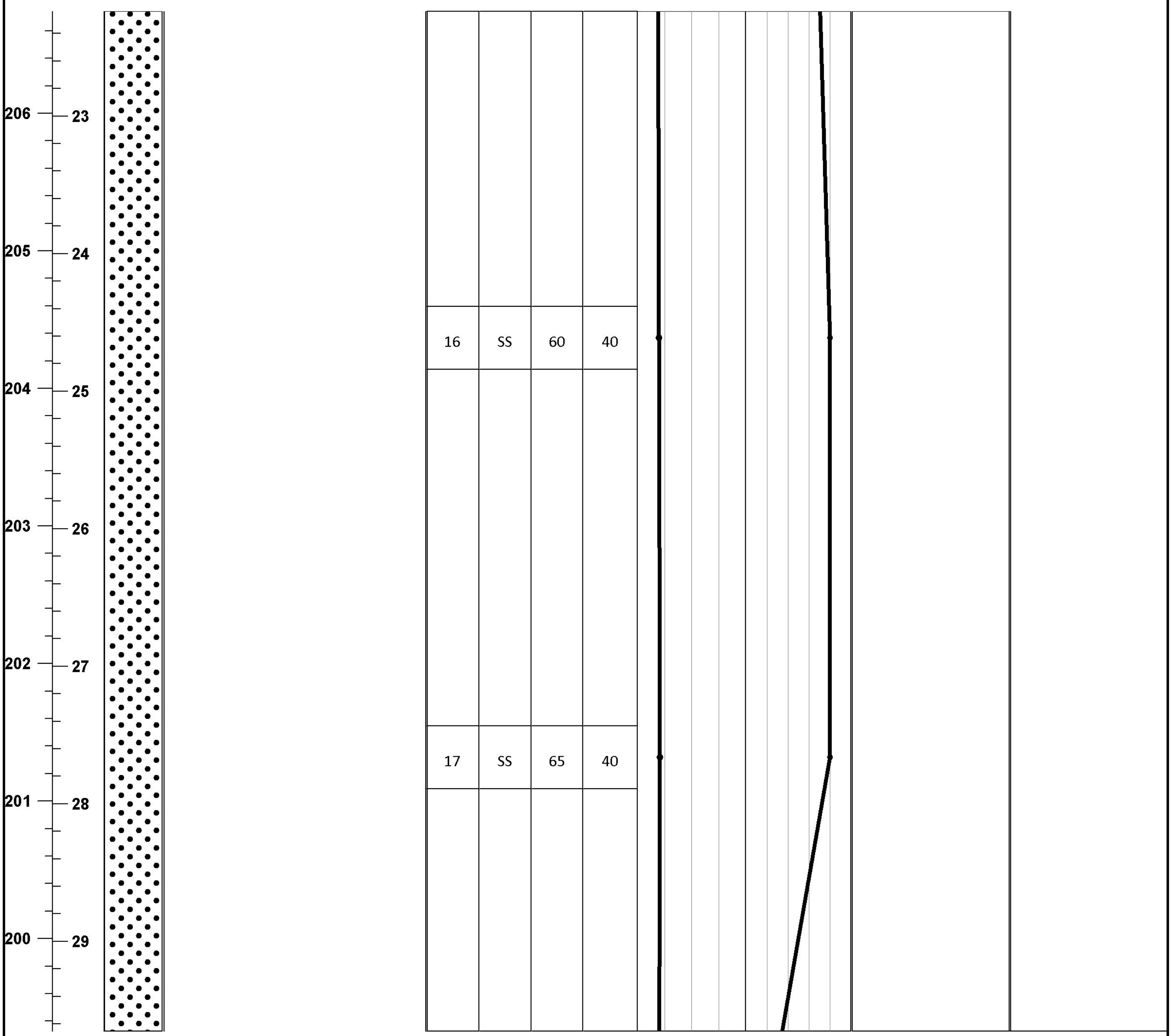


SUBSURFACE PROFILE		SAMPLE			
Description	Number	Type	% Recovery	SPT (N)	
Compact	13	SS	65	29	
Very dense	14	SS	60	50/ 280mm	
Dense	15	SS	70	34	

Logged By: JB
Input By: CM

Client:	Innovative Planning Solution	***Project Name:***	Geotech Investigation - 272 Innisfil St	***Project No.:***	8707-001
Contractor:	Walker Drilling	***Method:***	Hollow Stem Augers + Mud Rotary	***Date Completed:***	July 4-5, 2019
Location:	272 Innisfil Street, Barrie, ON	***UTM:***	17T 604026, 4914100	***Elevation:***	228.98 mASL

SUBSURFACE PROFILE				SAMPLE							
Elevation (m)	Depth	Lithology	Description	Number	Type	% Recovery	SPT (N)	% Moisture 25 50 75	SPT (N) 10 20 30 40	Well Installation	Remarks



Number	Type	% Recovery	SPT (N)
16	SS	60	40
17	SS	65	40

Logged By: JB **Input By:** CM

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
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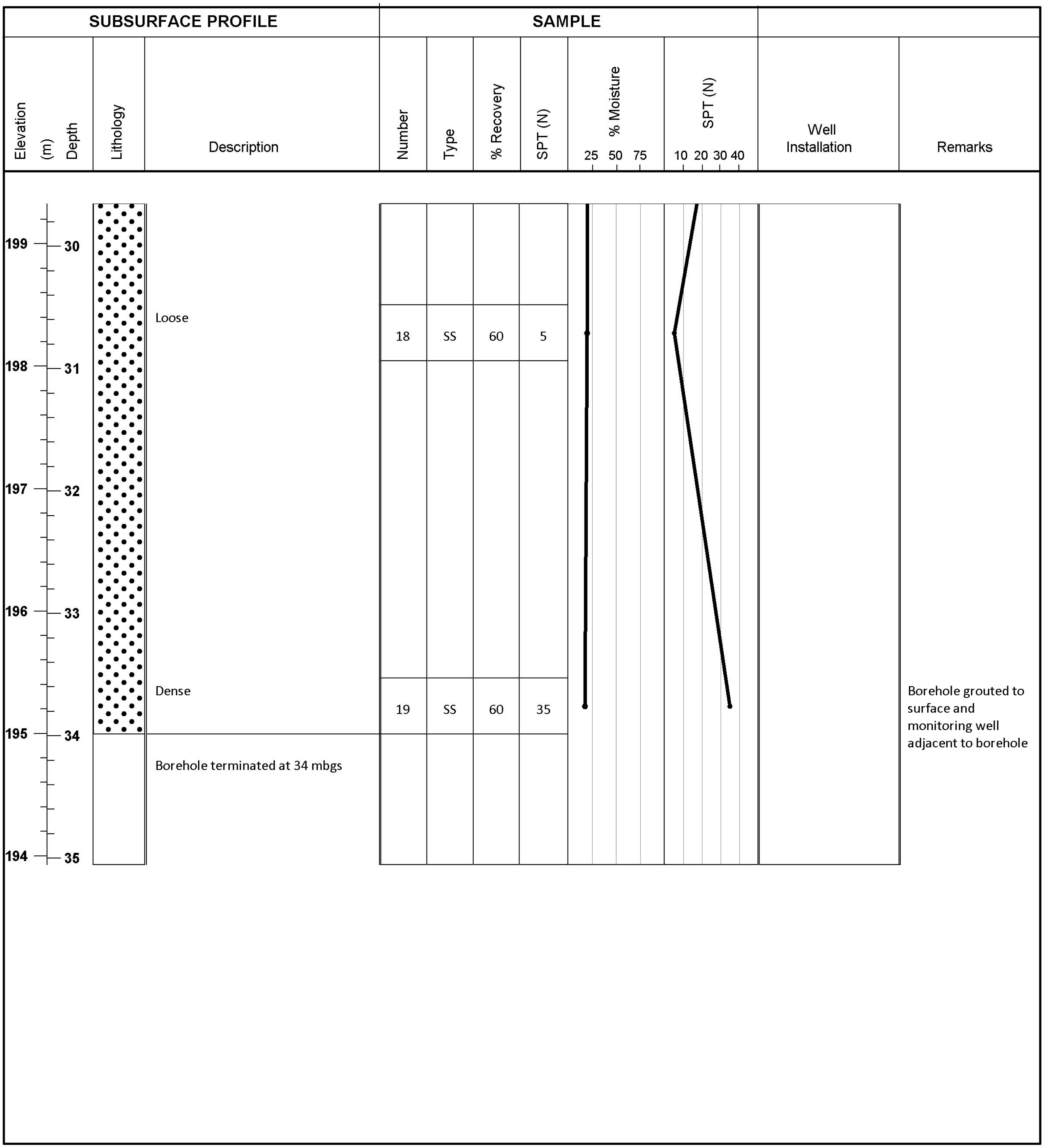
UTM: 17T 604026, 4914100
Elevation: 228.98 mASL



SUBSURFACE PROFILE			SAMPLE							
Elevation (m) Depth	Lithology	Description	Number	Type	% Recovery	SPT (N)	% Moisture	SPT (N)	Well Installation	Remarks
		Loose	18	SS	60	5				
		Dense	19	SS	60	35				Borehole grouted to surface and monitoring well adjacent to borehole
		Borehole terminated at 34 mbgs								

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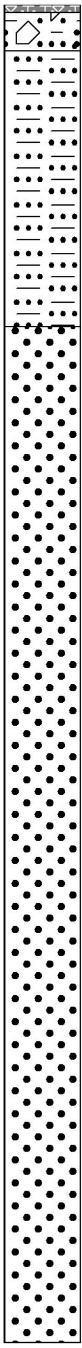
Log of Borehole: BH102-19

Page 1 of 4

Client:	Innovative Planning Solution	***Project Name:***	Geotech Investigation - 272 Innisfil St	***Project No.:***	8707-001
Contractor:	Walker Drilling	***Method:***	Hollow Stem Augers + Mud Rotary	***Date Completed:***	July 5 & 8, 2019
Location:	272 Innisfil Street, Barrie, ON	***UTM:***	17T 604017, 4914048	***Elevation:***	229.87 mASL

SUBSURFACE PROFILE

Elevation (m) Depth: 229 – 1; 228 – 2; 227 – 3; 226 – 4; 225 – 5; 224 – 6; 223 – 7; 222 – 8; 221

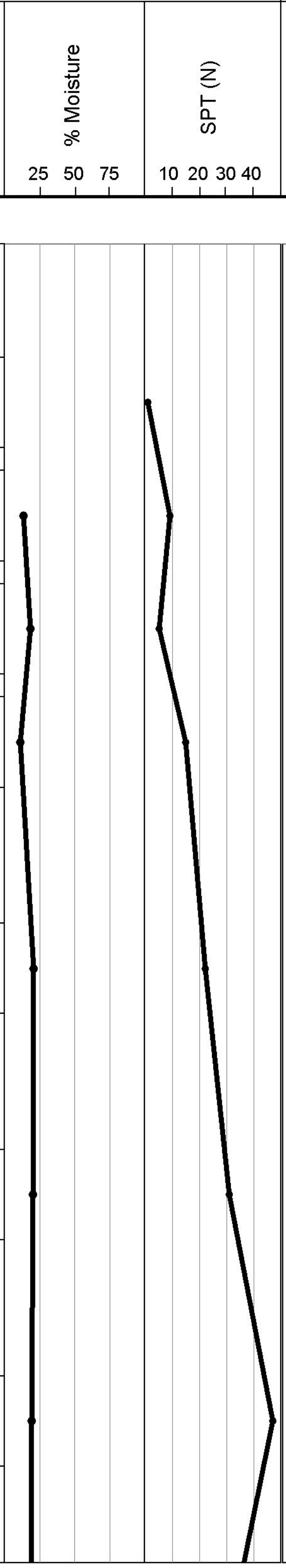


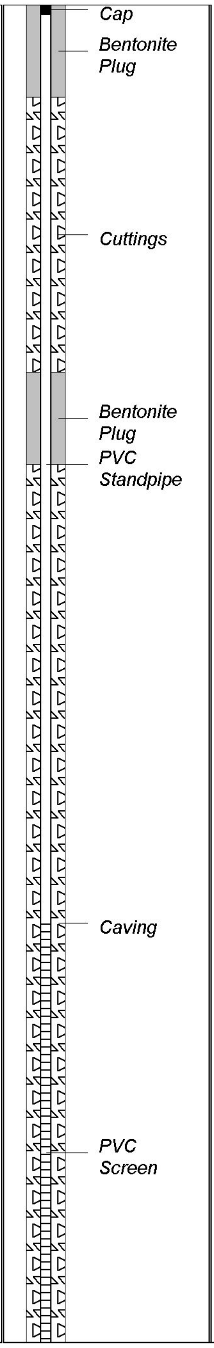
Description:

- Asphalt: 50 mm
- Gravelly Sand: Brown crushed gravelly sand, trace silt, moist, FILL
- Silty Sand: Brown silty sand, trace gravel, trace clay, moist, very loose, FILL
- Loose
- Sand: Brown sand, trace gravel, trace silt, moist, loose
- Wet
- Moist, compact
- Saturated, trace gravel
- Dense

SAMPLE

Number	Type	% Recovery	SPT (N)
2	SS	0	1
3	SS	70	9
4	SS	55	5
5	SS	75	15
6	SS	50	22
7	SS	60	31
8	SS	60	47





Remarks:

Top of well elevation: 229.80 mASL
Water level elevation measured on June 17, 2020: 225.99 mASL

GSA SS3:
1% Gravel
74% Sand
22% Silt
3% Clay

Switched drilling method to mud rotary at 3.0 mbgs

Logged By: JB ***Input By:*** CM

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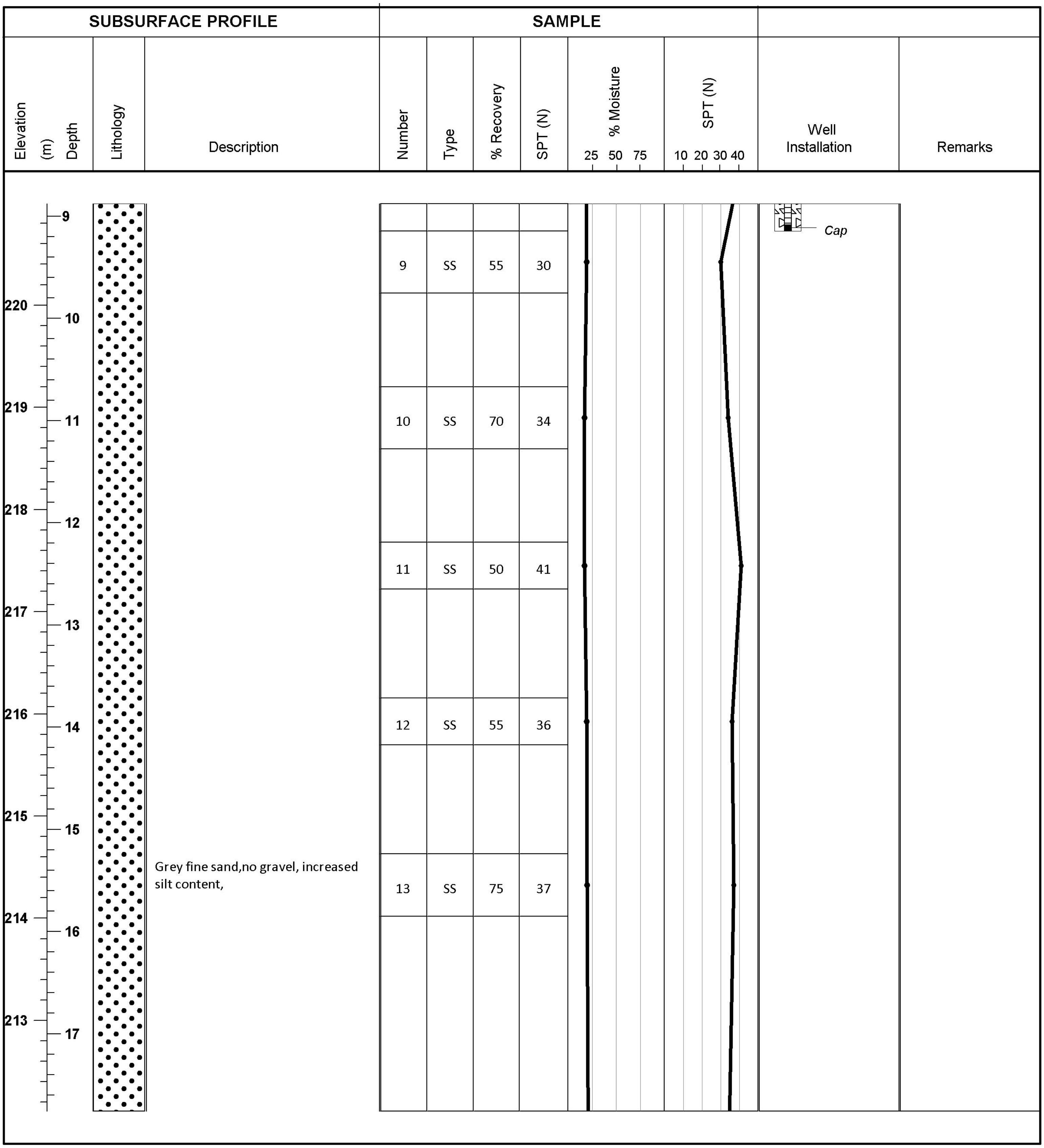
Log of Borehole: BH102-19

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
Project No.: 8707-001
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: July 5 & 8, 2019
Location: 272 Innisfil Street, Barrie, ON
UTM: 17T 604017, 4914048
Elevation: 229.87 mASL



Description	Number	Type	% Recovery	SPT (N)
	9	SS	55	30
	10	SS	70	34
	11	SS	50	41
	12	SS	55	36
Grey fine sand,no gravel, increased silt content,	13	SS	75	37

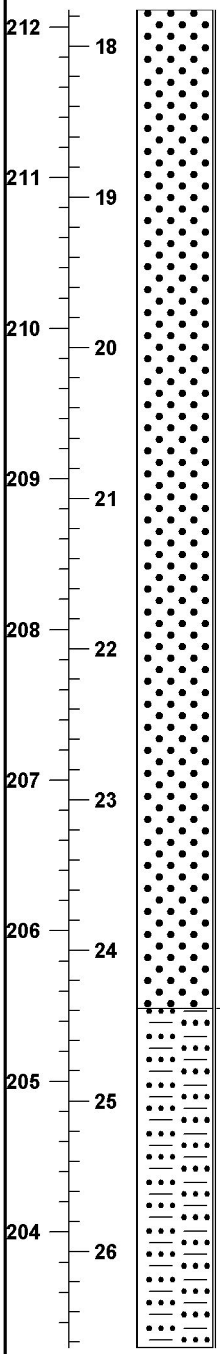
Logged By: JB
Input By: CM

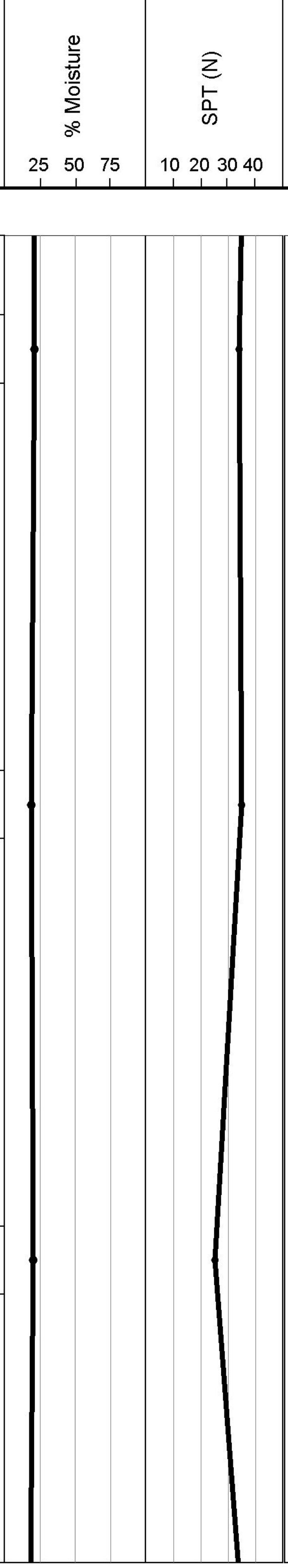
Log of Borehole: BH102-19

Page 3 of 4

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
Project No.: 8707-001
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: July 5 & 8, 2019
Location: 272 Innisfil Street, Barrie, ON
UTM: 17T 604017, 4914048
Elevation: 229.87 mASL

SUBSURFACE PROFILE		SAMPLE				Remarks
Lithology	Description	Number	Type	% Recovery	SPT (N)	Remarks
		14	SS	65	34	
		15	SS	70	35	
	Silty Sand: Brown silty sand, trace clay, moist, compact	16	SS	65	25	GSA SS16: 0% Gravel 63% Sand 34% Silt 3% Clay



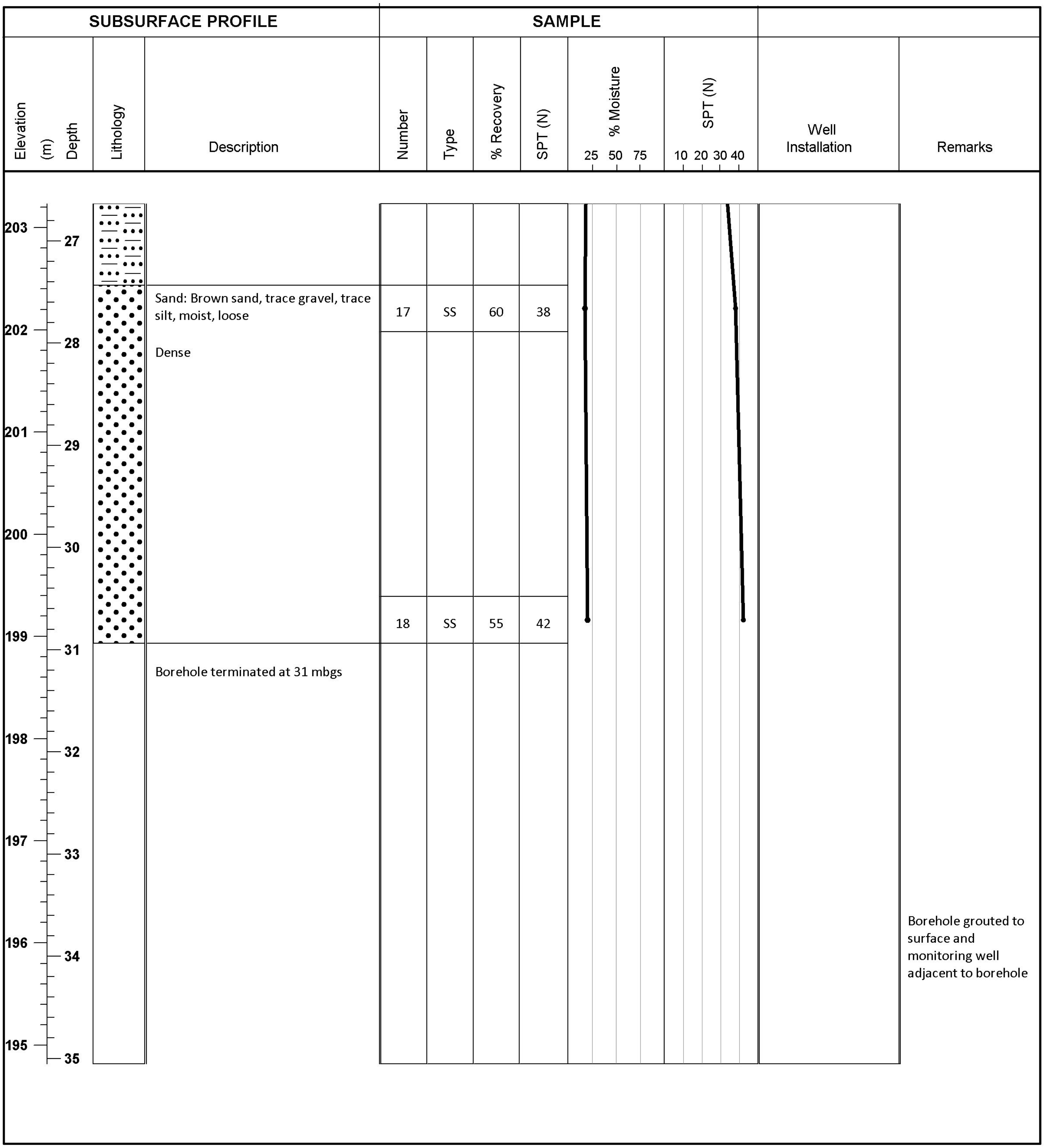


Well Installation

Logged By: JB ***Input By:*** CM

Log of Borehole: BH102-19

Client: Innovative Planning Solution
Project Name: Geotech Investigation - 272 Innisfil St
Project No.: 8707-001
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: July 5 & 8, 2019
Location: 272 Innisfil Street, Barrie, ON
UTM: 17T 604017, 4914048
Elevation: 229.87 mASL



Description	Number	Type	% Recovery	SPT (N)	Remarks
Sand: Brown sand, trace gravel, trace silt, moist, loose	17	SS	60	38	
Dense					
	18	SS	55	42	
Borehole terminated at 31 mbgs					Borehole grouted to surface and monitoring well adjacent to borehole

Logged By: JB ***Input By:*** CM

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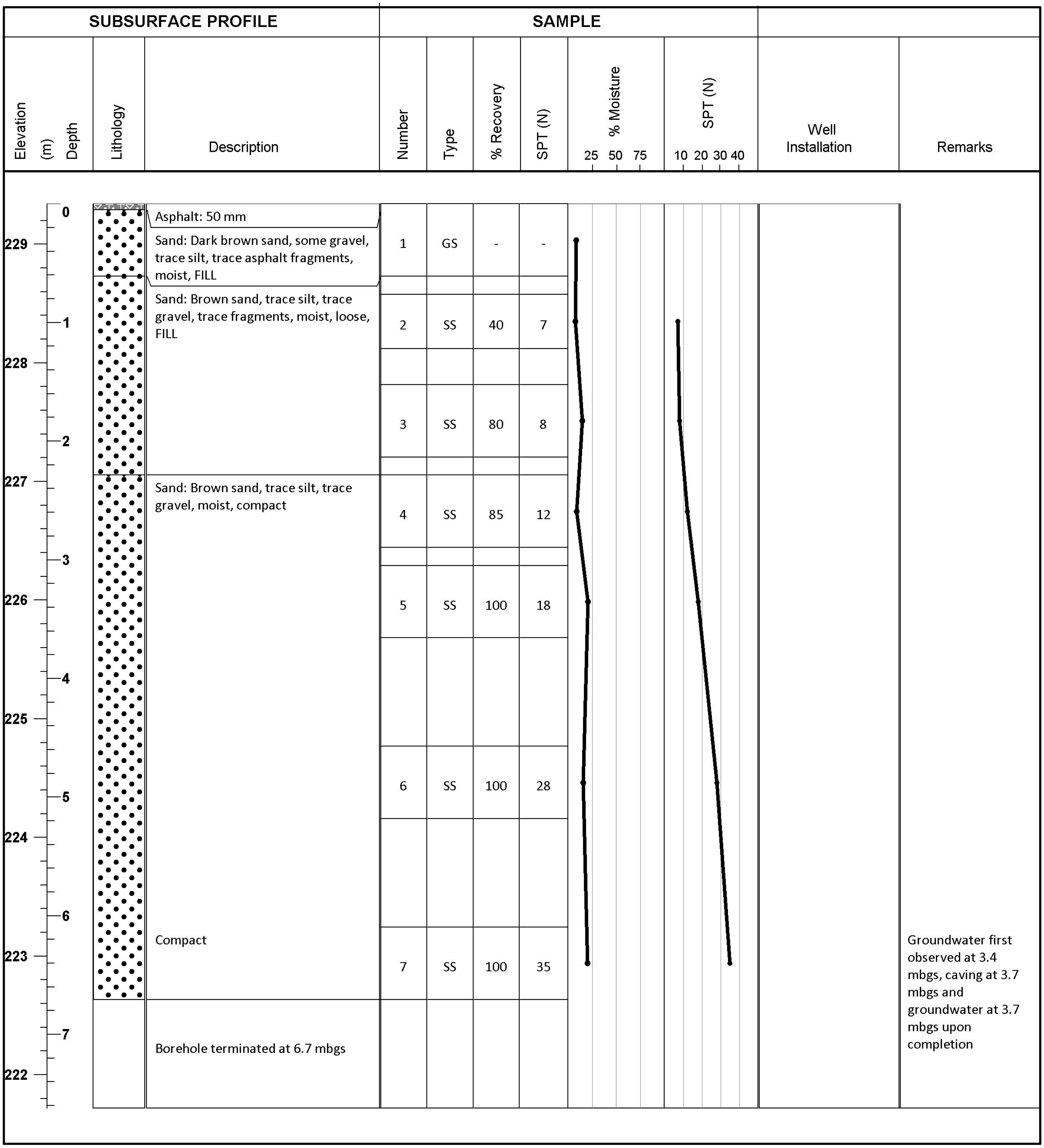
Log of Borehole: BH103-19

Page 1 of 1

Client: Innovative Planning Solution
Contractor: Walker Drilling
Location: 272 Innisfil Street, Barrie, ON
Project Name: Geotech Investigation - 272 Innisfil St
Method: Hollow Stem Augers
UTM: 17T 604052, 4914058
Project No.: 8707-001
Date Completed: August 14, 2019
Elevation: 229.34 mASL



SUBSURFACE PROFILE

Description
Asphalt: 50 mm
Sand: Dark brown sand, some gravel, trace silt, trace asphalt fragments, moist, FILL
Sand: Brown sand, trace silt, trace gravel, trace fragments, moist, loose, FILL
Sand: Brown sand, trace silt, trace gravel, moist, compact Compact
Borehole terminated at 6.7 mbgs

SAMPLE

Number	Type	% Recovery	SPT (N)
1	GS	-	-
2	SS	40	7
3	SS	80	8
4	SS	85	12
5	SS	100	18
6	SS	100	28
7	SS	100	35

Remarks

Groundwater first observed at 3.4 mbgs, caving at 3.7 mbgs and groundwater at 3.7 mbgs upon completion

Logged By: CM ***Input By:*** CM

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Log of Borehole: BH104-19

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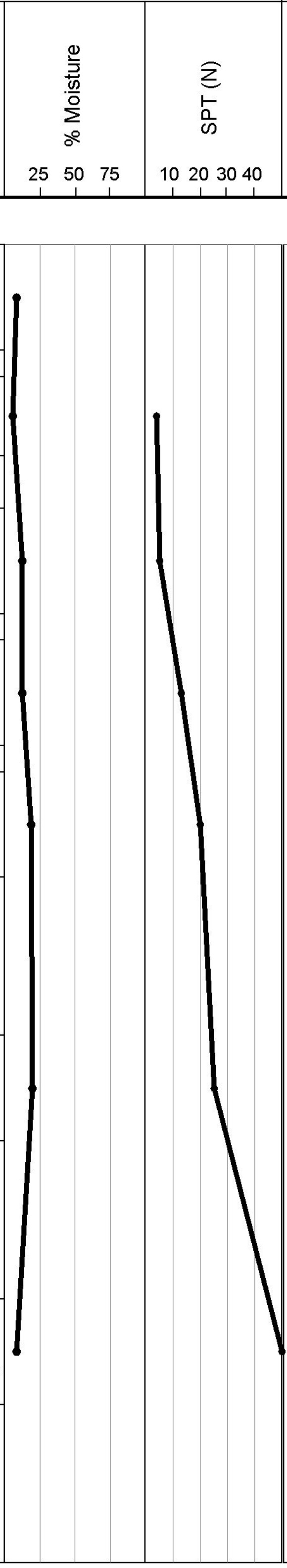
Client:	Innovative Planning Solution	***Project Name:***	Geotech Investigation - 272 Innisfil St	***Project No.:***	8707-001
Contractor:	Walker Drilling	***Method:***	Hollow Stem Augers	***Date Completed:***	August 14, 2019
Location:	272 Innisfil Street, Barrie, ON	***UTM:***	17T 604041, 4914034	***Elevation:***	229.80 mASL

SUBSURFACE PROFILE		SAMPLE				
Elevation (m) Depth	Description	Number	Type	% Recovery	SPT (N)	Remarks
0	Asphalt: 50 mm					
	Sand: Brown sand, trace silt, trace gravel, some asphalt fragments, moist, loose, FILL	1	GS	-	-	
229 / 1		2	SS	40	4	
228 / 2	Trace asphalt fragments	3	SS	50	5	
	Sand: Brown sand, some silt, trace clay, moist, compact	4	SS	90	13	GSA SS4: 0% Gravel 81% Sand 16% Silt 3% Clay
227 / 3	Dense	5	SS	70	20	
226 / 4						
225 / 5		6	SS	70	25	
224 / 6	Very dense	7	SS	100	50/ 279mm	Groundwater first observed at 4.0 mbgs, caving at 4.0 mbgs and groundwater at 4.0 mbgs upon completion
223 / 7	Borehole terminated at 6.7 mbgs					



Well Installation

Logged By: CM ***Input By:*** CM

Appendix B

Borehole Logs – 2022 Additional Investigation

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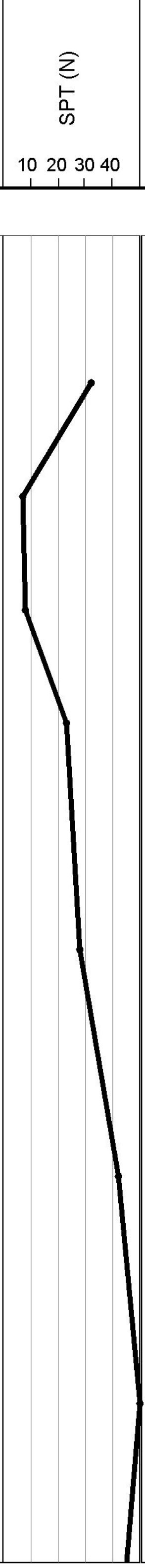
Log of Borehole: BH101-22

Client: Typhon Group Limited
Contractor: Walker Drilling
Location: 272 Innisfil St., Barrie, ON
Project Name: 272 Innisfil St., Barrie, ON
Method: Hollow Stem Augers + Mud Rotary
UTM: 17T 603990, 4914068
Project No.: 8707-003
Date Completed: Feb. 7, 2022
Elevation: 229.86 mASL

SUBSURFACE PROFILE				SAMPLE							
Elevation (m)	Depth	Lithology	Description	Number	Type	% Recovery	SPT (N)	% Moisture 25 50 75	SPT (N) 10 20 30 40	Well Installation	Remarks
	0		ASPHALT: Asphalt 50mm								
			SAND: Brown sand, some gravel, moist [FILL]	1	GS	-	-				
229	1		SAND: Brown sand, moist, loose	2A 2B	SS	50	32				
228	2			3	SS	30	7				
			-trace silt	4	SS	30	8				Switched drilling method to mud rotary at 2.3 mbgs
227	3		-compact	5	SS	30	23				
226	4										
225	5		-grey, saturated	6	SS	60	28				
224	6		-trace gravel, dense	7	SS	40	42				
223	7										
222	8		-brown, fine gravel seam between 7.6m and 7.7m	8	SS	50	50				
221											



Logged By: J. Schweighofer ***Input By:*** J. Schweighofer

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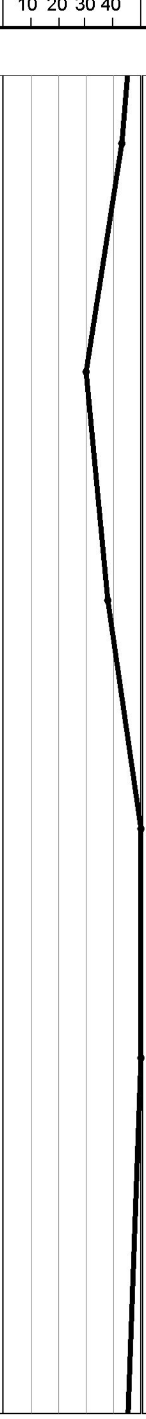
Log of Borehole: BH101-22

Client: Typhon Group Limited
Contractor: Walker Drilling
Location: 272 Innisfil St., Barrie, ON
Project Name: 272 Innisfil St., Barrie, ON
Method: Hollow Stem Augers + Mud Rotary
UTM: 17T 603990, 4914068
Project No.: 8707-003
Date Completed: Feb. 7, 2022
Elevation: 229.86 mASL

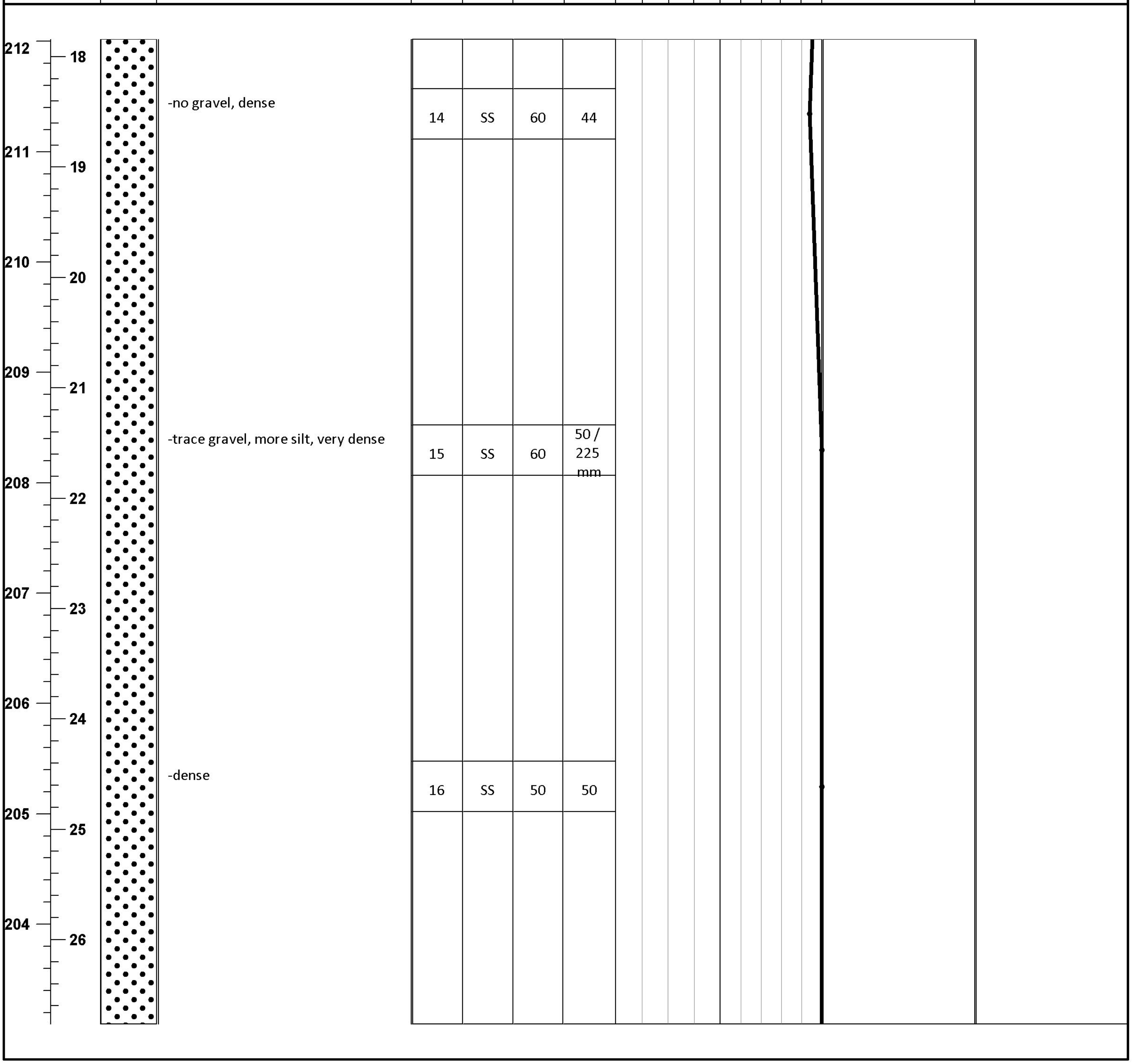
SUBSURFACE PROFILE			SAMPLE						
Elevation (m)	Depth	Description	Number	Type	% Recovery	SPT (N)	% Moisture	Well Installation	Remarks
	9								
			9	SS	50	43			
220	10								
219	11	-more silt	10	SS	60	30			
218	12								
		SAND: Brown sand, trace silt, trace clay, trace gravel, dense	11	SS	70	38			GSA SS11: 2% Gravel 83% Sand 8% Silt 7% Clay
217	13								
216	14		12	SS	60	50			
215	15								
		SAND: Grey sand, some silt, trace gravel, very dense	13	SS	70	50 / 250 mm			
214	16								
213	17								



Logged By: J. Schweighofer
Input By: J. Schweighofer

Client: Typhon Group Limited
Contractor: Walker Drilling
Location: 272 Innisfil St., Barrie, ON
Project Name: 272 Innisfil St., Barrie, ON
Method: Hollow Stem Augers + Mud Rotary
UTM: 17T 603990, 4914068
Project No.: 8707-003
Date Completed: Feb. 7, 2022
Elevation: 229.86 mASL

SUBSURFACE PROFILE	SAMPLE				
Description	Number	Type	% Recovery	SPT (N)	Remarks
-no gravel, dense	14	SS	60	44	
-trace gravel, more silt, very dense	15	SS	60	50 / 225 mm	
-dense	16	SS	50	50	



Logged By: J. Schweighofer ***Input By:*** J. Schweighofer

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Log of Borehole: BH101-22

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Client: Typhon Group Limited
Contractor: Walker Drilling
Location: 272 Innisfil St., Barrie, ON
Project Name: 272 Innisfil St., Barrie, ON
Method: Hollow Stem Augers + Mud Rotary
UTM: 17T 603990, 4914068
Project No.: 8707-003
Date Completed: Feb. 7, 2022
Elevation: 229.86 mASL

SUBSURFACE PROFILE			SAMPLE							
Elevation (m) Depth	Lithology	Description	Number	Type	% Recovery	SPT (N)	% Moisture 25 50 75	SPT (N) 10 20 30 40	Well Installation	Remarks
203 27										
		-very dense	17	SS	30	50 / 150 mm				
202 28										
201 29										
200 30										
		-trace gravel, increased silt content, trace clay	18	SS	50	50 / 150 mm				
199 31		Borehole terminated in SAND at 31 mbgs due to target exploration depth achieved.								
198 32										

Logged By: J. Schweighofer ***Input By:*** J. Schweighofer

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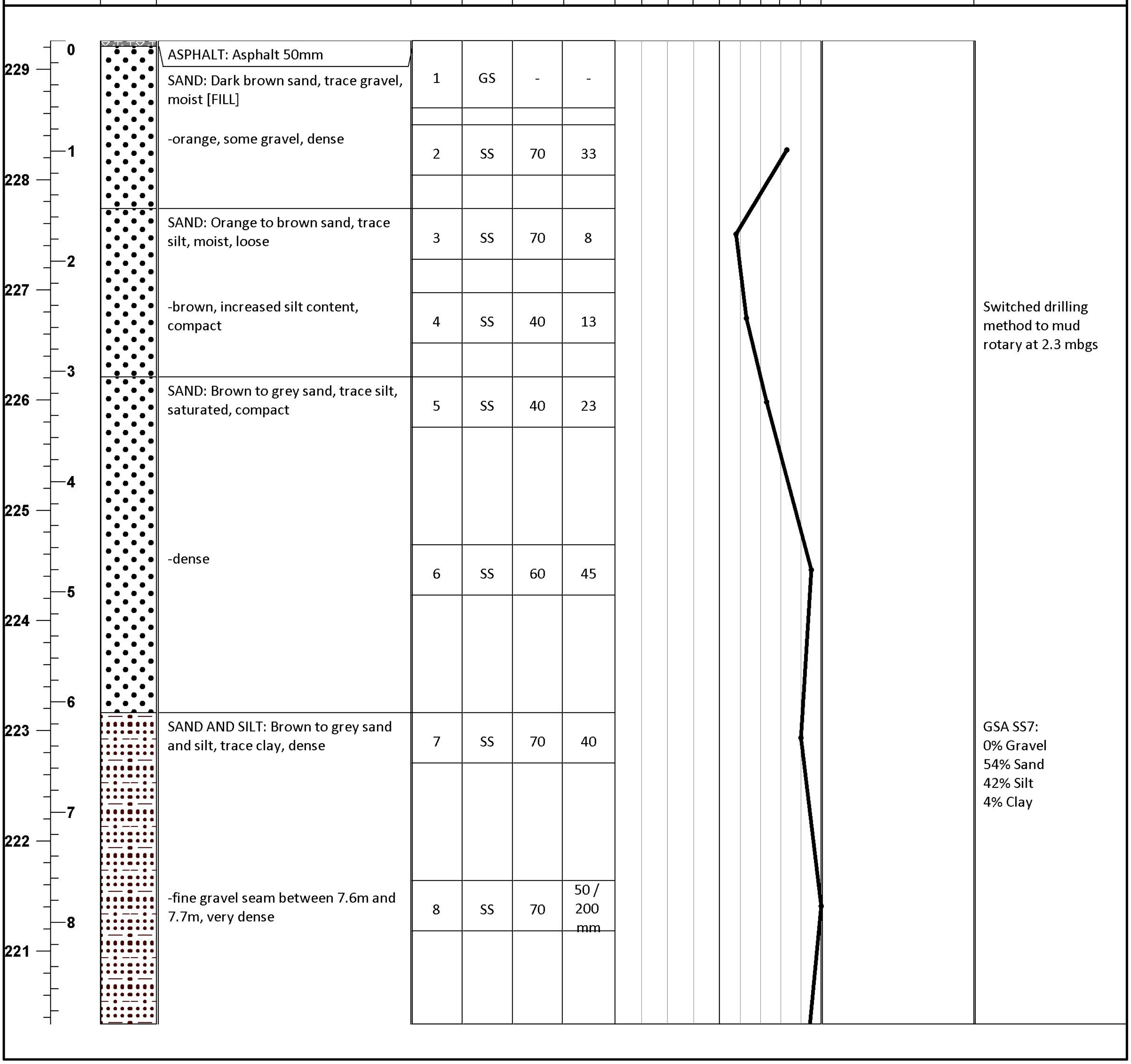
Log of Borehole: BH102-22

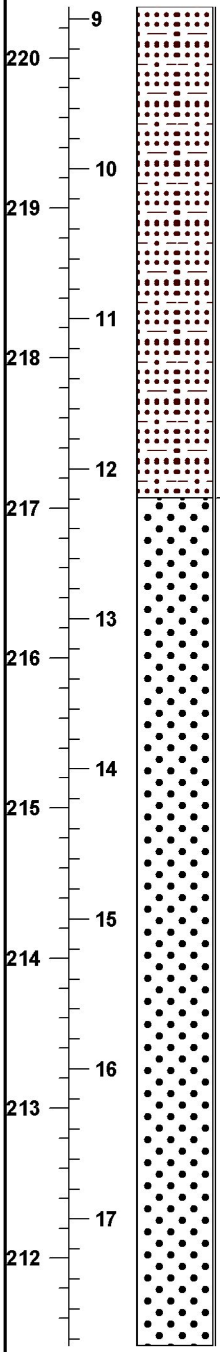
Client: Typhon Group Limited
Contractor: Walker Drilling
Location: 272 Innisfil St., Barrie
Project Name: 272 Innisfil St., Barrie, ON
Method: Hollow Stem Augers + Mud Rotary
UTM: 17T 604054, 4914058
Project No.: 8707-003
Date Completed: Feb. 8, 2022
Elevation: 229.26 mASL



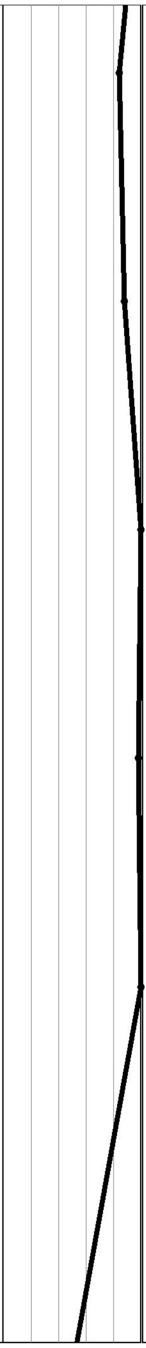
Description	Number	Type	% Recovery	SPT (N)	Remarks
ASPHALT: Asphalt 50mm					
SAND: Dark brown sand, trace gravel, moist [FILL]	1	GS	-	-	
-orange, some gravel, dense	2	SS	70	33	
SAND: Orange to brown sand, trace silt, moist, loose	3	SS	70	8	
-brown, increased silt content, compact	4	SS	40	13	Switched drilling method to mud rotary at 2.3 mbgs
SAND: Brown to grey sand, trace silt, saturated, compact	5	SS	40	23	
-dense	6	SS	60	45	
SAND AND SILT: Brown to grey sand and silt, trace clay, dense	7	SS	70	40	GSA SS7: 0% Gravel 54% Sand 42% Silt 4% Clay
-fine gravel seam between 7.6m and 7.7m, very dense	8	SS	70	50 / 200 mm	

Logged By: J. Schweighofer ***Input By:*** J. Schweighofer

Client:	Typhon Group Limited	***Project Name:***	272 Innisfil St., Barrie, ON	***Project No.:***	8707-003
Contractor:	Walker Drilling	***Method:***	Hollow Stem Augers + Mud Rotary	***Date Completed:***	Feb. 8, 2022
Location:	272 Innisfil St., Barrie	***UTM:***	17T 604054, 4914058	***Elevation:***	229.26 mASL



SUBSURFACE PROFILE		SAMPLE							
Description	Number	Type	% Recovery	SPT (N)	% Moisture 25 50 75	SPT (N) 10 20 30 40	Well Installation	Remarks
-dense	9	SS	70	42				
-less silt	10	SS	60	44				
SAND: Brown sand, some gravel, dense	11	SS	70	50				
	12	SS	80	49				
-very dense	13	SS	70	50 / 250 mm				



Logged By: J. Schweighofer **Input By:** J. Schweighofer

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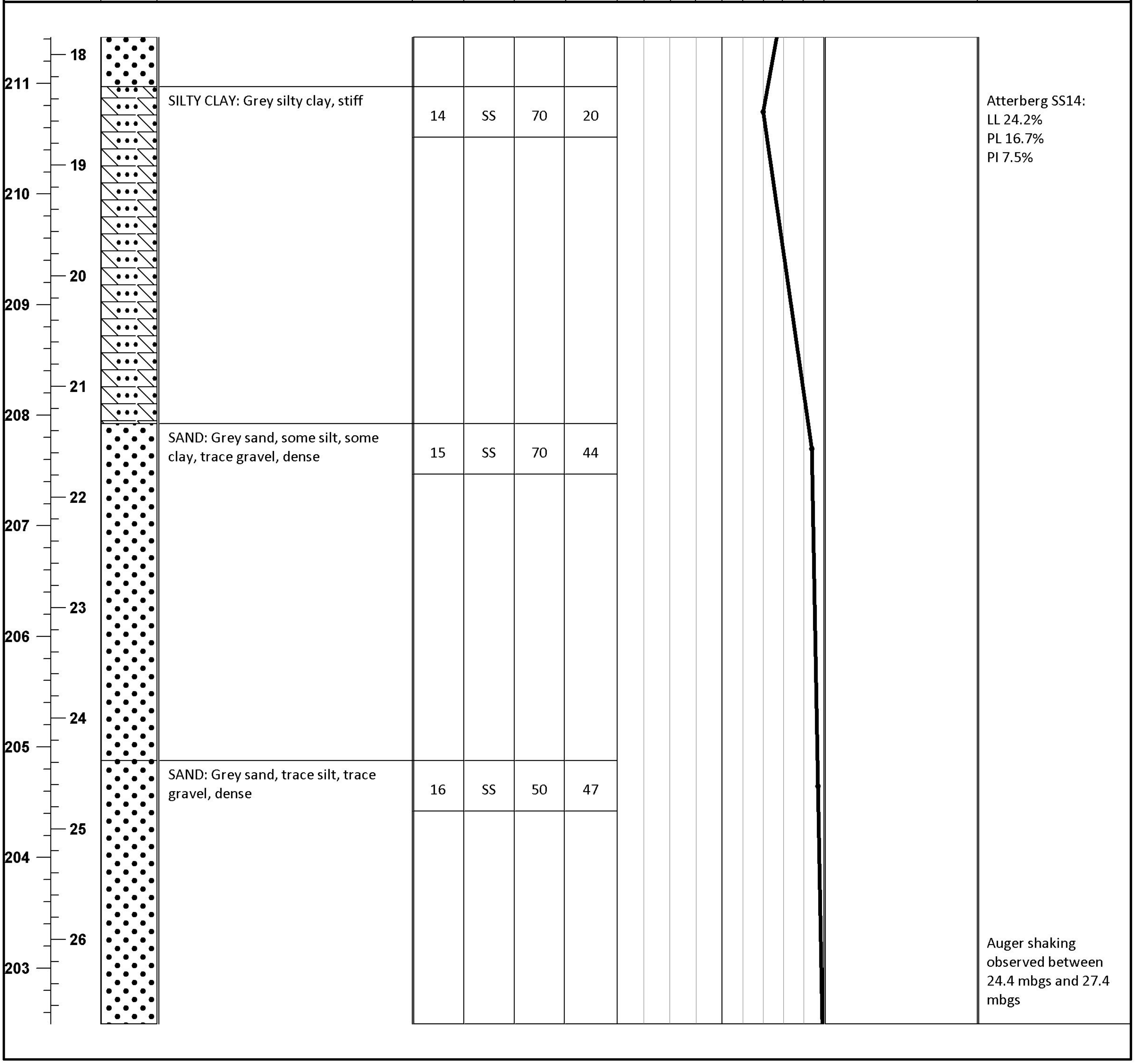
Log of Borehole: BH102-22

Client: Typhon Group Limited
Project Name: 272 Innisfil St., Barrie, ON
Project No.: 8707-003
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: Feb. 8, 2022
Location: 272 Innisfil St., Barrie
UTM: 17T 604054, 4914058
Elevation: 229.26 mASL



Description	Number	Type	% Recovery	SPT (N)	Remarks
SILTY CLAY: Grey silty clay, stiff	14	SS	70	20	Atterberg SS14: LL 24.2% PL 16.7% PI 7.5%
SAND: Grey sand, some silt, some clay, trace gravel, dense	15	SS	70	44	
SAND: Grey sand, trace silt, trace gravel, dense	16	SS	50	47	Auger shaking observed between 24.4 mbgs and 27.4 mbgs

Logged By: J. Schweighofer **Input By:** J. Schweighofer

Log of Borehole: BH102-22

Client: Typhon Group Limited
Project Name: 272 Innisfil St., Barrie, ON
Project No.: 8707-003
Contractor: Walker Drilling
Method: Hollow Stem Augers + Mud Rotary
Date Completed: Feb. 8, 2022
Location: 272 Innisfil St., Barrie
UTM: 17T 604054, 4914058
Elevation: 229.26 mASL

SUBSURFACE PROFILE				SAMPLE							
Elevation (m)	Depth	Lithology	Description	Number	Type	% Recovery	SPT (N)	% Moisture 25 50 75	SPT (N) 10 20 30 40	Well Installation	Remarks
	27										
202											
			-more gravel	17	SS	30	50				
	28										
201											
	29										
200											
	30										
199											
			-trace gravel, very dense	18	SS	40	50 / 175 mm				
	31										
198			Borehole terminated in SAND at 31 mbgs due to target exploration depth achieved.								
	32										

Logged By: J. Schweighofer
Input By: J. Schweighofer

Appendix C

Physical Laboratory Testing Results

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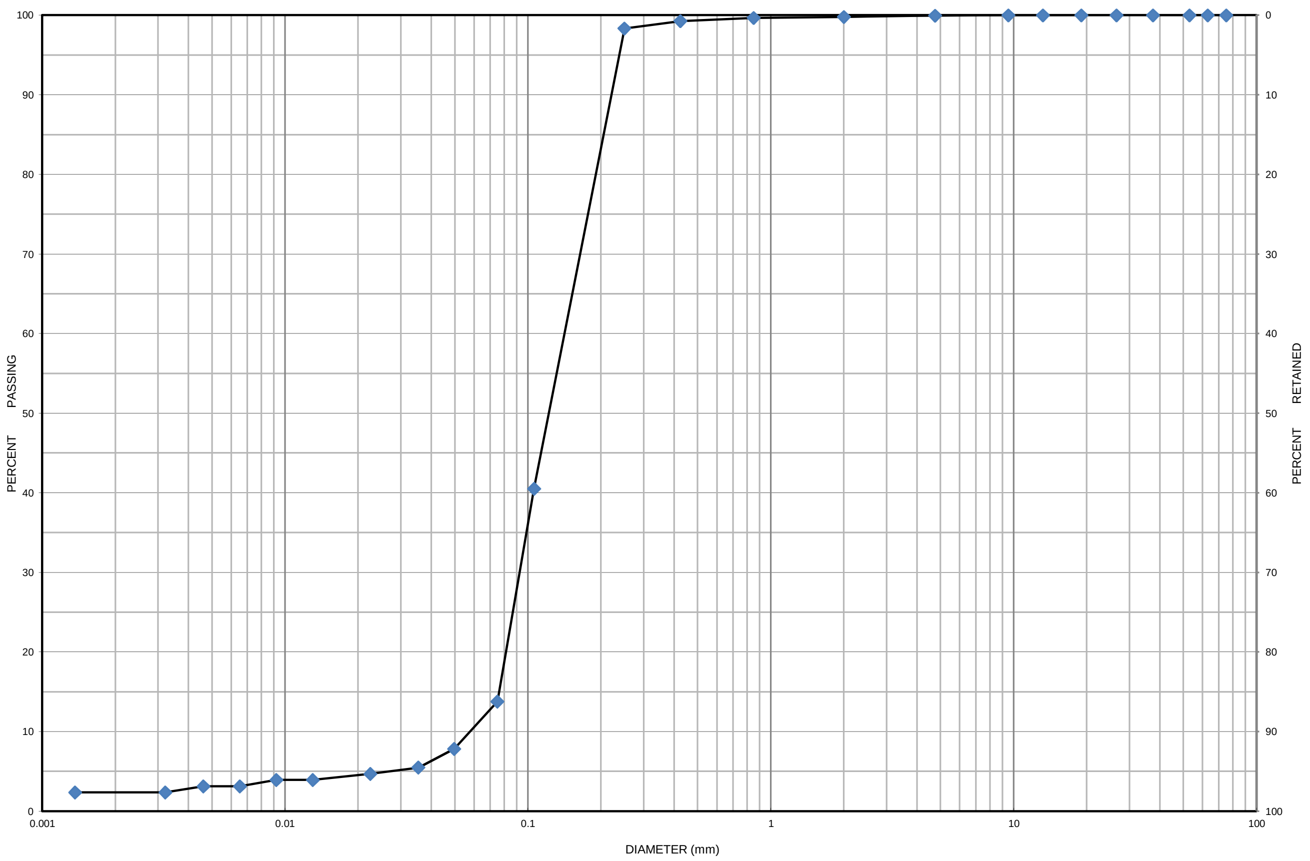
For specific tests as listed on www.ccil.com

Grain Size Distribution Chart

Project Number: *8707-001*
Client: *Innovative Planning Solutions*
Project Name: *272 Innisfil St., Barrie, ON*
Sample Date: *July 4, 2019*
Sampled By: *Jacob Bell - Cambium Inc.*
Location: *BH 101-19 SS 5*
Depth: *3 m to 3.7 m*
Lab Sample No: *S-19-0674*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 101-19	SS 5	3 m to 3.7 m	0	86	14		21.8

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Silt trace Clay	SP	0.150	0.092	0.057	2.63	0.99

Issued By: ______________________
(Senior Project Manager)

Date Issued: *August 29, 2019*

CAMBIUM

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For specific tests as listed on www.ccil.com

Grain Size Distribution Chart

Project Number: *8707-001* **Client:** *Innovative Planning Solutions*

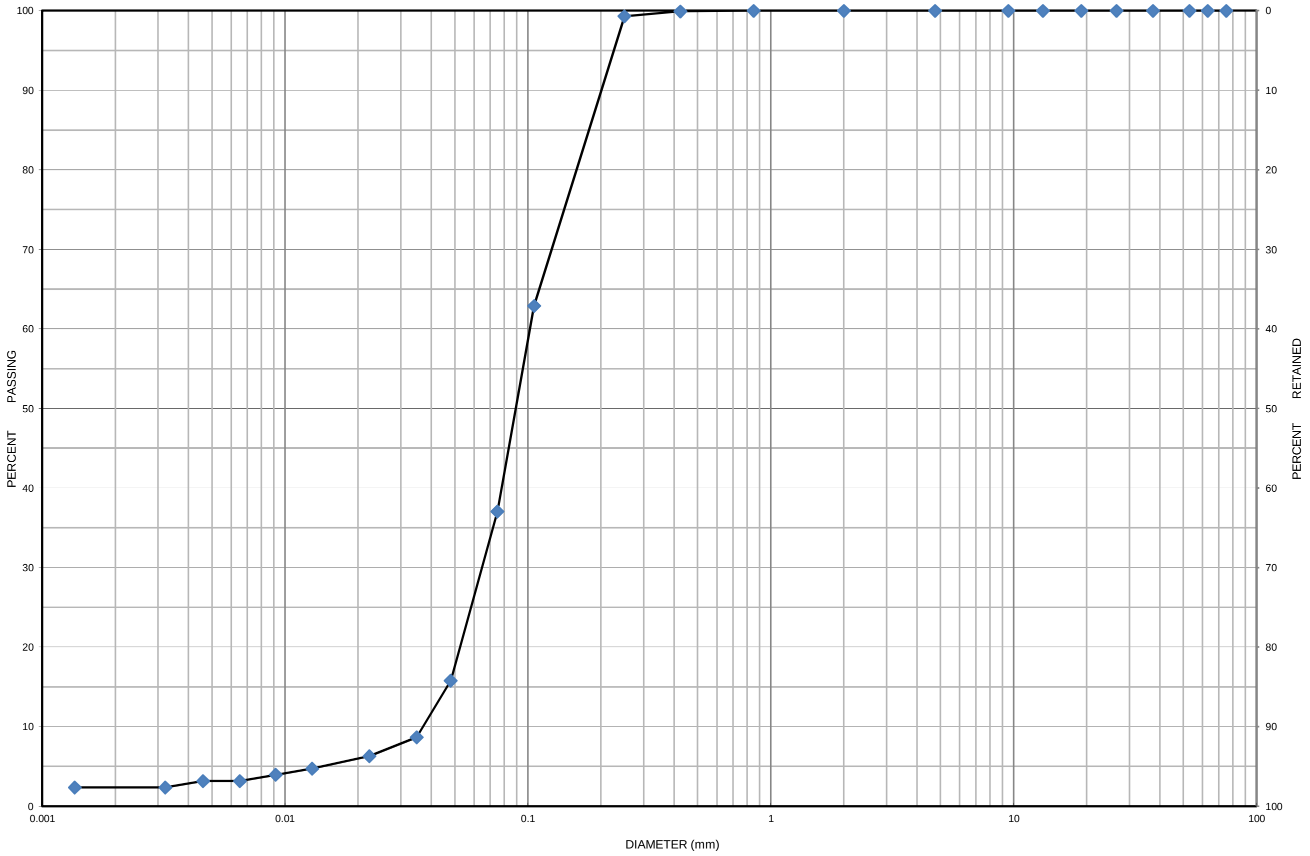
Project Name: *272 Innisfil St., Barrie, ON*

Sample Date: *July 4, 2019* **Sampled By:** *Jacob Bell - Cambium Inc.*

Location: *BH 102-19 SS 16* **Depth:** *24.4 m to 24.8 m* **Lab Sample No:** *S-19-0675*

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM

CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 102-19	SS 16	24.4 m to 24.8 m	0	63	37		19.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silty Sand trace Clay	SM	0.110	0.065	0.038	2.89	1.01

Issued By: ______________________ *Date Issued:* *August 29, 2019*

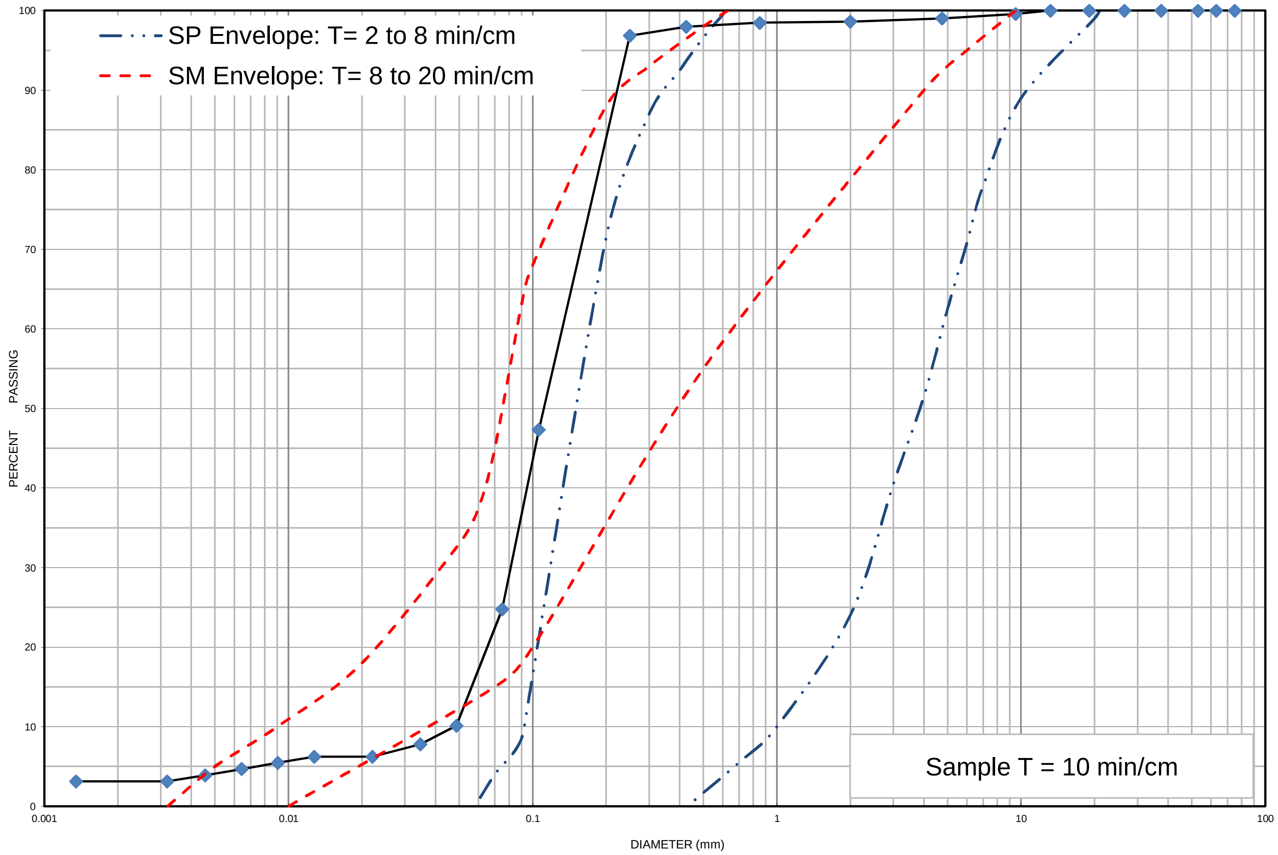
(Senior Project Manager)

Grain Size Distribution Chart

Project Number:	*8707-001*	**Client:**	*Innovative Planning Solutions*
Project Name:	*272 Innisfil St., Barrie, ON*		
Sample Date:	*July 4, 2019*	**Sampled By:**	*Jacob Bell - Cambium Inc.*
Location:	*BH 102-19 SS 3*	**Depth:**	*1.5 m to 2.1 m*

Lab Sample No: *S-19-0676*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 102-19	SS 3	1.5 m to 2.1 m	1	74	25		12.9

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silty Sand trace Clay trace Gravel	SM	0.140	0.0820	0.0	2.92	1.00

Issued By: ______________________ *Date Issued:* *August 29, 2019*

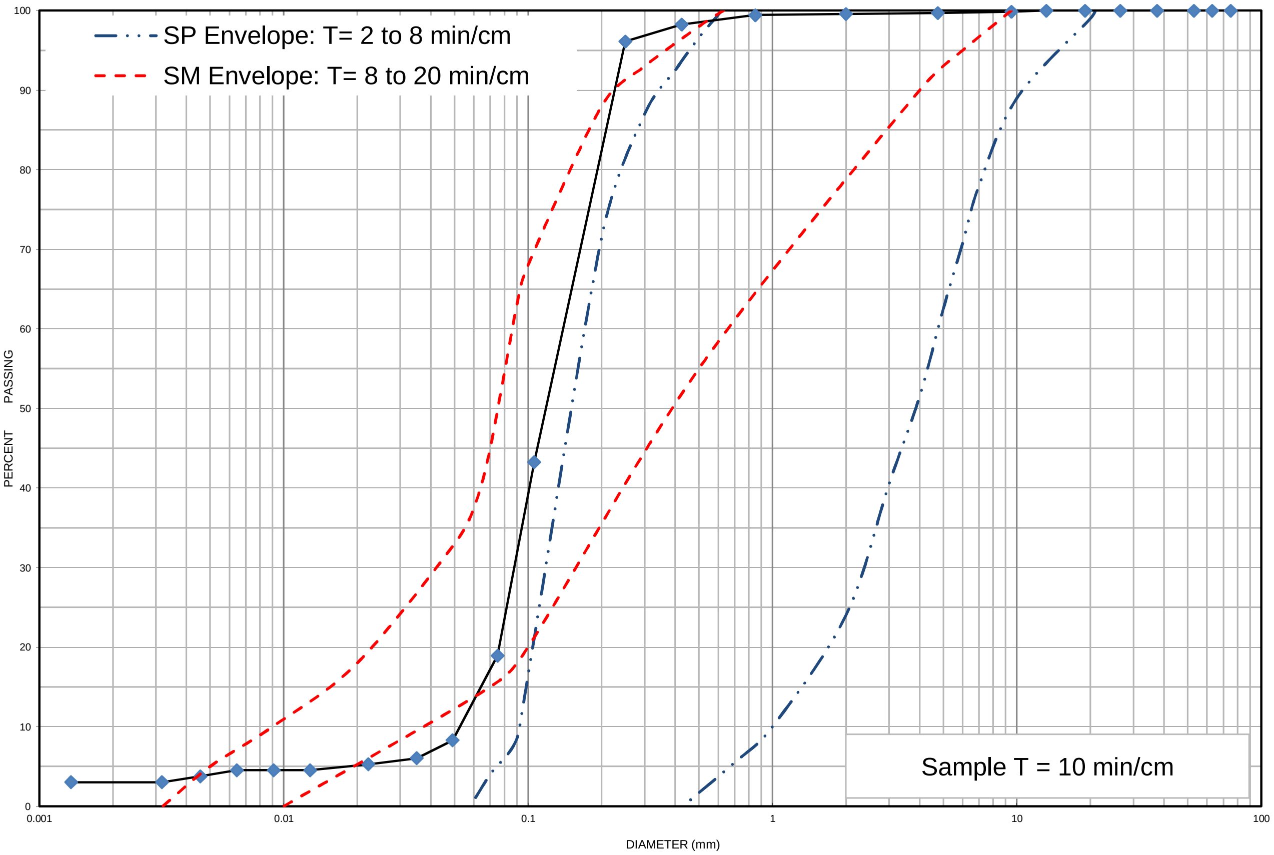
(Senior Project Manager)

Grain Size Distribution Chart

Project Number:	*8707-001*	**Client:**	*Innovative Planning Solutions*		
Project Name:	*272 Innisfil St., Barrie, ON*				
Sample Date:	*July 4, 2019*	**Sampled By:**	*Jacob Bell - Cambium Inc.*		
Location:	*BH 104-19 SS 4*	**Depth:**	*2.3 m to 2.7 m*	**Lab Sample No:**	*S-19-0677*

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM

CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 104-19	SS 4	2.3 m to 2.7 m	0	81	19		8.2

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Silt trace Clay	SP	0.140	0.087	0.053	2.64	1.02

Issued By: ______________________________ *Date Issued:* *August 29, 2019*

(Senior Project Manager)

Cambium Inc. (Laboratory)
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701 The Queensway | Units 5-6 | Peterborough | ON | K9J 7J6

Moisture Content

Project Number:
Project Name: 8707-001
Client: Innovative Planning Solutions
Date Taken: 2019-07-04
Lab Number: S-19-0673
Date Tested: 2019-08-20
Tested By: KD

Borehole Number	Sample Number	Sample Depth (m)	Water Weight (g)	Water Content (%)	Additional Observations
101	5	10-12	139.8	21.8	
102	3	5-7	85.6	12.9	
102	16	80-81.5	94.7	19.7	
104	4	7.5-9	69.8	8.2	
101	1	-	0.0	NaN	NO RECOVERY?
101	2	2.5-4.5	10.5	5.4	
101	3	5-7	21.8	11.6	
101	4	7.5-9.5	46.5	17.3	
101	6	15-17	49.6	20.3	
101	7	20-22	75.2	22.0	
101	8	25-27	53.5	20.4	
101	9	30-31.5	52.9	18.3	
101	10	35-37	40.4	15.3	
101	11	40-41.5	50.8	17.7	
101	12	45-46.5	41.9	13.5	
101	13	50-52	53.8	19.1	
101	14	60-61.5	57.7	17.0	
101	15	70-71.5	42.6	18.7	
101	17	80-81.5	62.2	20.2	
101	18	100-101.5	58.7	19.5	
101	19	110-111.5	50.0	17.2	
102	1	-	0.0	NaN	No Recovery
102	2	-	0.0	NaN	No Recovery
102	4	7.5-9	39.1	18.0	
102	5	10-11.5	21.4	11.0	
102	6	15-116.5	34.7	20.5	
102	7	20-21.5	60.5	19.9	

1 – Contains organics
2 – Contains rubble
3 – Hydrocarbon Odour
4 – Unknown Chemical Odour
5 – Saturated – free water visible
6 – Very moist – near optimum moisture content
7 – Moist – below optimum moisture
8 – Dry – dry texture – powdery
9 – Very small – caution may not be representative
10 – Hold sample for gradation analysis

Moisture Content

Project Number:
Project Name: 8707-001
Client: Innovative Planning Solutions
Date Taken: 2019-07-04

Lab Number: S-19-0673
Date Tested: 2019-08-20
Tested By: KD

Borehole Number	Sample Number	Sample Depth (m)	Water Weight (g)	Water Content (%)	Additional Observations
102	8	25-26.5	48.5	18.8	
102	9	30-31.5	56.3	18.9	
102	10	35-36.5	39.3	16.7	
102	11	40-41.5	42.2	16.8	
102	12	45-46.5	41.5	19.1	
102	13	50-51.5	51.8	19.4	
102	14	60-61.5	49.4	20.7	
102	15	70-71.5	50.0	18.4	
102	17	90-91.5	45.1	17.4	
102	18	100-101.5	34.2	19.8	
103	1 GS	0-2	17.0	8.3	1
103	2	2.5-4	13.8	7.1	
103	3	5-6.5	25.8	14.4	
103	4	7.5-9	19.8	8.5	
103	5	10-11.5	49.5	20.2	
103	6	15-16.5	33.0	15.4	
103	7	20-21.5	42.3	20.1	
104	1 GS	0-2	15.6	8.1	
104	2	2.5-4	11.5	5.6	
104	3	5-6.5	22.8	12.3	
104	5	10-11.5	20.5	12.2	
104	6	15-16.5	43.7	18.4	
104	7	21-21.5	46.5	19.3	

1 – Contains organics
2 – Contains rubble
3 – Hydrocarbon Odour
4 – Unknown Chemical Odour
5 – Saturated – free water visible
6 – Very moist – near optimum moisture content
7 – Moist – below optimum moisture
8 – Dry – dry texture – powdery
9 – Very small – caution may not be representative
10 – Hold sample for gradation analysis

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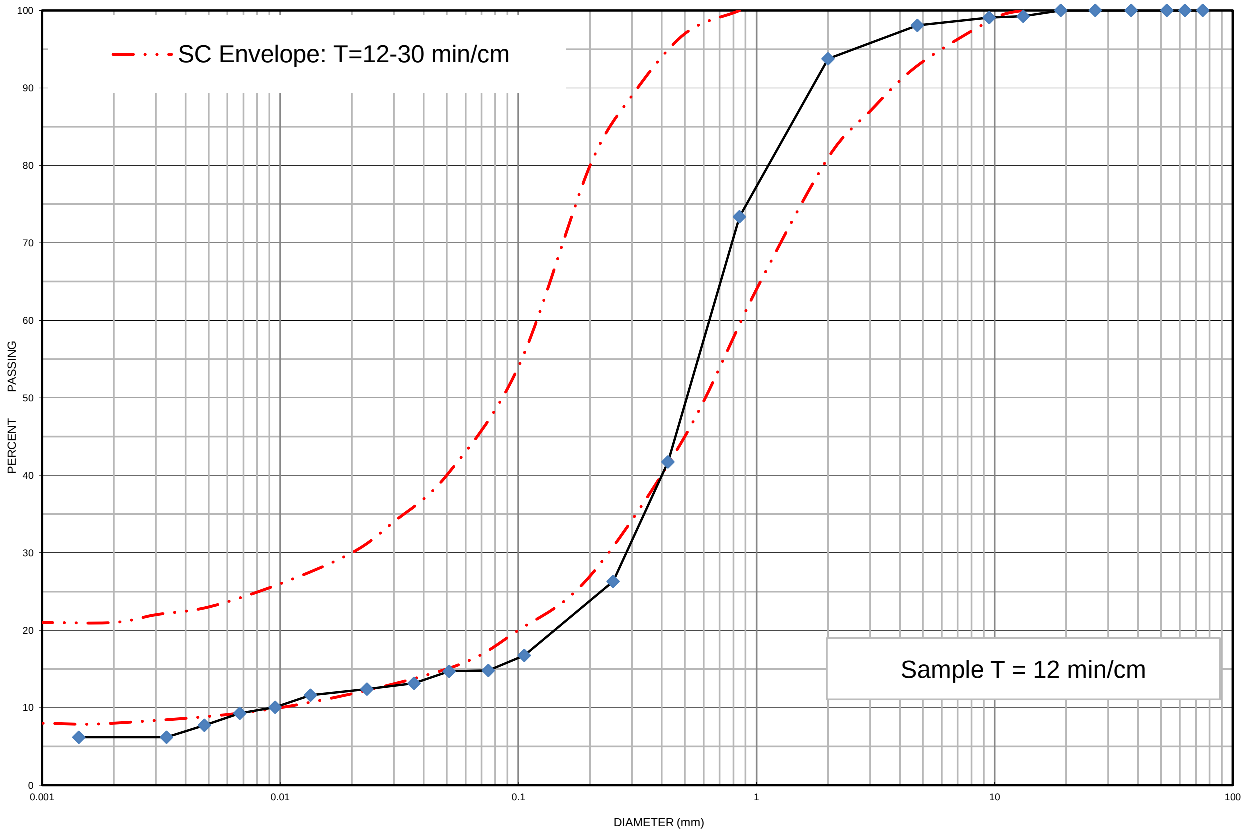
Grain Size Distribution Chart

Project Number: *8707-003* **Client:** *Ollie Switch Developments*
Project Name: *Updated Geotech & HydroG - 272 Innisfil St., Barrie*
Sample Date: *February 7-9, 2022* **Sampled By:** *Josef Schweighofer - Cambium Inc.*
Location: *BH 101-22 SS 11* **Depth:** *12.2 m to 12.7 m* **Lab Sample No:** *S-22-0313*

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM

CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 101-22	SS 11	12.2 m to 12.7 m	2	83	8	7	14.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand trace Silt trace Clay trace Gravel	SM	0.6400	0.2800	0.0086	74.42	14.24

Additional information available upon request

Issued By: ______________________ *Date Issued:* *February 27, 2022*

(Senior Project Manager)

Cambium Inc. (Laboratory)
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For specific tests as listed on www.ccil.com

Grain Size Distribution Chart

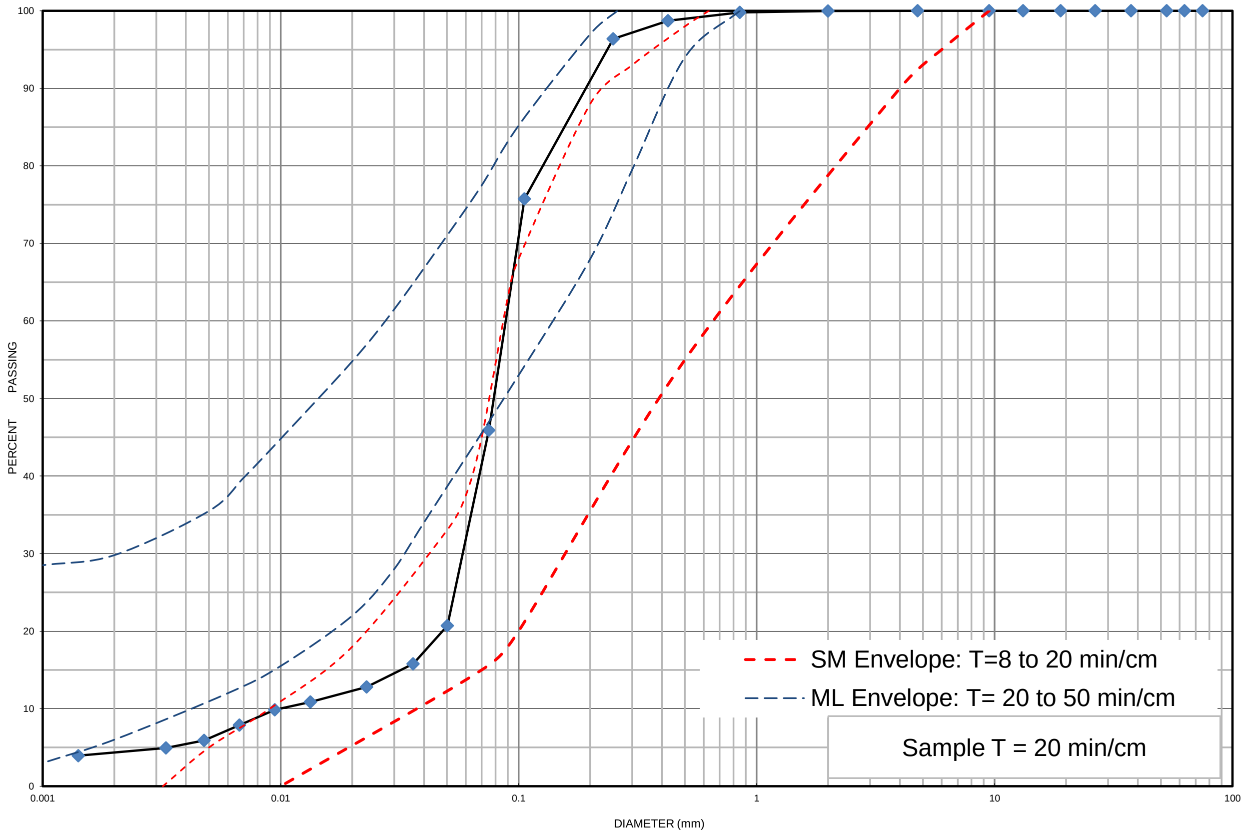
Project Number: 8707-003 **Client:** Ollie Switch Developments

Project Name: Updated Geotech & HydroG - 272 Innisfil St., Barrie

Sample Date: February 7-9, 2022 **Sampled By:** Josef Schweighofer - Cambium Inc.

Location: BH 102-22 SS 7 **Depth:** 6.1 m to 6.6 m **Lab Sample No:** S-22-0314

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 102-22	SS 7	6.1 m to 6.6 m	0	54	42	4	21.9

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand and Silt trace Clay	SM	0.089	0.058	0.010	8.90	3.78

Additional information availabe upon request

Issued By: ______________________
(Senior Project Manager)

Date Issued: *February 27, 2022*

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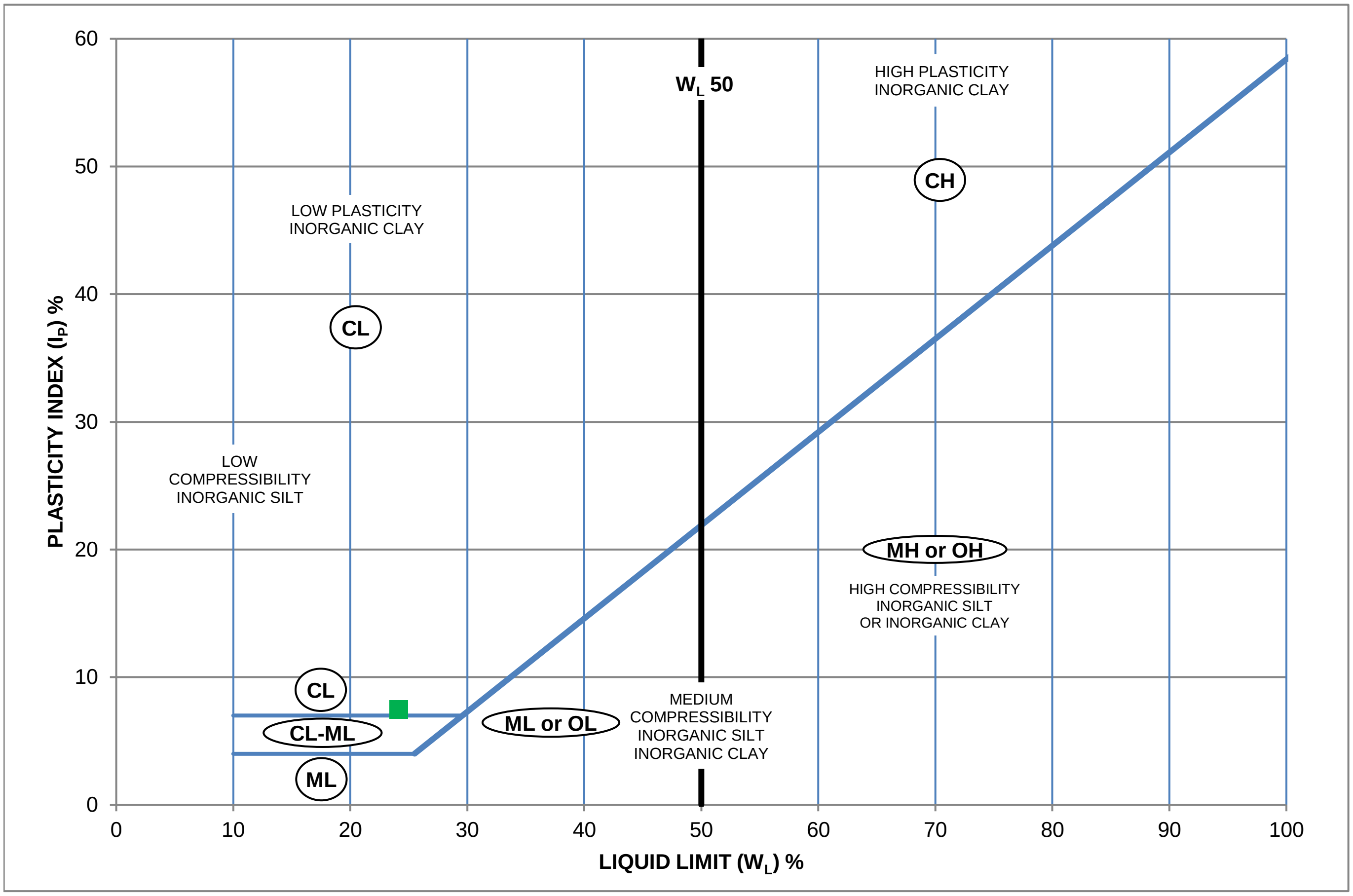
194 Sophia St.| Peterborough | ON | K9H 1E5

Plasticity Chart

Project Number: *8707-003* **Client:** *Ollie Switch Developments*
Project Name: *Updated Geotech & HydroG - 272 Innisfil St., Barrie*
Sampled By: *Josef Schweighofer - Cambium Inc.* **Sample Date:** *February 7-9, 2022*

Hole No.: *BH 102-22 SS 14* **Depth:** *18.3 m to 18.7 m* **Lab Sample No:** *S-22-0315*

Low Plasticity	High Plasticity



Symbol	Borehole	Sample	Depth	Description
🟩	BH 102-22	SS 14	18.3 m to 18.7 m	CL-ML

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
24.2	***16.7***	***7.5***

Issued By: ____________________ *Date Issued:* *February 27, 2022*

(Senior Project Manager)

Moisture Content

Project Number: 8707-003
Project Name: Updated Geotech & HydroG - 272 Innisfil St., Barrie
Client: Ollie Switch Developments
Date Taken: 2022-02-07
Lab Number: S-22-0312
Date Tested: 2022-02-18
Tested By: K. Dickson

Borehole Number	Sample Number	Sample Depth (m)	Water Weight (g)	Water Content (%)	Additional Observations
101	01	0.06-0.61	6.6	2.9	
101	02A	0.76-0.91	18.1	17.9	NR
101	02B	0.91-1.22	25.3	8.2	NR
101	03	1.52-1.98	11.5	7.2	
101	04	2.29-2.74	23.6	16.5	
101	05	3.05-3.51	39.9	21.2	
101	06	4.57-5.03	55.5	19.7	
101	07	6.10-6.55	65.5	23.7	NR
101	08	7.62-8.08	42.1	14.3	
101	09	9.14-9.60	42.5	17.3	
101	10	10.67-11.13	45.4	18.3	
101	11	12.19-12.65	91.6	14.8	
101	12	13.72-14.17	37.6	17.6	
101	13	15.24-15.70	46.1	18.4	
101	14	18.29-18.75	32.3	17.1	
101	15	21.34-21.79	40.8	18.8	
101	16	24.38-24.84	45.9	19.3	
101	17	27.43-27.89	56.4	19.5	
101	18	30.48-30.94	50.6	16.1	
102	01	0.00-0.61	19.8	13.7	
102	02	0.76-1.22	24.9	13.2	
102	03	1.52-1.98	20.0	10.9	
102	04	2.29-2.74	28.8	13.6	
102	05	3.05-3.51	50.2	24.0	
102	06	4.57-5.03	60.7	20.9	
102	07	6.10-6.55	87.8	23.1	
102	08	7.62-8.08	40.7	17.4	

1 – Contains organics
2 – Contains rubble
3 – Hydrocarbon Odour
4 – Unknown Chemical Odour
5 – Saturated – free water visible
6 – Very moist – near optimum moisture content
7 – Moist – below optimum moisture
8 – Dry – dry texture – powdery
9 – Very small – caution may not be representative
10 – Hold sample for gradation analysis

Moisture Content

Project Number: 8707-003
Project Name: Updated Geotech & HydroG - 272 Innisfil St., Barrie
Client: Ollie Switch Developments
Date Taken: 2022-02-07

Lab Number: S-22-0312
Date Tested: 2022-02-18
Tested By: K. Dickson

Borehole Number	Sample Number	Sample Depth (m)	Water Weight (g)	Water Content (%)	Additional Observations
102	09	9.14-9.60	51.8	19.9	
102	10	10.67-11.13	59.8	20.4	
102	11	12.19-12.65	42.5	16.1	
102	12	13.72-14.17	60.6	15.2	
102	13	15.24-15.70	48.8	16.0	
102	14	18.29-18.75	142.5	19.8	NR
102	15	21.34-21.79	50.4	19.9	
102	16	24.38-24.84	47.3	15.3	
102	17	27.43-27.89	49.7	13.0	NR
102	18	30.48-30.94	58.3	19.6	NR
103	01	0.30-1.52	10.4	6.4	
104	01	0.15-1.52	16.1	8.2	
105	01	0.18-0.91	20.7	9.8	
106	01	0.00-0.00	11.3	5.4	

1 – Contains organics
2 – Contains rubble
3 – Hydrocarbon Odour
4 – Unknown Chemical Odour
5 – Saturated – free water visible

6 – Very moist – near optimum moisture content
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Appendix D

CPT Investigation Report

PRESENTATION OF SITE INVESTIGATION RESULTS

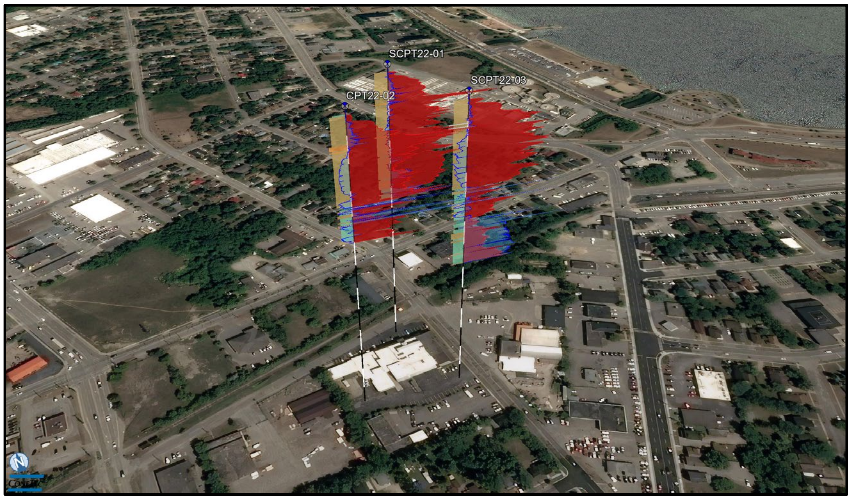
272 Innisfil St. Barrie

Prepared for:

Cambium Inc.

ConeTec Job No: 22-05-23587

Project Start Date: 10-Feb-2022
Project End Date: 10-Feb-2022
Report Date: 17-Feb-2022



Prepared by:

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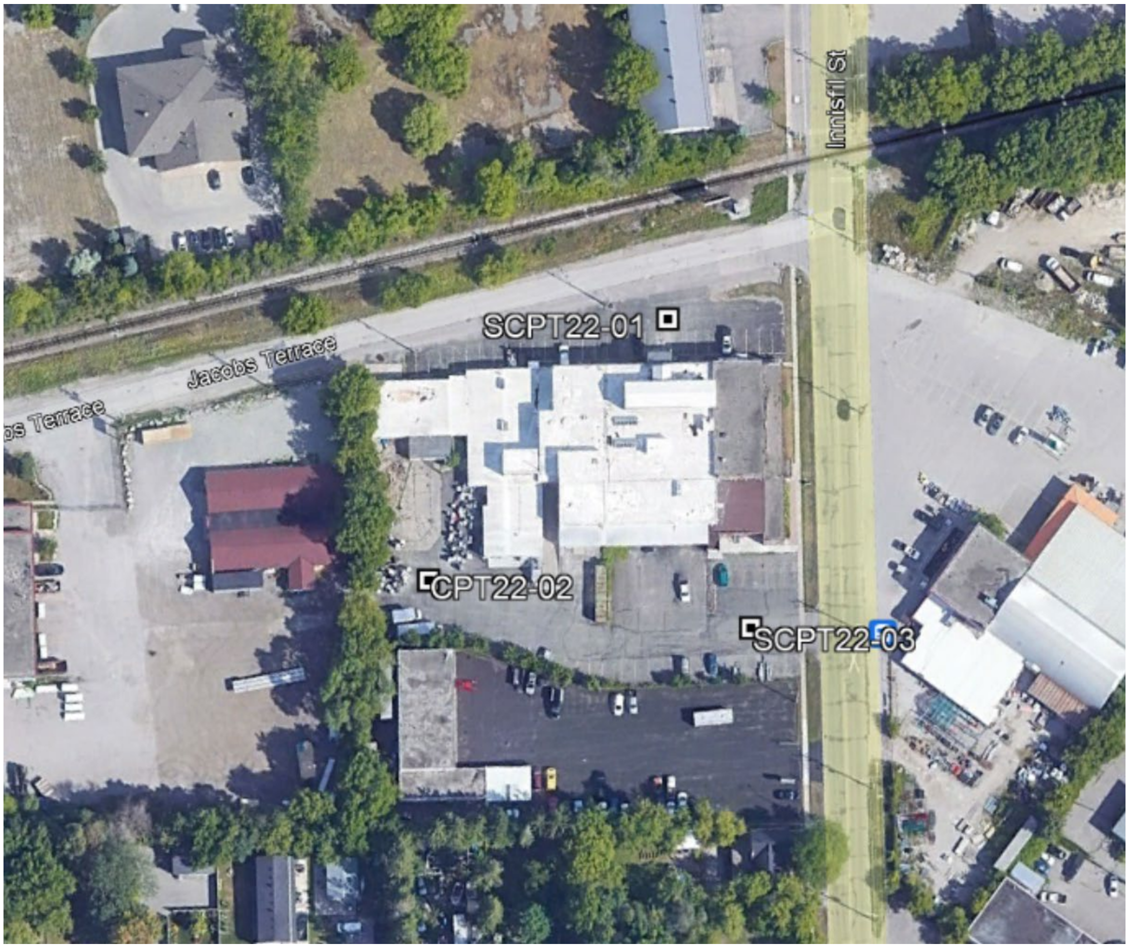
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Cambium Inc. at 272 Innisfil St. Barrie, ON. The program consisted of one cone penetration tests (CPTu), and two 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	Cambium Inc.
Project	272 Innisfil St. Barrie
ConeTec project number	22-05-23587

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



Rig Description	Deployment System	Test Type
CPT Truck Rig (C3)	30 ton rig cylinder	CPTu, SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu, SCPTu	Consumer grade GPS	32617

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)
642:T1500F15U500	642	15	225	1500	15	35

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	• Advanced plots with Ic, phi and N1(60) • Soil Behaviour Type (SBT) scatter plots • Seismic Cone Penetration Test Plots and Vs Wave Traces

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). 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 “272 Innisfil St. Barrie”, referred to as the (“Report”), was prepared by ConeTec for Cambium Inc.. 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 Cambium Inc. to collect and provide the raw data (“Data”) which is included in this report titled “272 Innisfil St. 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.

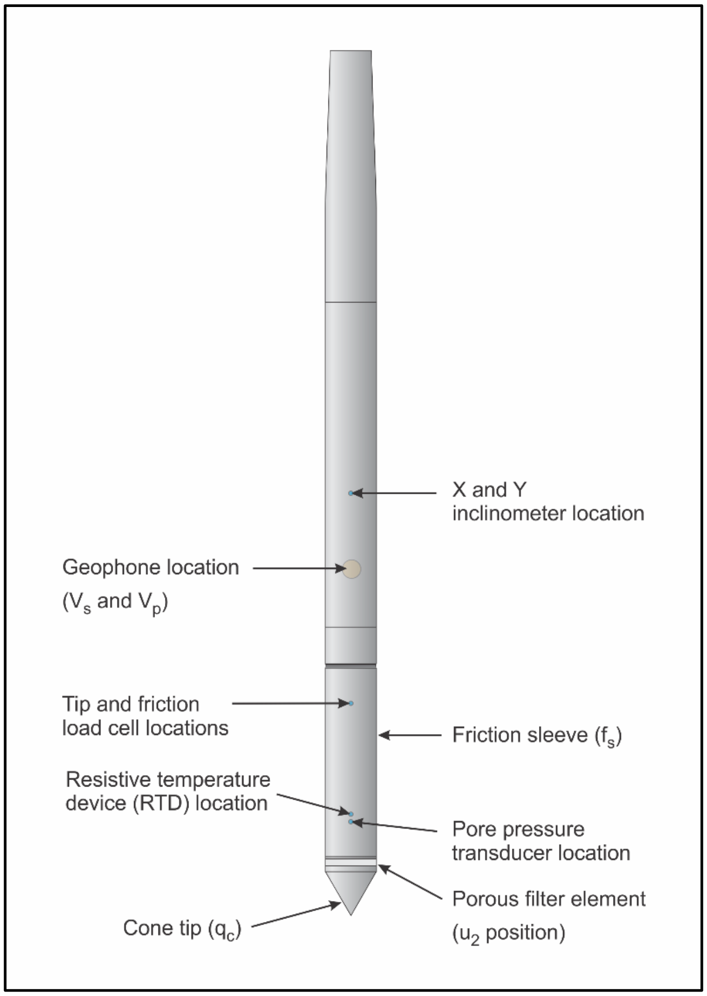


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

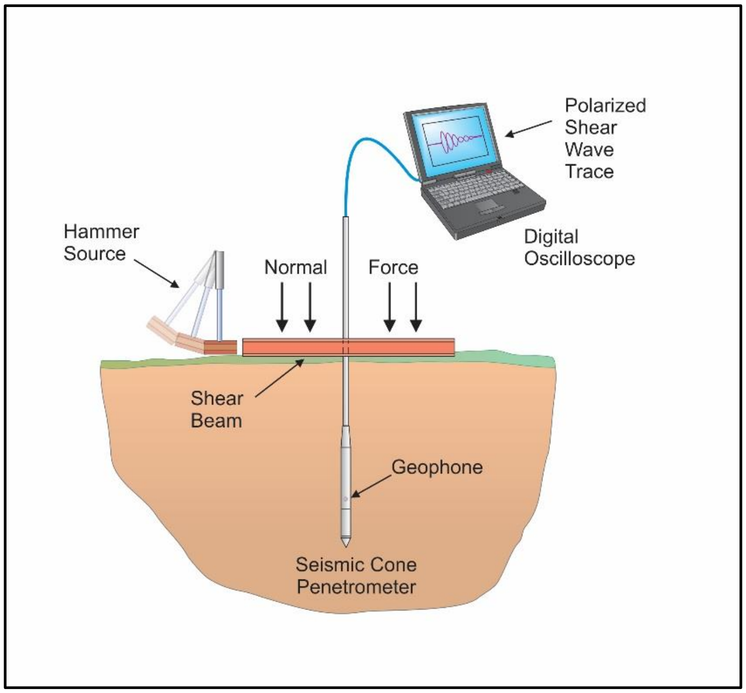


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

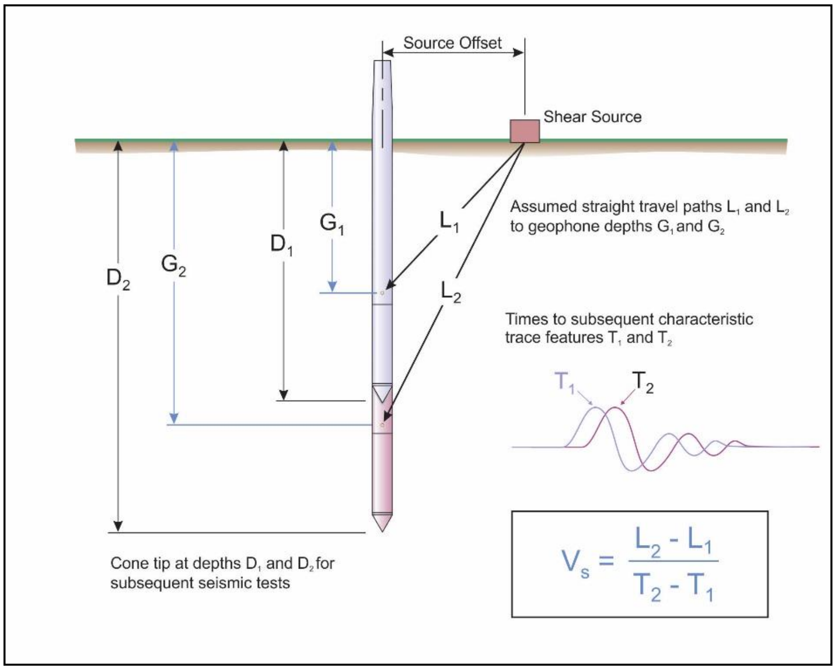


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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.

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

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).

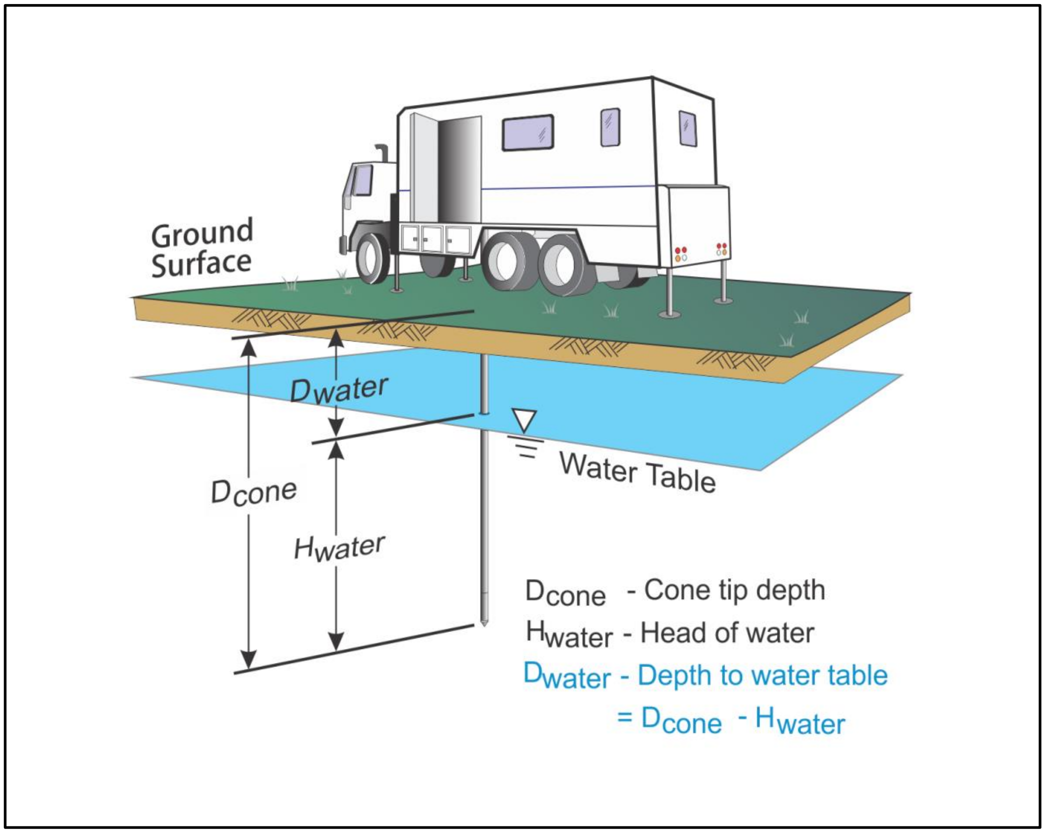


Figure PPD-1. Pore pressure dissipation test setup

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.

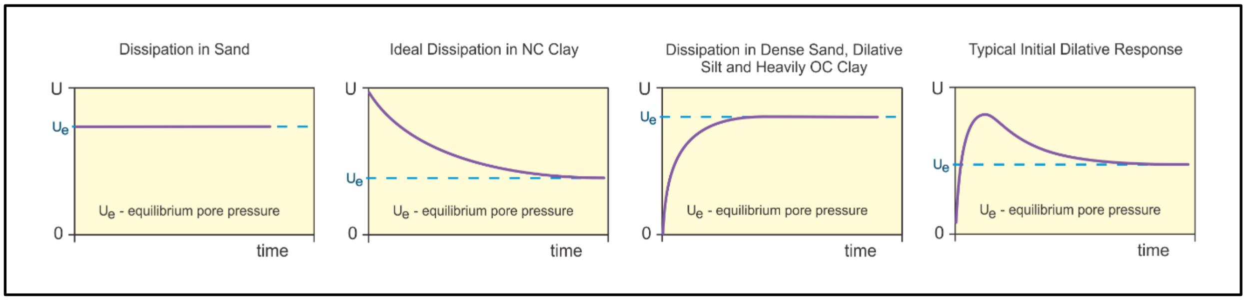


Figure PPD-2. Pore pressure dissipation curve examples

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.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

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

References

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

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Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

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

CONETEC

Job No: 22-05-23587
Client: Cambium Inc
Project: 272 Innisfil St. Barrie
Start Date: 10-Feb-2022
End Date: 10-Feb-2022

CONE PENETRATION TEST SUMMARY

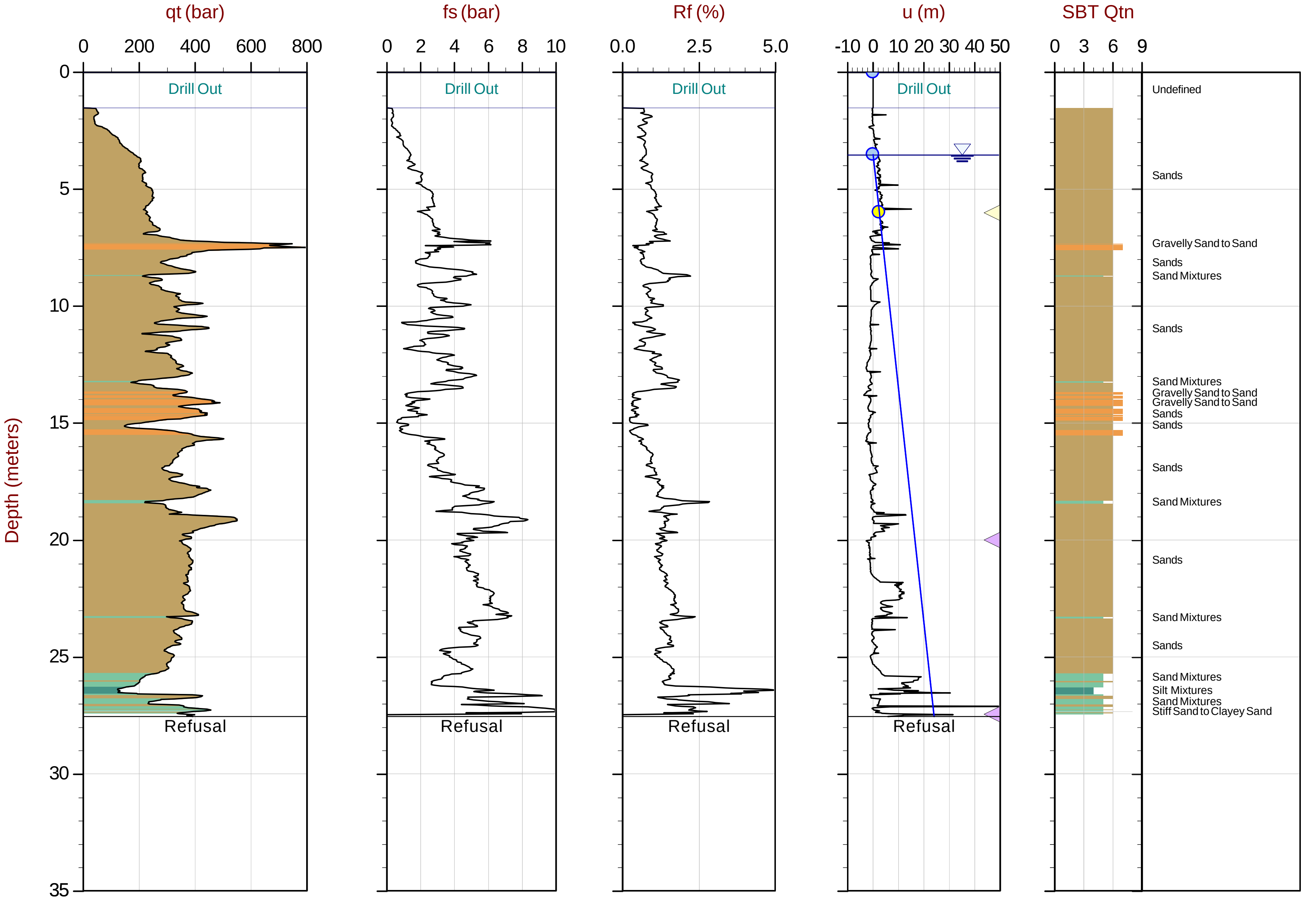
Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)
SCPT22-01	22-05-23587_SP01	10-Feb-2022	642:T1500F15U35	15	3.5	27.550	4914109	604028
CPT22-02	22-05-23587_CP02	10-Feb-2022	642:T1500F15U35	15	4.1	25.200	4914043	603986
SCPT22-03	22-05-23587_SP03	10-Feb-2022	642:T1500F15U35	15	3.0	34.125	4914045	604058

1. Phreatic surface was assumed based on pore pressure dissipation tests unless otherwise noted. A hydrostatic profile was used for the calculated parameters.
2. Coordinates were collected using a consumer grade hand-held GPS. Datum: WGS84 / UTM 17N

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 08:47
Site: 272 Innsfil St.

Sounding: SCPT22-01
Cone: 642:T1500F15U35



Max Depth: 27.550 m / 90.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914109 E: 604028
Sheet No: 1 of 1

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

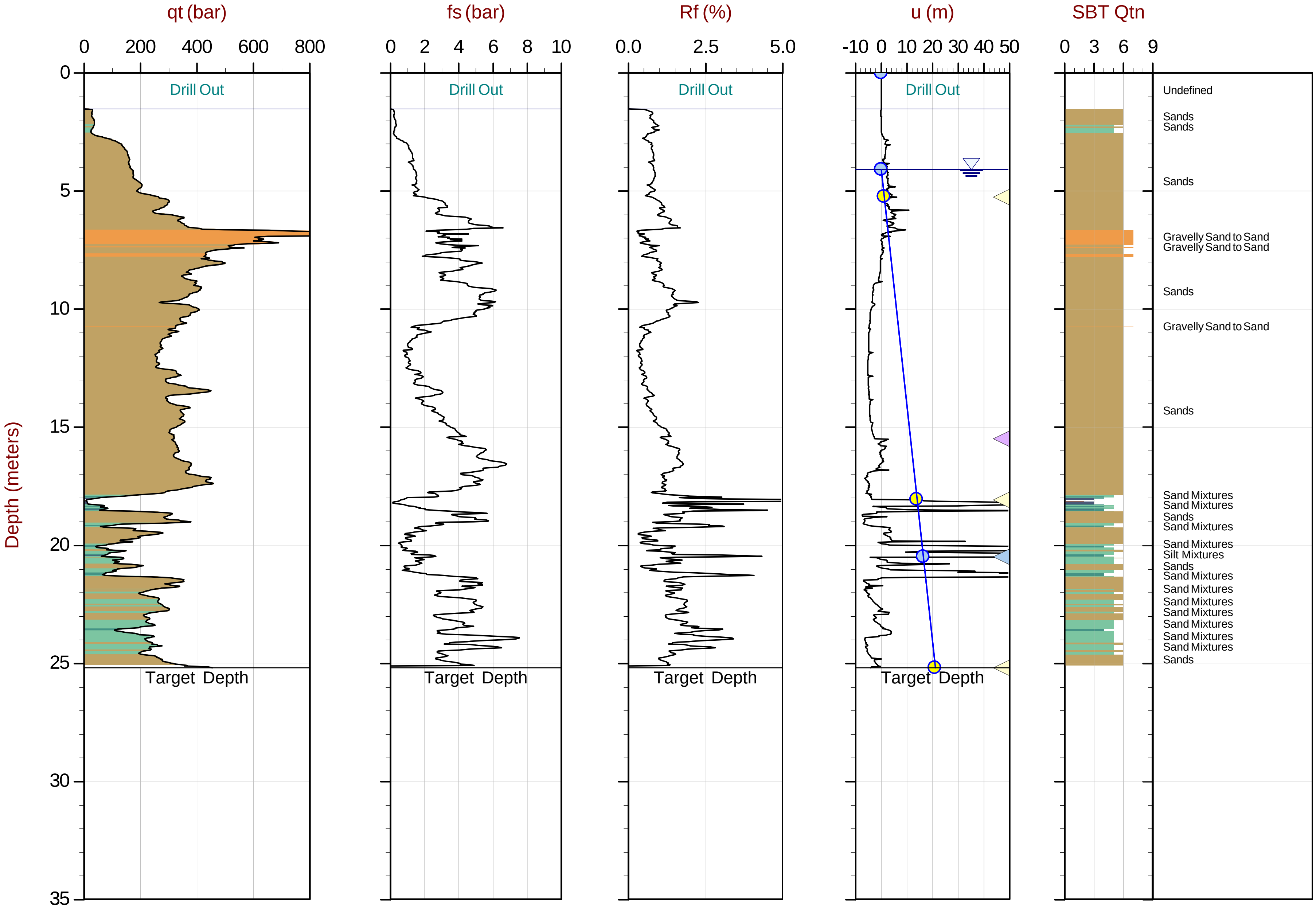
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35



Max Depth: 25.200 m / 82.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914043 E: 603986
Sheet No: 1 of 1

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

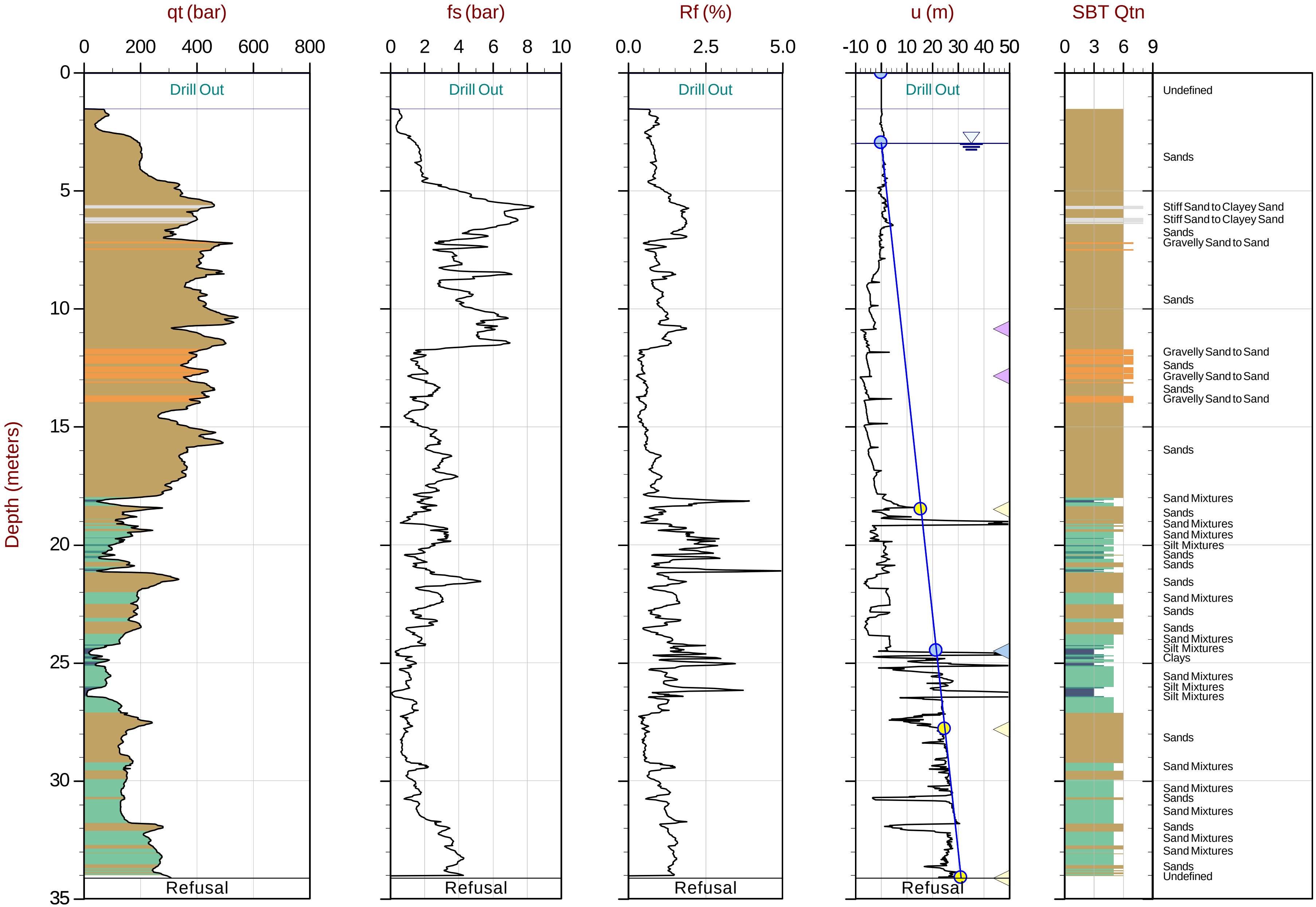
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35



Max Depth: 34.125 m / 111.96 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914045 E: 604058
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.

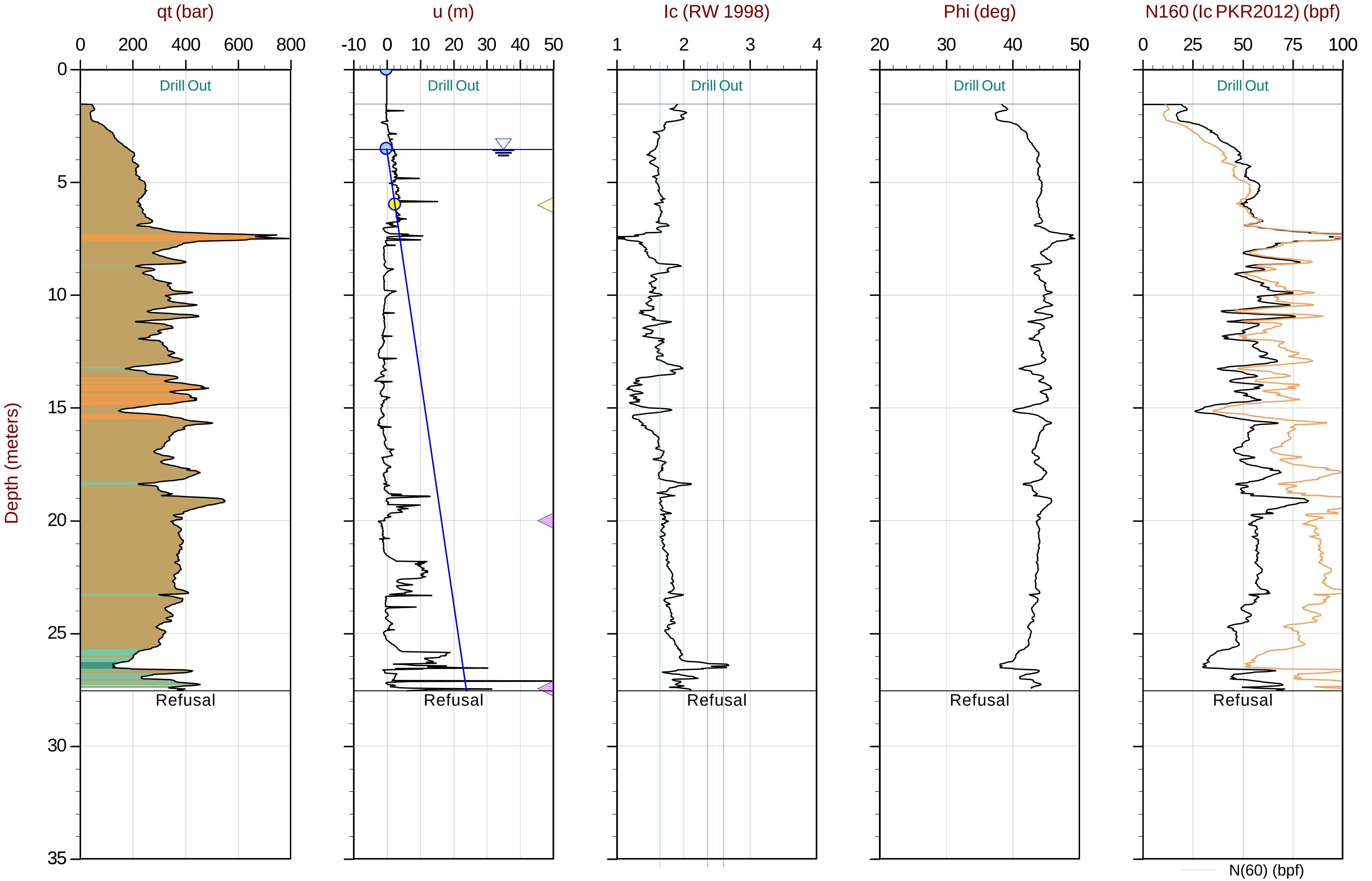
Advanced Cone Penetration Plots with Ic, Phi, and N1(60)lc

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 08:47
Site: 272 Innsfil St.

Sounding: SCPT22-01
Cone: 642:T1500F15U35



Max Depth: 27.550 m / 90.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914109 E: 604028
Sheet No: 1 of 1

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

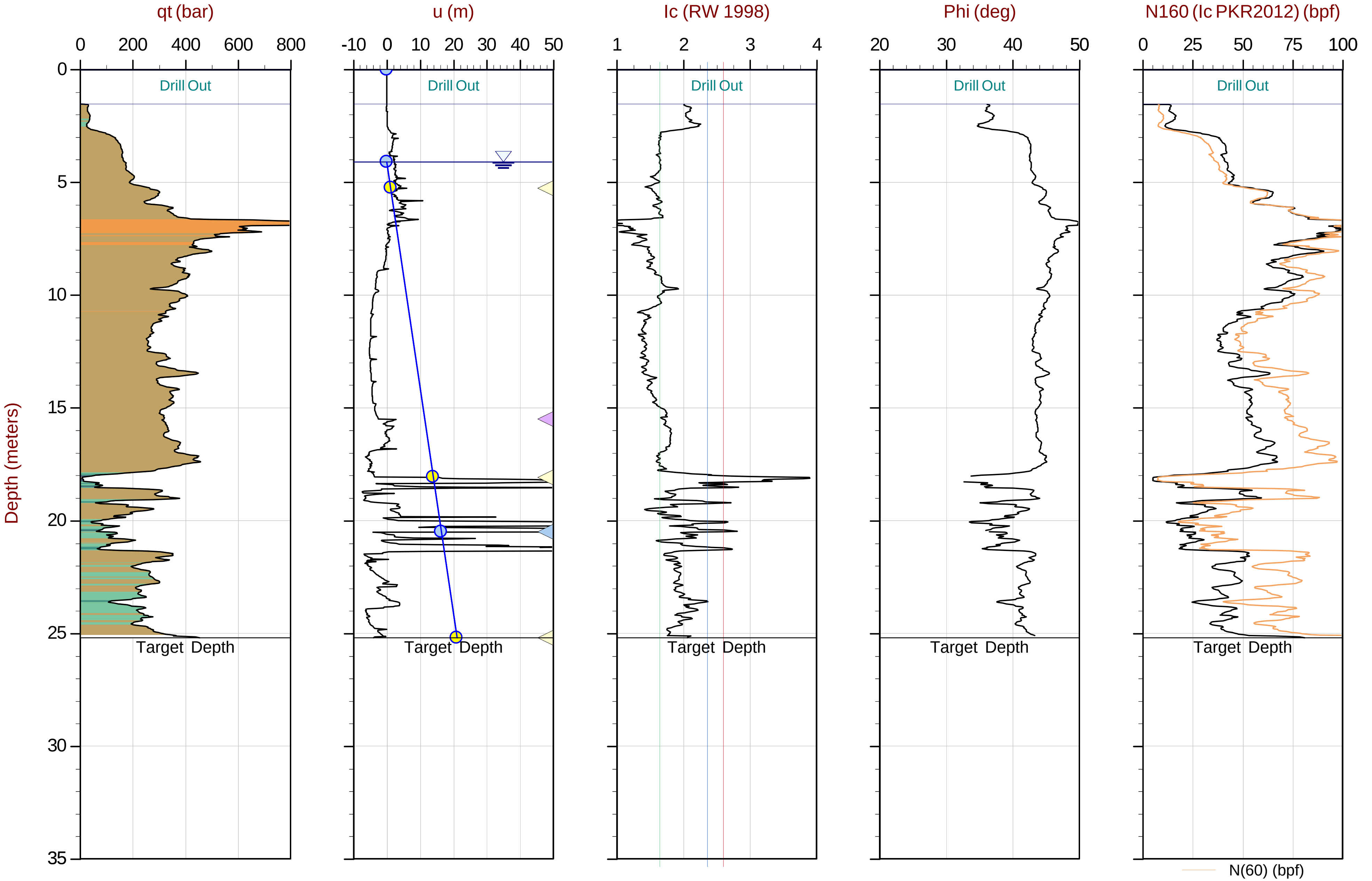
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35



Max Depth: 25.200 m / 82.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914043 E: 603986
Sheet No: 1 of 1

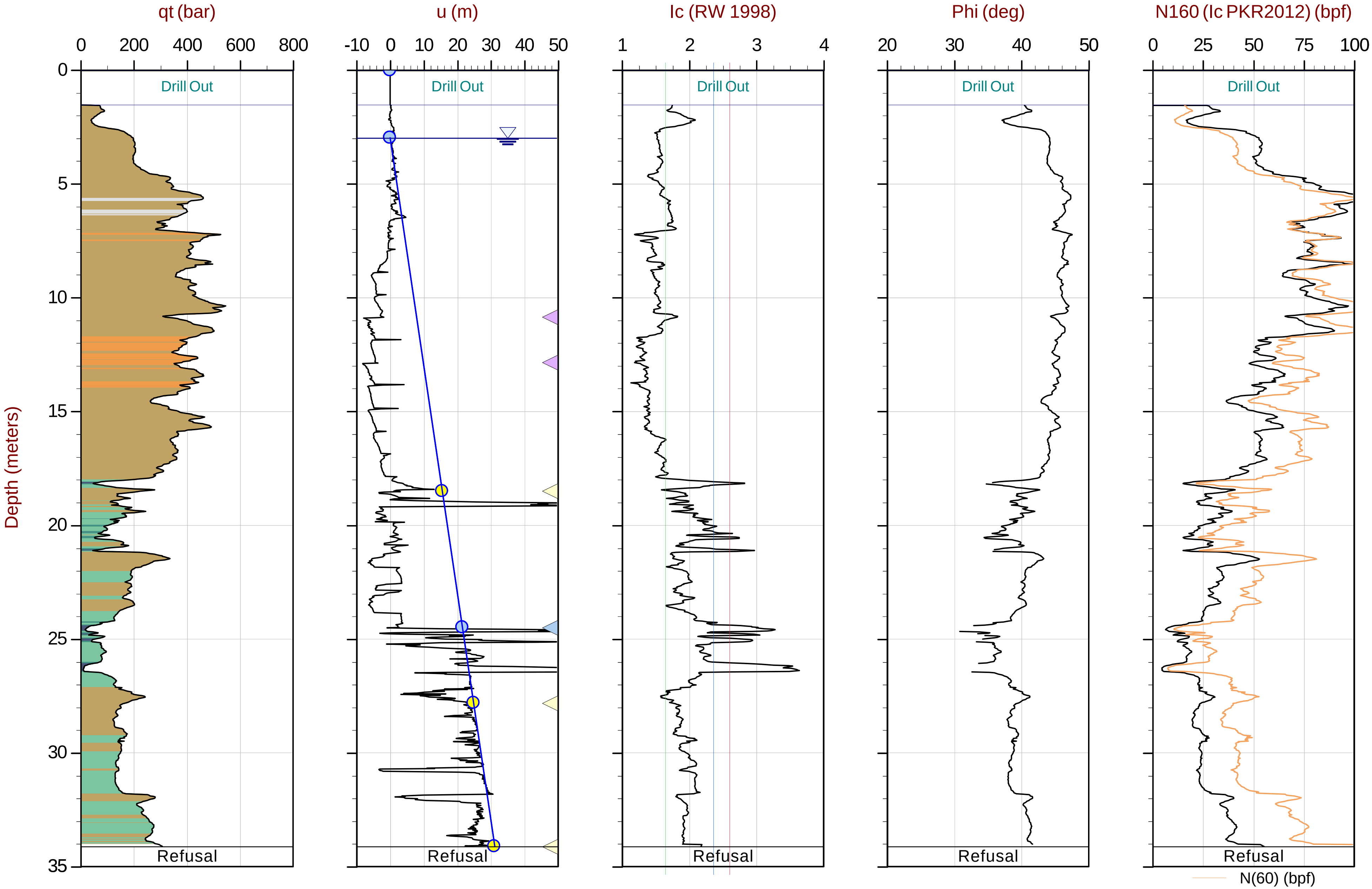
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

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

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35



Max Depth: 34.125 m / 111.96 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914045 E: 604058
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.

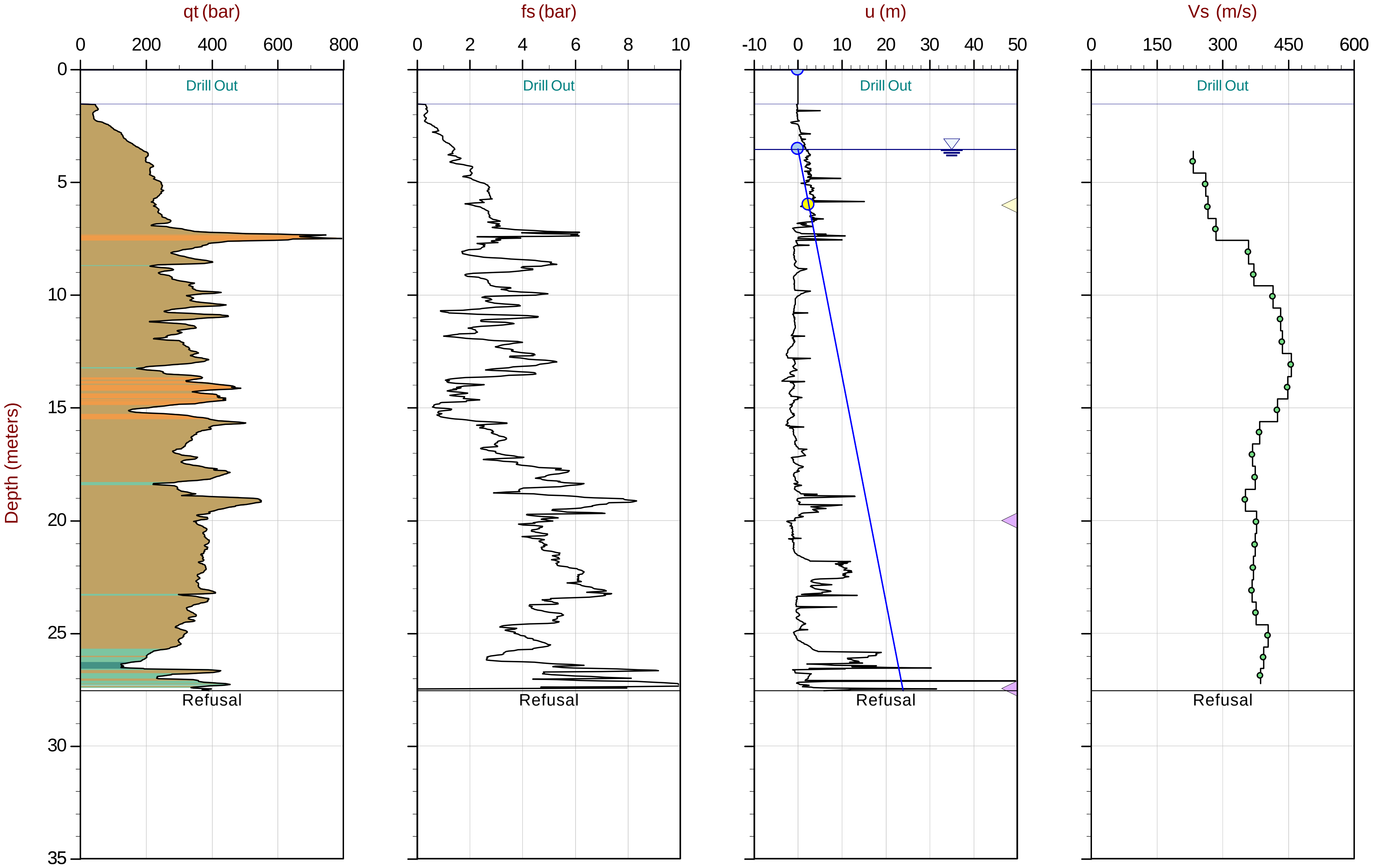
Seismic Cone Penetration Test Plots

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 08:47
Site: 272 Innsfil St.

Sounding: SCPT22-01
Cone: 642:T1500F15U35



Max Depth: 27.550 m / 90.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914109 E: 604028
Sheet No: 1 of 1

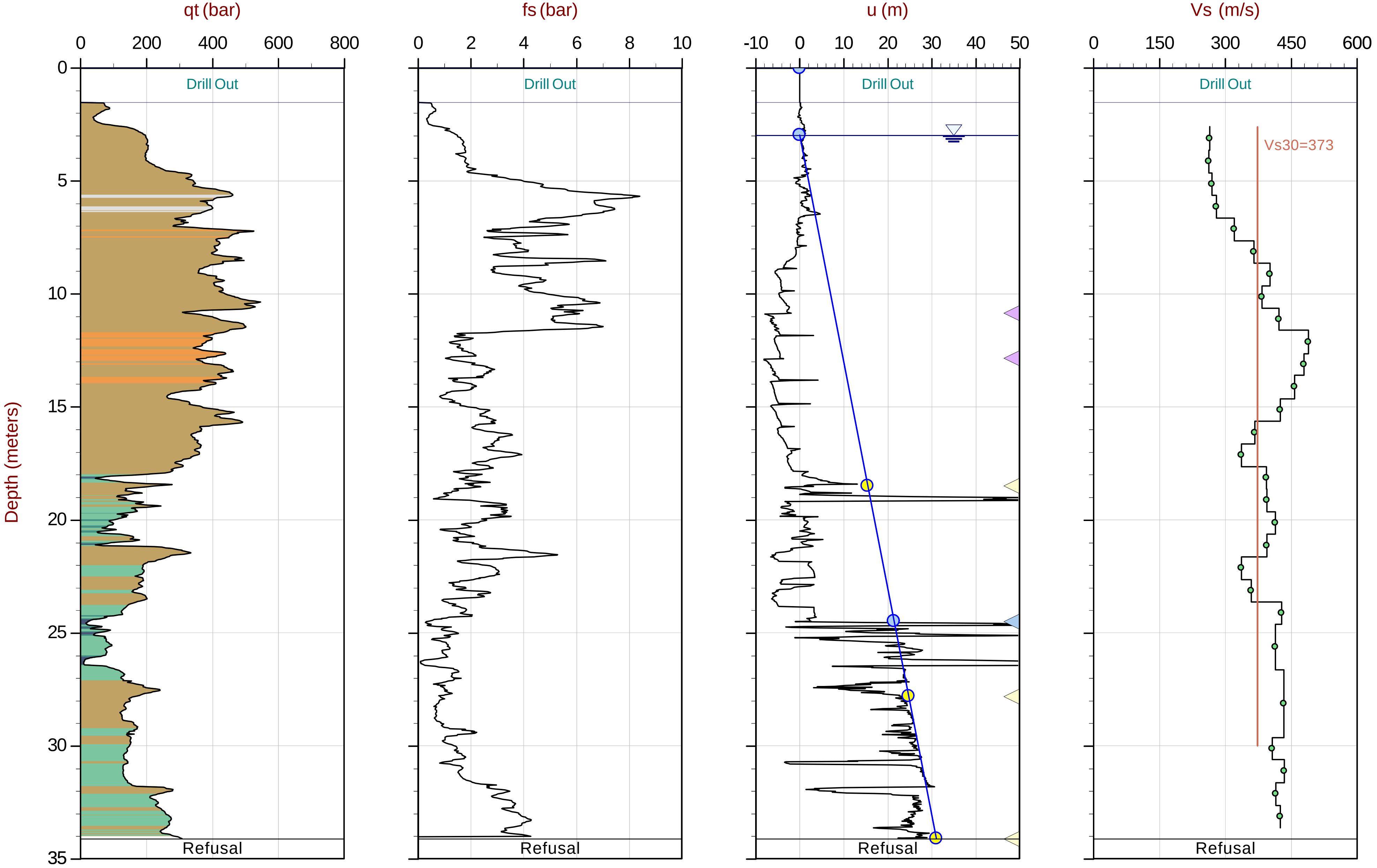
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

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

Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35



Max Depth: 34.125 m / 111.96 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-05-23587_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4914045 E: 604058
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

CONETEC

Job No: 22-05-23587
Client: Cambium Inc
Project: 272 Innisfil St. Barrie
Sounding ID: SCPT22-01
Date: 10-Feb-2022

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)
3.82	3.62	3.76			
4.80	4.60	4.71	0.95	4.07	234
5.82	5.62	5.71	1.00	3.83	262
6.82	6.62	6.70	0.99	3.70	267
7.78	7.58	7.65	0.95	3.34	285
8.83	8.63	8.69	1.04	2.90	359
9.80	9.60	9.65	0.96	2.59	372
10.78	10.58	10.63	0.98	2.35	416
11.80	11.60	11.64	1.02	2.35	433
12.80	12.60	12.64	1.00	2.29	436
13.82	13.62	13.66	1.02	2.22	458
14.82	14.62	14.65	1.00	2.22	449
15.82	15.62	15.65	1.00	2.35	425
16.82	16.62	16.65	1.00	2.59	385
17.80	17.60	17.63	0.98	2.65	369
18.82	18.62	18.65	1.02	2.72	375
19.80	19.60	19.63	0.98	2.78	352
20.78	20.58	20.60	0.98	2.59	378
21.80	21.60	21.62	1.02	2.72	375
22.83	22.63	22.65	1.03	2.78	370
23.83	23.63	23.65	1.00	2.72	368
24.83	24.63	24.65	1.00	2.65	376
25.83	25.63	25.65	1.00	2.47	405
26.78	26.58	26.60	0.95	2.41	394
27.45	27.25	27.27	0.67	1.73	387

CONETEC

Job No: 22-05-23587
Client: Cambium Inc
Project: 272 Innisfil St. Barrie
Sounding ID: SCPT22-03
Date: 10-Feb-2022

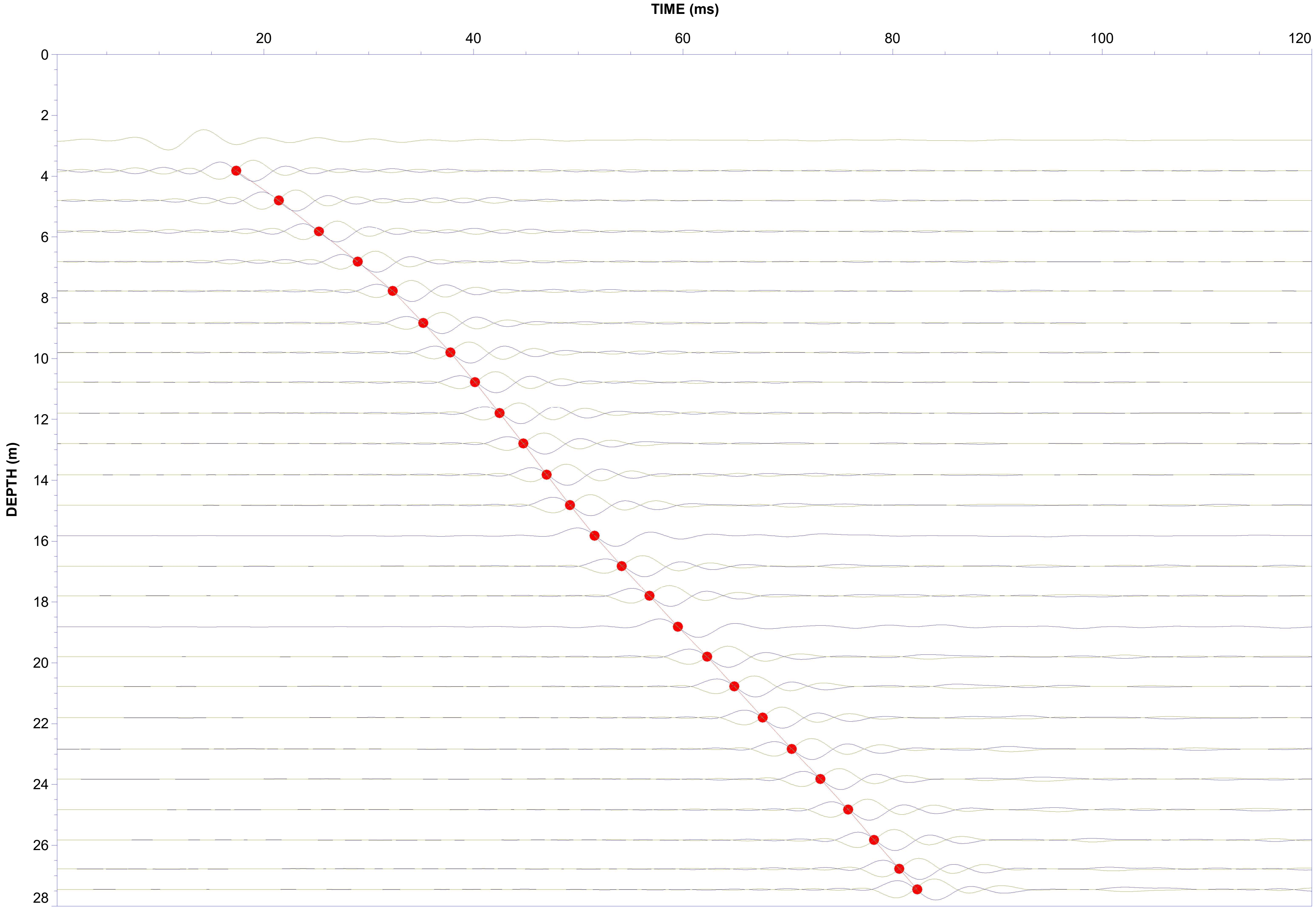
Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

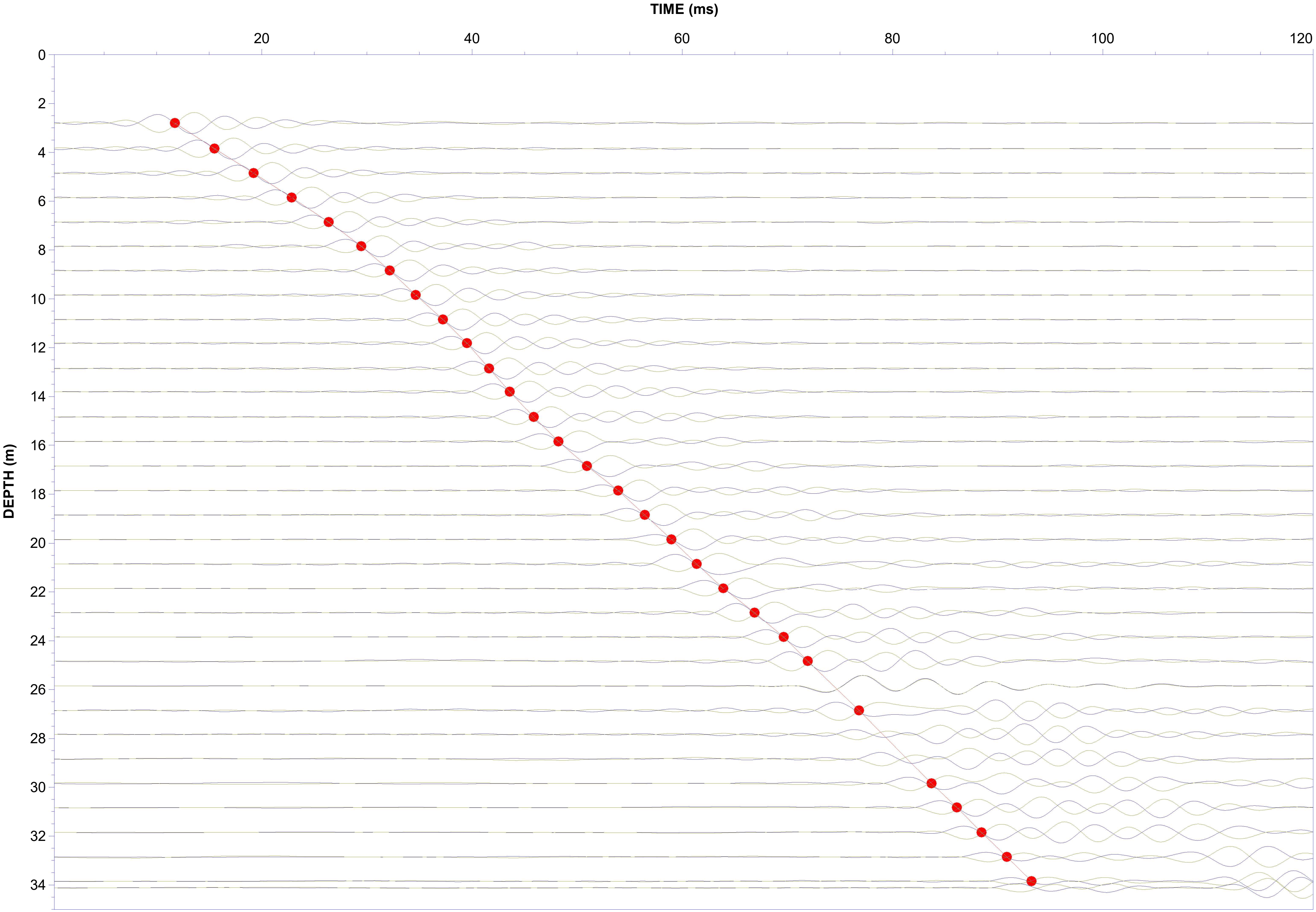
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.80	2.60	2.79			
3.85	3.65	3.79	1.00	3.77	265
4.85	4.65	4.76	0.97	3.70	262
5.85	5.65	5.74	0.98	3.64	270
6.85	6.65	6.73	0.99	3.52	281
7.85	7.65	7.72	0.99	3.09	321
8.85	8.65	8.71	0.99	2.72	366
9.85	9.65	9.70	0.99	2.47	403
10.85	10.65	10.70	1.00	2.59	384
11.82	11.62	11.66	0.97	2.28	423
12.85	12.65	12.69	1.03	2.10	489
13.80	13.60	13.64	0.95	1.98	480
14.85	14.65	14.68	1.05	2.28	459
15.85	15.65	15.68	1.00	2.35	425
16.85	16.65	16.68	1.00	2.72	368
17.85	17.65	17.68	1.00	2.96	337
18.85	18.65	18.68	1.00	2.53	395
19.85	19.65	19.68	1.00	2.53	395
20.85	20.65	20.67	1.00	2.41	415
21.85	21.65	21.67	1.00	2.53	395
22.85	22.65	22.67	1.00	2.96	337
23.85	23.65	23.67	1.00	2.78	360
24.83	24.63	24.65	0.98	2.28	429
26.85	26.65	26.67	2.02	4.88	414
29.85	29.65	29.67	3.00	6.91	434
30.83	30.63	30.65	0.98	2.41	407
31.85	31.65	31.67	1.02	2.35	435
32.85	32.65	32.67	1.00	2.41	415
33.85	33.65	33.67	1.00	2.35	426

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 22-05-23587	Client: Cambium Inc	Project: 272 Innisfil St. Barrie	Filtering: 0-200 Hz	Sounding: SCPT22-01	Site: 272 Innsfil St
Date: 10-Feb-2022	Cone: 642:T1500F15U35				



Job No: 22-05-23587	Client: Cambium Inc	Project: 272 Innisfil St. Barrie	Filtering: 0-200 Hz	Sounding: SCPT22-03	Site: 272 Innsfil St
Date: 10-Feb-2022	Cone: 642:T1500F15U35				



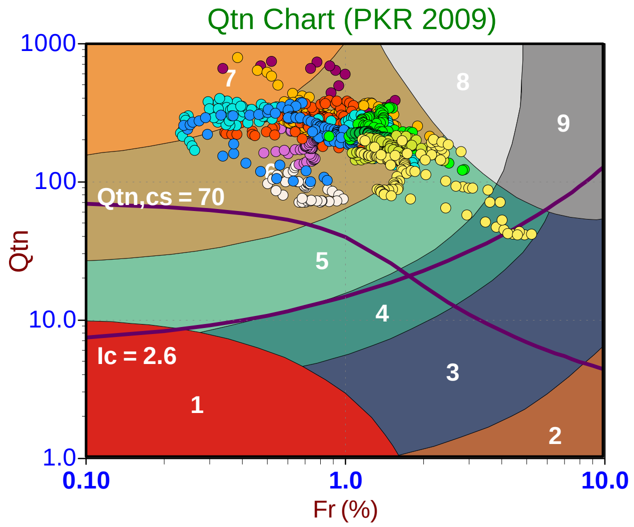
Soil Behaviour Type (SBT) Scatter Plots

CONETEC

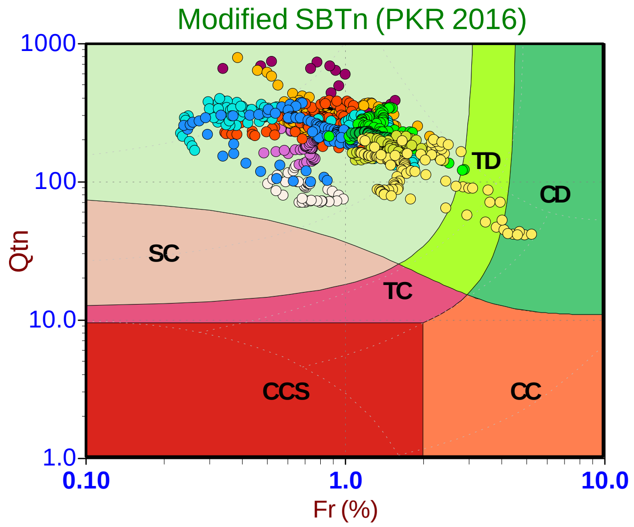
Cambium Inc

Job No: 22-05-23587
Date: 2022-02-10 08:47
Site: 272 Innsfil St.

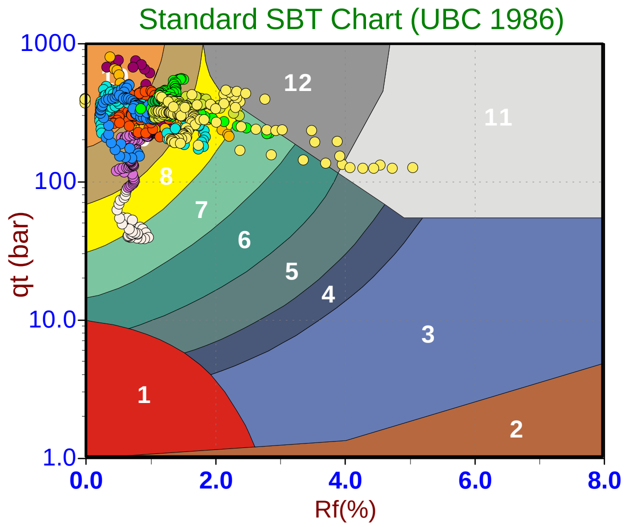
Sounding: SCPT22-01
Cone: 642:T1500F15U35



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

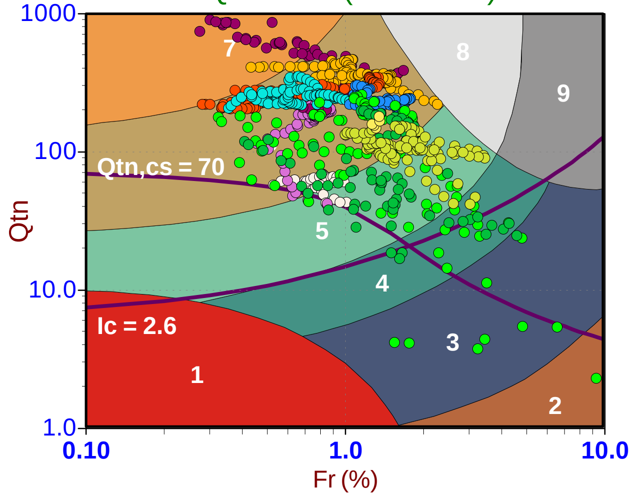
CONETEC

Cambium Inc

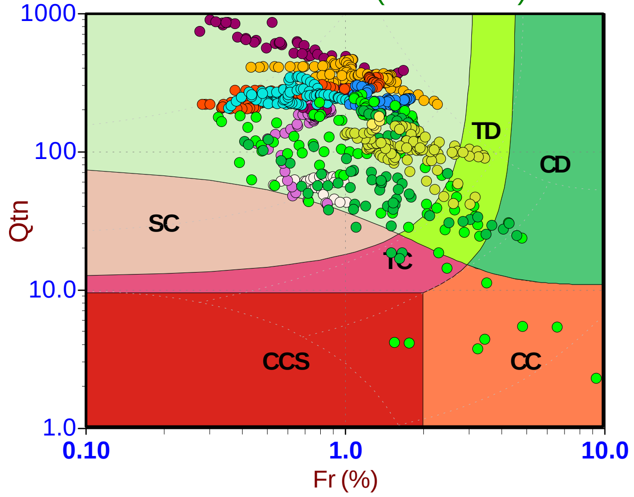
Job No: 22-05-23587
Date: 2022-02-10 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35

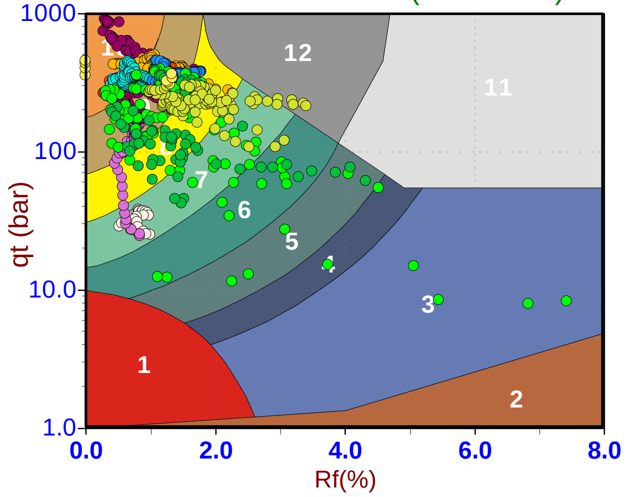
Qtn Chart (PKR 2009)



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

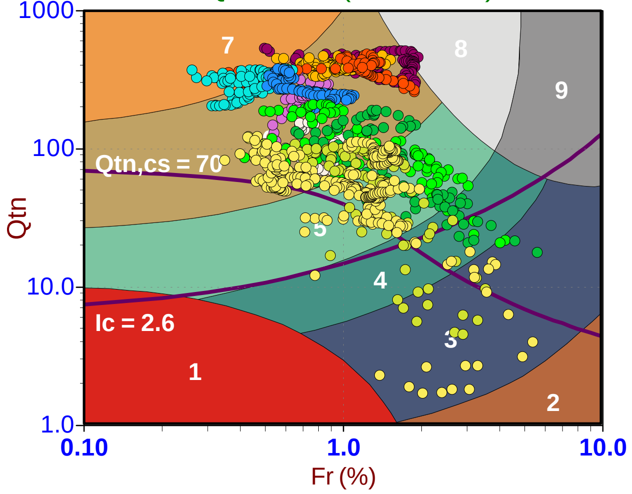
CONETEC

Cambium Inc

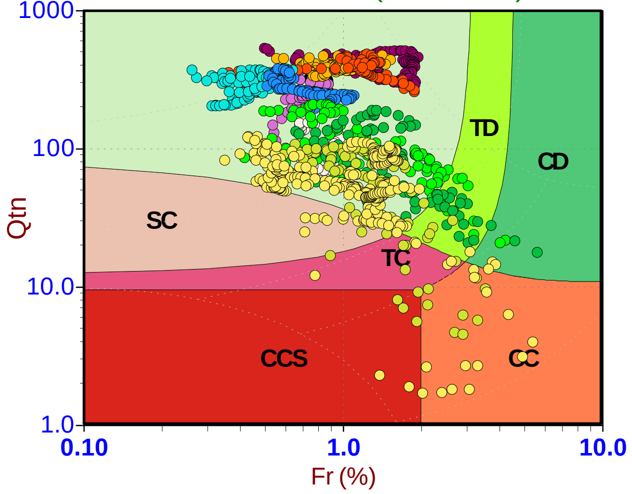
Job No: 22-05-23587
Date: 2022-02-10 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35

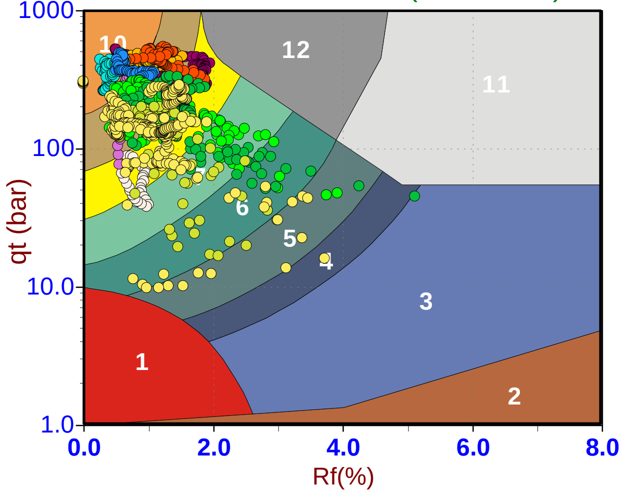
Qtn Chart (PKR 2009)



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

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

CONETEC

Job No:	22-05-23587
Client:	Cambium Inc
Project:	272 Innisfil St. Barrie
Start Date:	10-Feb-2022
End Date:	10-Feb-2022

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t_{50}[a] (s)	Assumed Rigidity Index (I_r)	c_h[b] (cm^2/min)
SCPT22-01	22-05-23587_SP01	15	600	6.000	2.5	3.5				
SCPT22-01	22-05-23587_SP01	15	600	20.000						
SCPT22-01	22-05-23587_SP01	15	900	27.450						
CPT22-02	22-05-23587_CP02	15	400	5.250	1.2	4.1				
CPT22-02	22-05-23587_CP02	15	1000	15.500						
CPT22-02	22-05-23587_CP02	15	400	18.075	13.9	4.1				
CPT22-02	22-05-23587_CP02	15	300	20.500	16.4		4.1	24	100	29.6
CPT22-02	22-05-23587_CP02	15	850	25.200	21.1	4.1				
SCPT22-03	22-05-23587_SP03	15	750	10.850						
SCPT22-03	22-05-23587_SP03	15	300	12.850						
SCPT22-03	22-05-23587_SP03	15	900	18.500	15.5	3.0				
SCPT22-03	22-05-23587_SP03	15	800	24.500	21.5		3.0	454	100	1.5
SCPT22-03	22-05-23587_SP03	15	555	27.825	24.8	3.0				
SCPT22-03	22-05-23587_SP03	15	680	34.125	31.1	3.0				

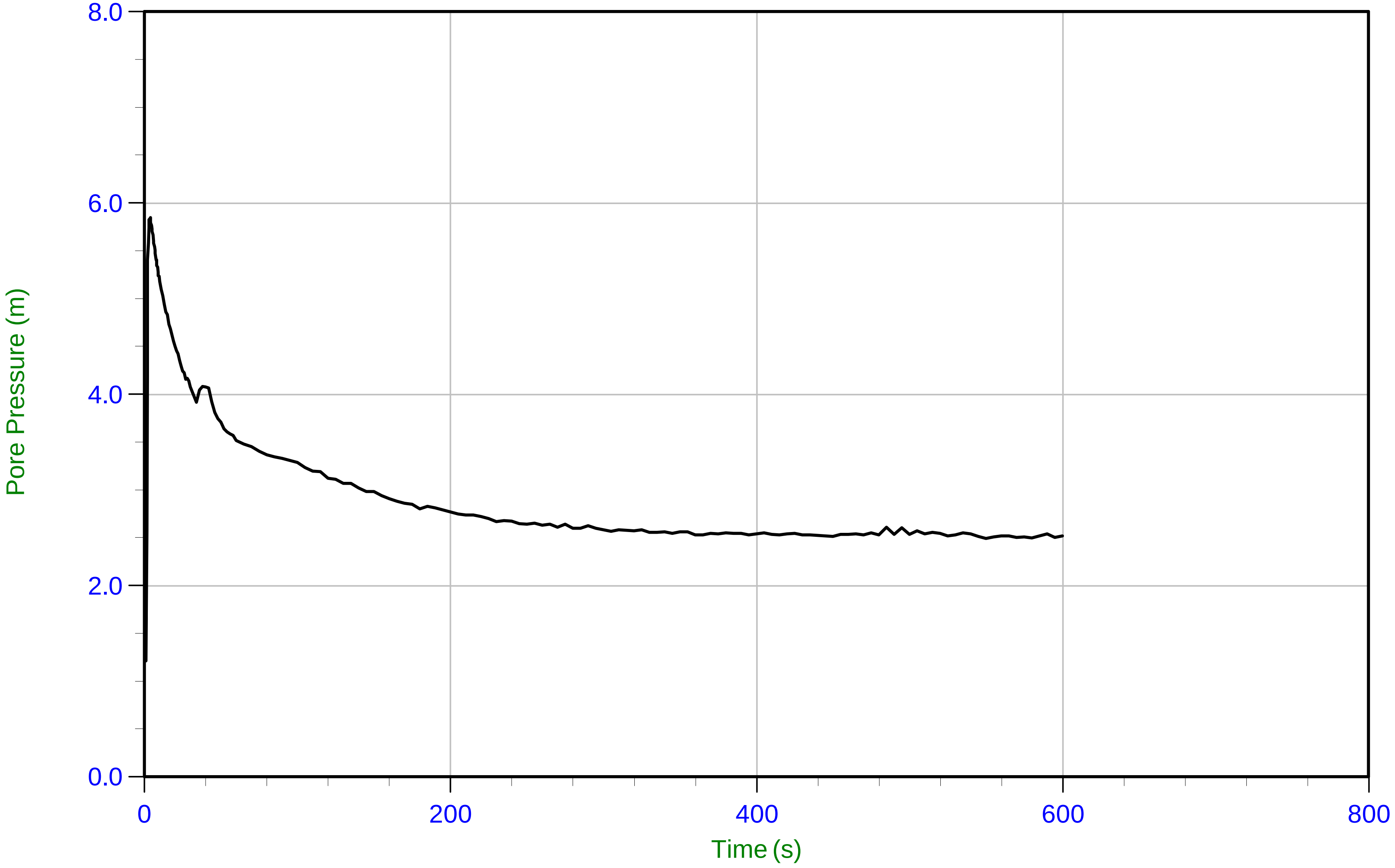
a. Time is relative to where umax occurred.

b. Houlsby and Teh, 1991.

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 08:47
Site: 272 Innsfil St.

Sounding: SCPT22-01
Cone: 642:T1500F15U35 Area=15 cm²

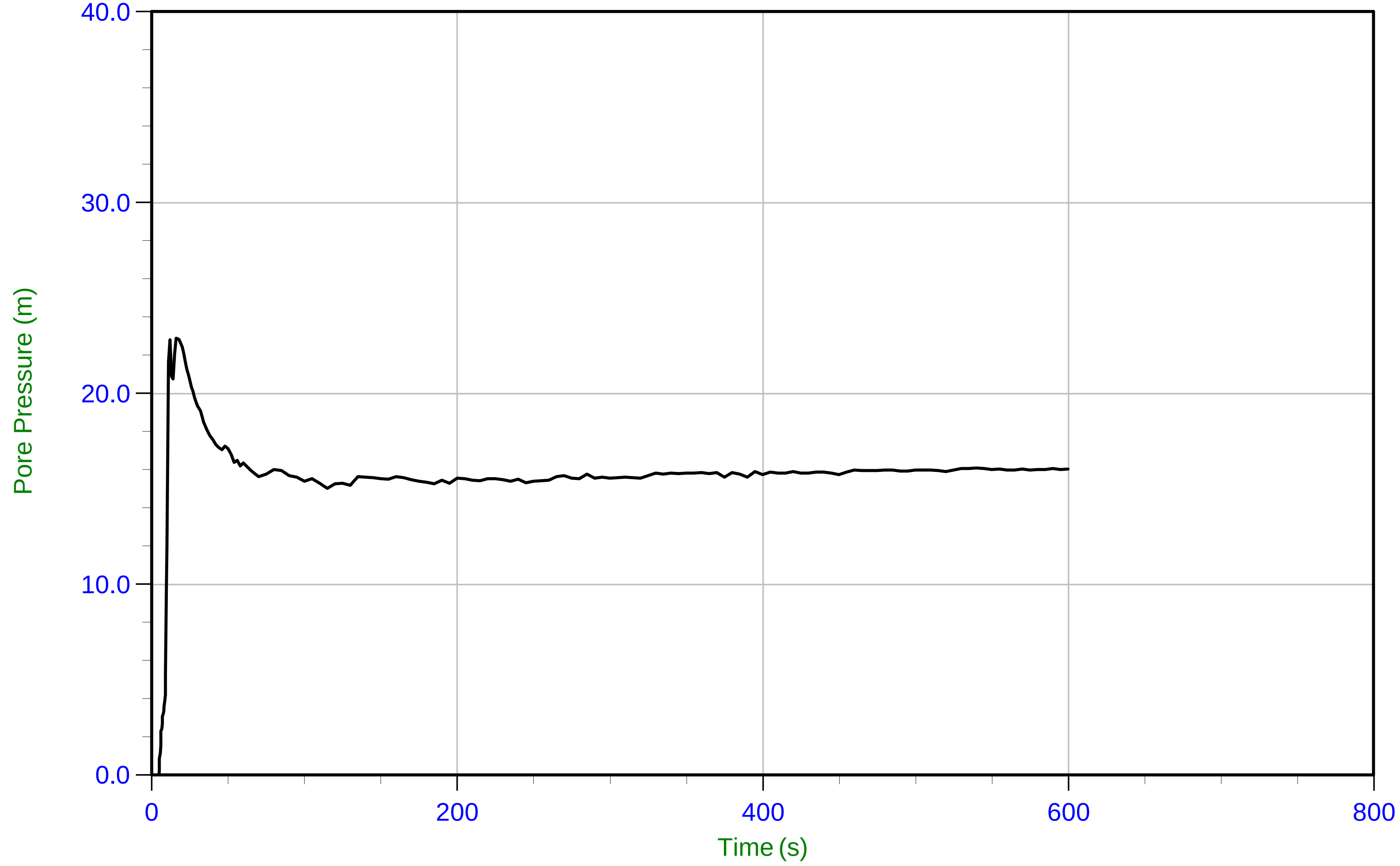


Trace Summary:

Filename: 22-05-23587_SP01.PPF
Depth: 6.000 m / 19.685 ft
Duration: 600.0 s

u Min: 1.2 m
u Max: 5.9 m
u Final: 2.5 m

WT: 3.537 m / 11.604 ft
Ueq: 2.5 m



Trace Summary:

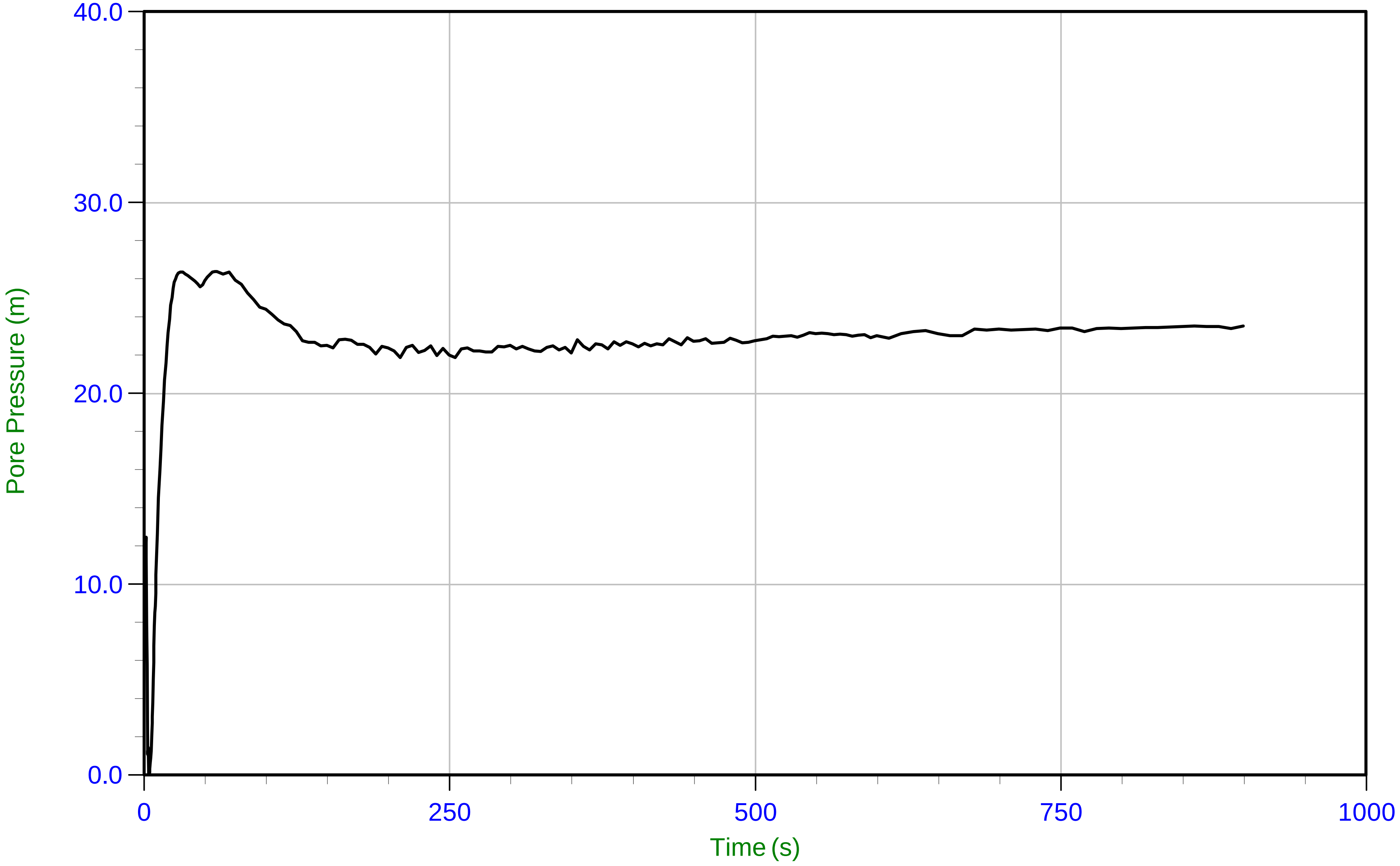
Filename: 22-05-23587_SP01.PPF
Depth: 20.000 m / 65.616 ft
Duration: 600.0 s

u Min: -1.2 m
u Max: 22.9 m
u Final: 16.0 m

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 08:47
Site: 272 Innsfil St.

Sounding: SCPT22-01
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_SP01.PPF
Depth: 27.450 m / 90.058 ft
Duration: 900.0 s

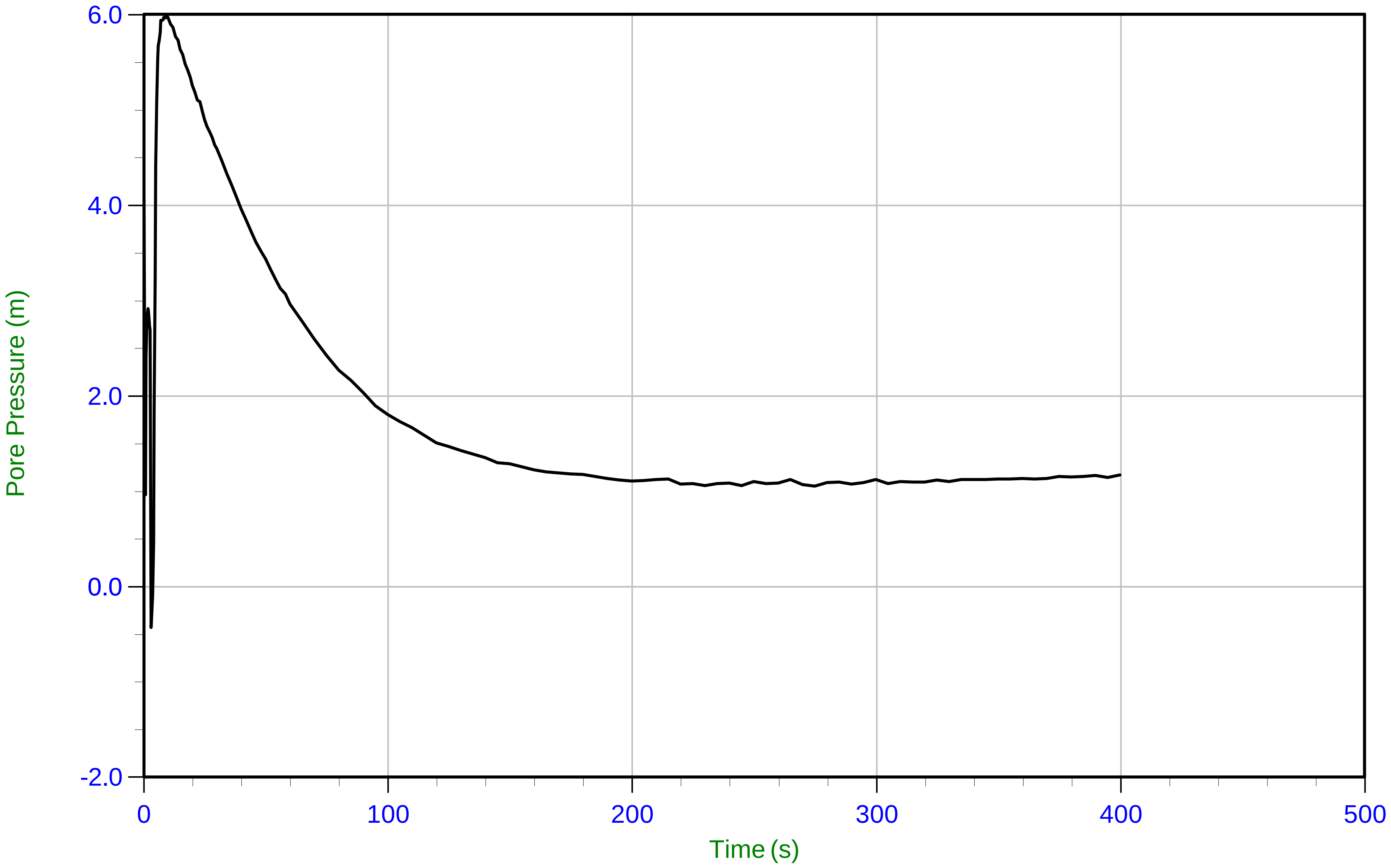
u Min: -0.0 m
u Max: 26.4 m
u Final: 23.5 m

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_CP02.PPF
Depth: 5.250 m / 17.224 ft
Duration: 400.0 s

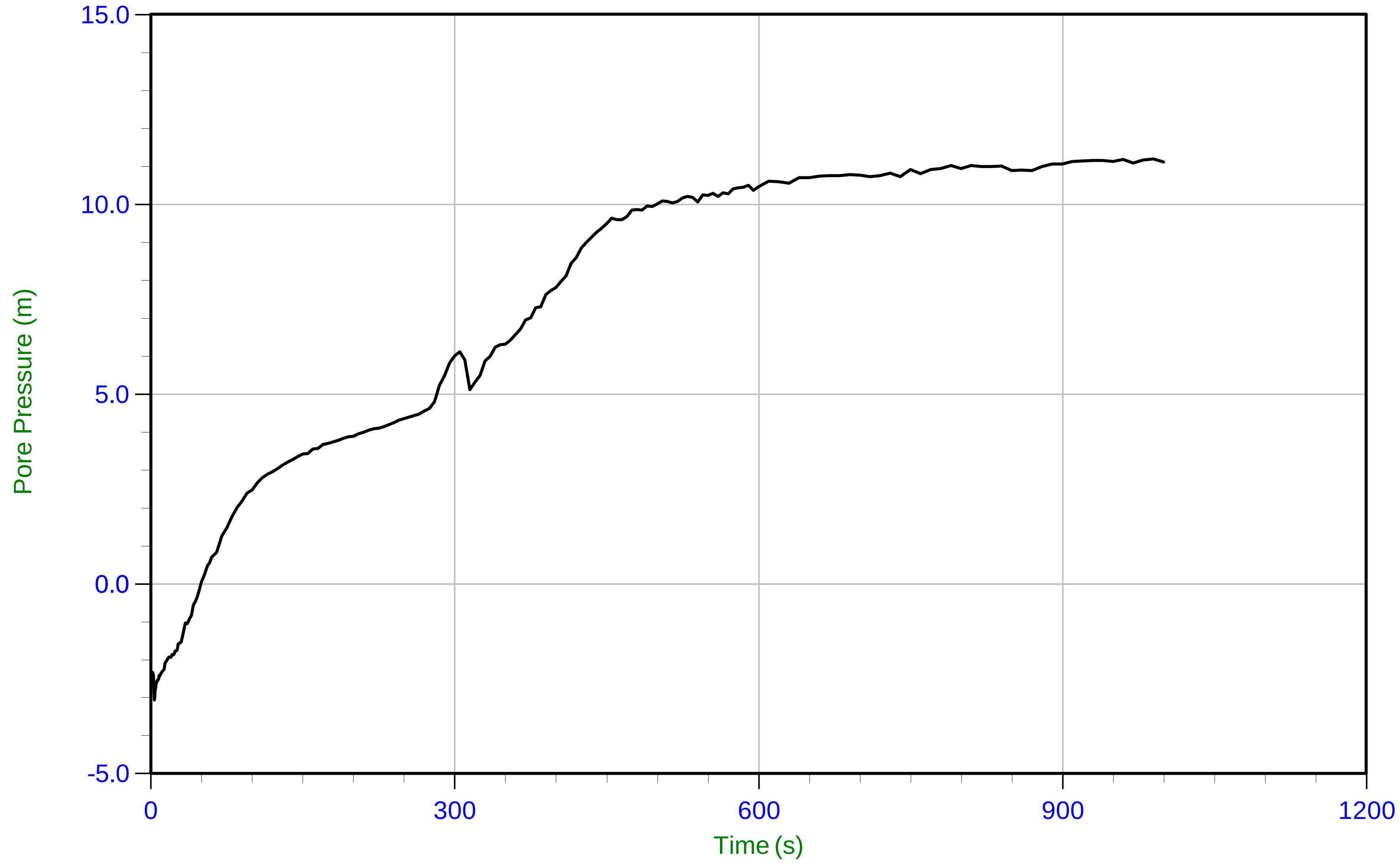
u Min: -0.4 m
u Max: 6.0 m
u Final: 1.2 m

WT: 4.092 m / 13.425 ft
Ueq: 1.2 m

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_CP02.PPF
Depth: 15.500 m / 50.852 ft
Duration: 1000.0 s

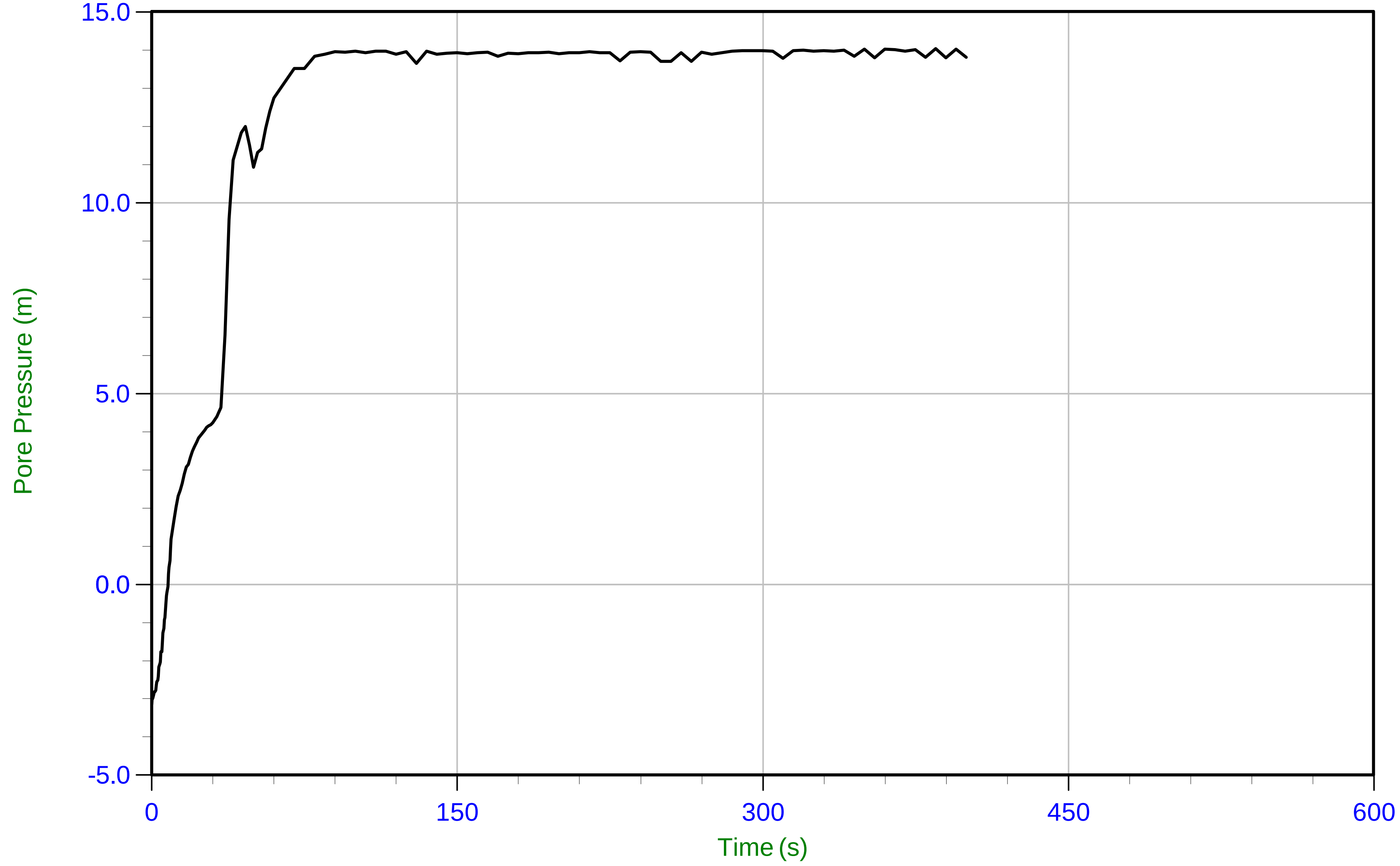
u Min: -3.1 m
u Max: 11.2 m
u Final: 11.1 m

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_CP02.PPF
Depth: 18.075 m / 59.300 ft
Duration: 400.0 s

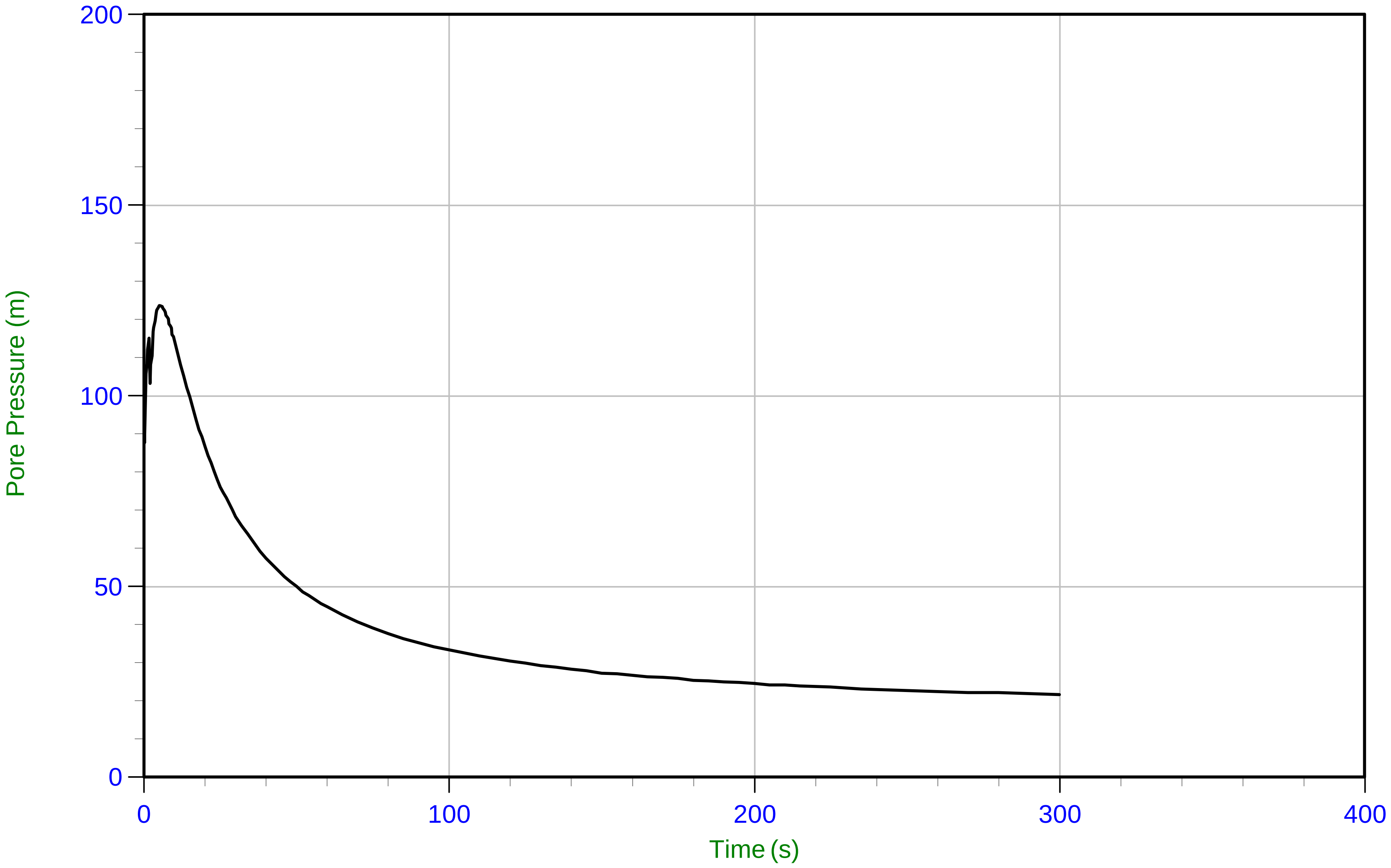
u Min: -3.2 m
u Max: 14.0 m
u Final: 13.8 m

WT: 4.128 m / 13.542 ft
Ueq: 13.9 m

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_CP02.PPF
Depth: 20.500 m / 67.256 ft
Duration: 300.0 s

u Min: 21.7 m
u Max: 123.7 m
u Final: 21.7 m

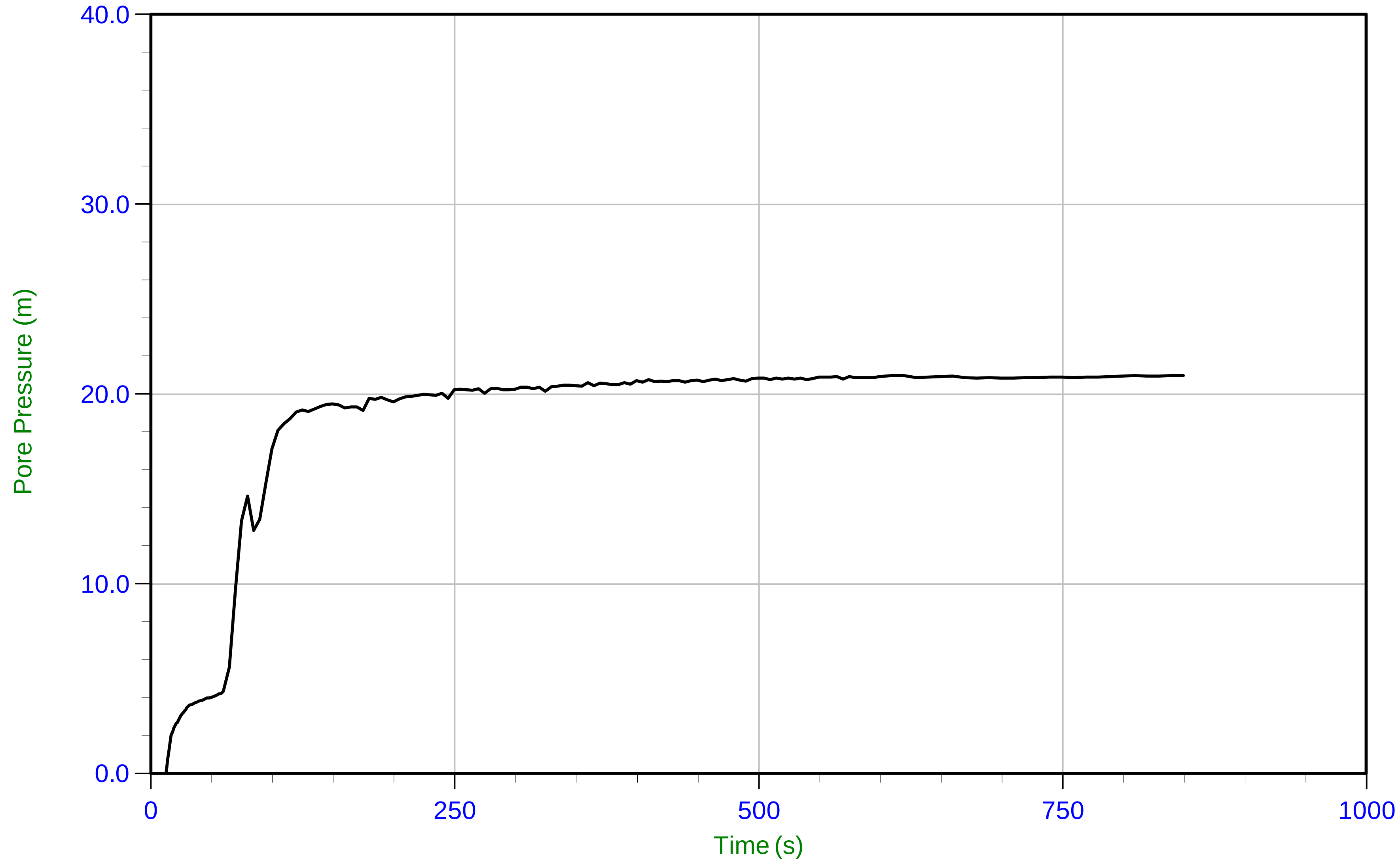
WT: 4.128 m / 13.543 ft
Ueq: 16.4 m
U(50): 70.01 m

T(50): 23.7 s
Ir: 100
Ch: 29.6 cm²/min

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 11:32
Site: 272 Innsfil St.

Sounding: CPT22-02
Cone: 642:T1500F15U35 Area=15 cm²

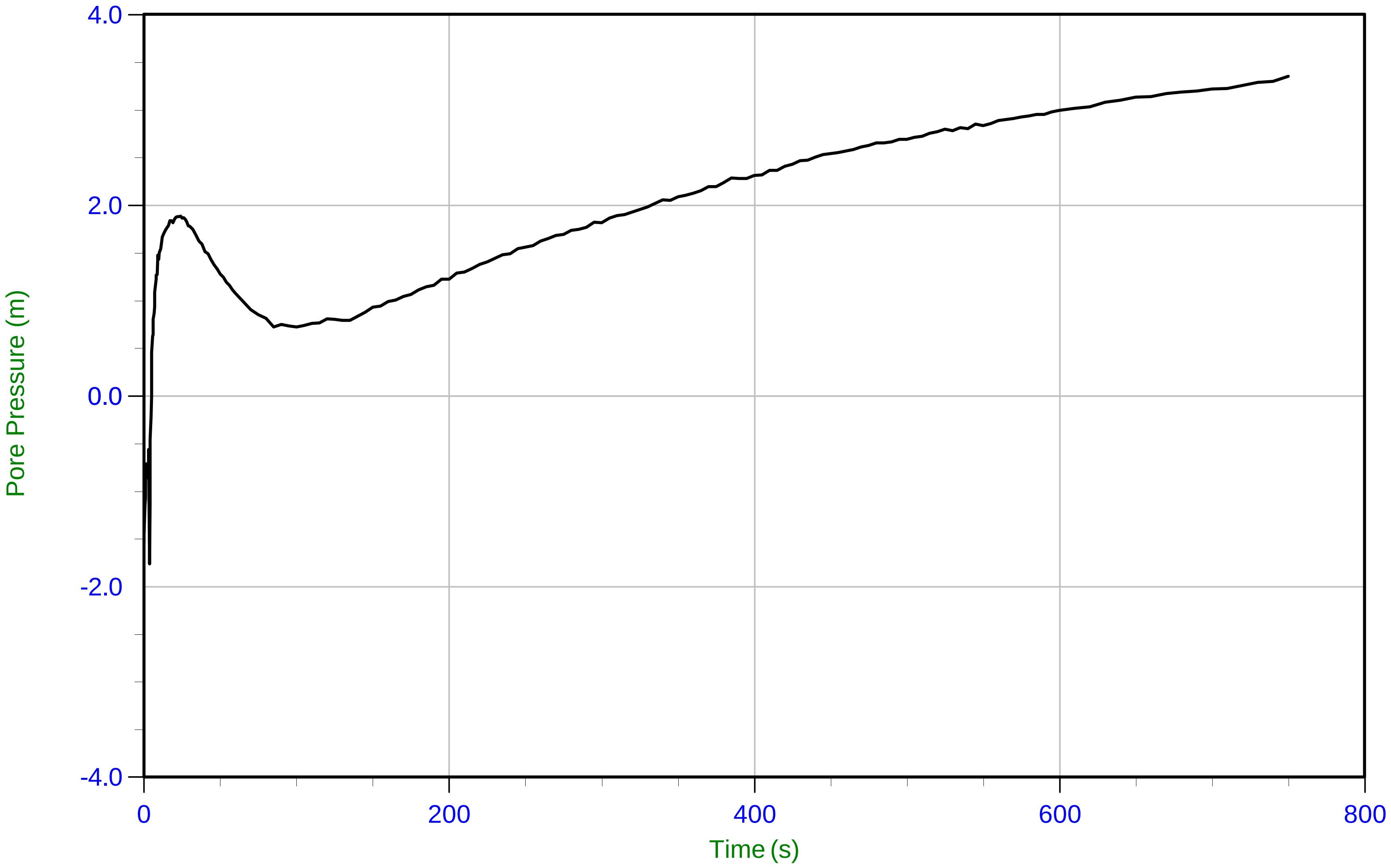


Trace Summary:

Filename: 22-05-23587_CP02.PPF
Depth: 25.200 m / 82.676 ft
Duration: 850.0 s

u Min: -5.0 m
u Max: 21.0 m
u Final: 21.0 m

WT: 4.147 m / 13.607 ft
Ueq: 21.1 m



Trace Summary:

Filename: 22-05-23587_SP03.PPF
Depth: 10.850 m / 35.597 ft
Duration: 750.0 s

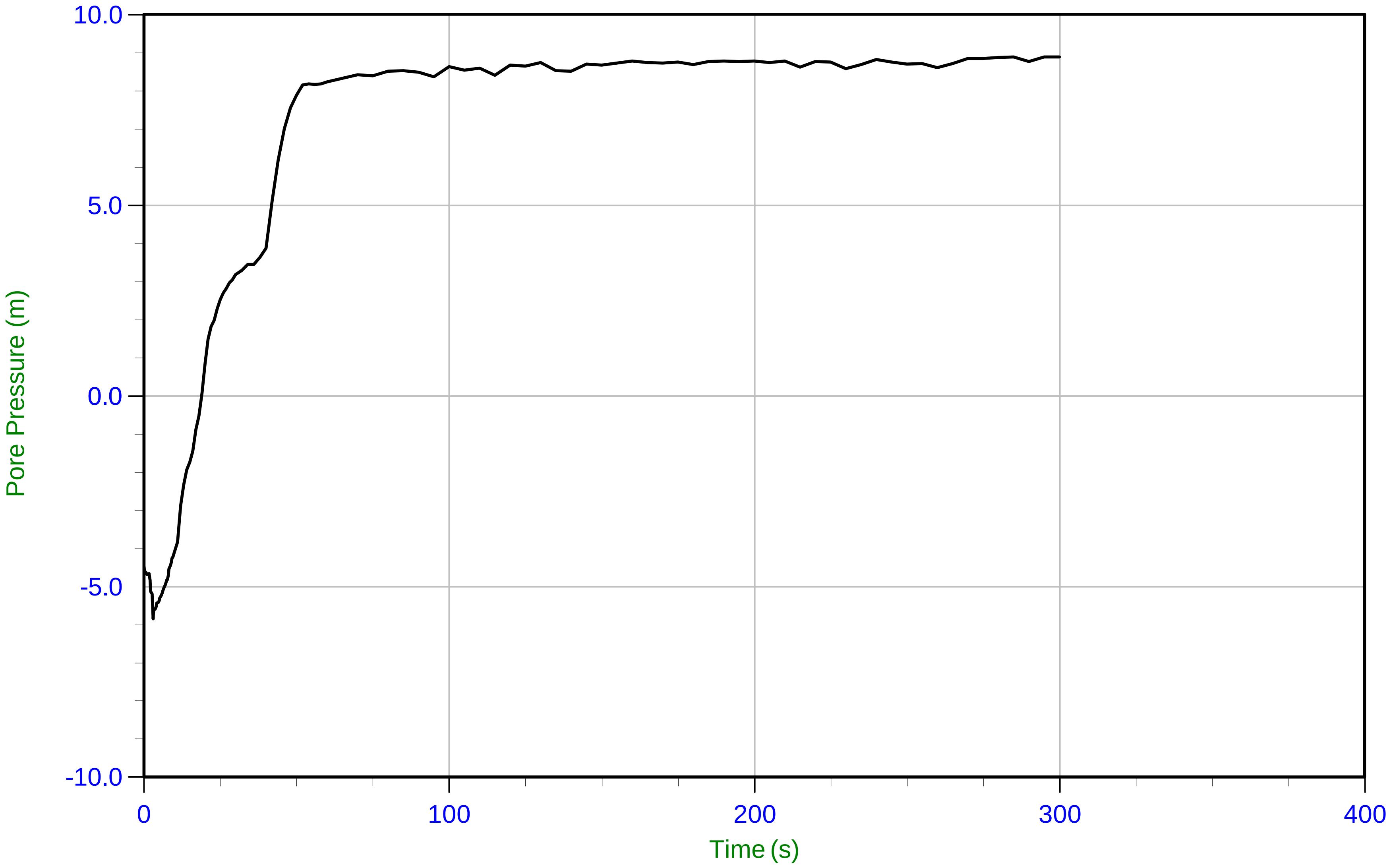
u Min: -1.8 m
u Max: 3.3 m
u Final: 3.3 m

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

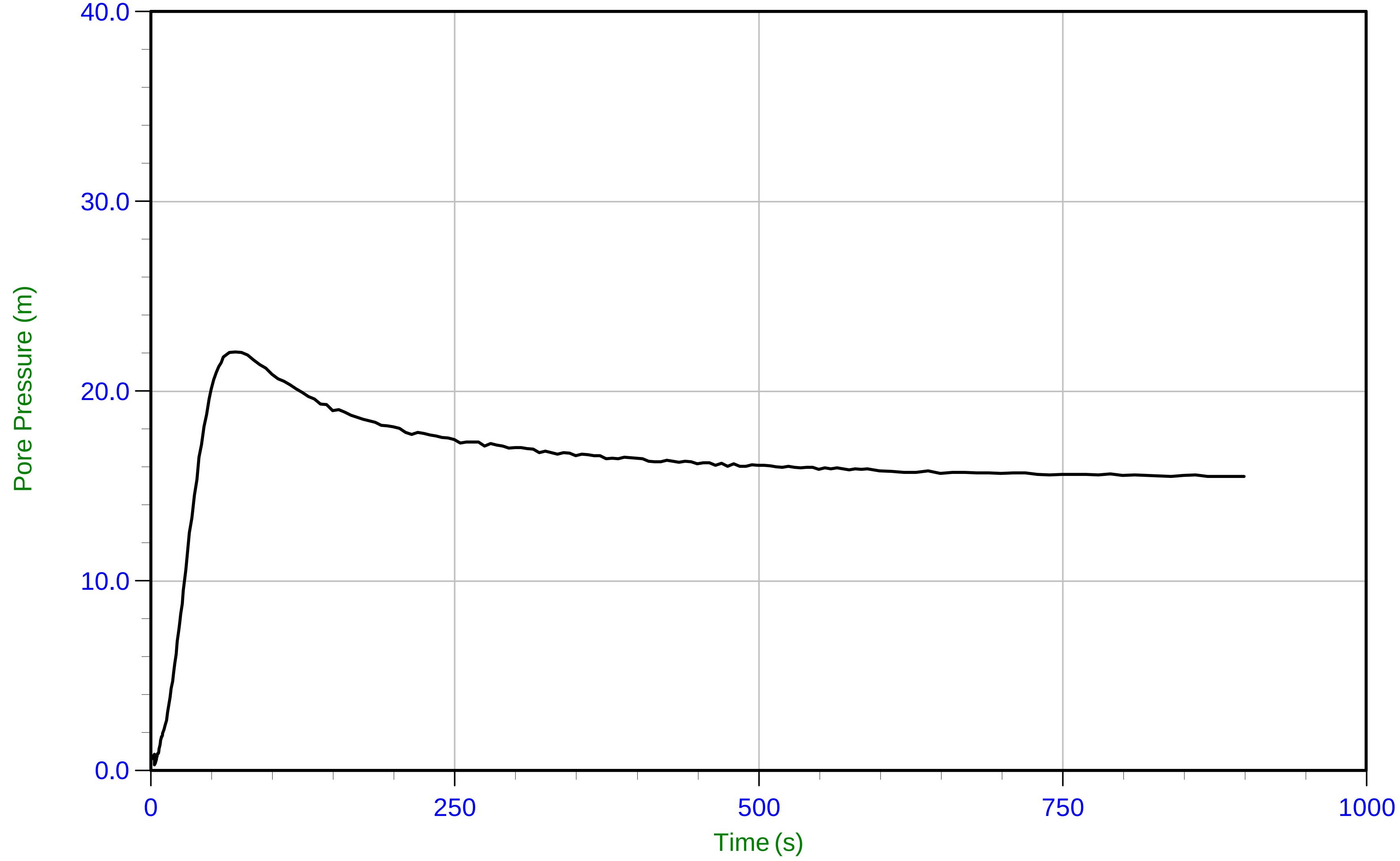
Filename: 22-05-23587_SP03.PPF
Depth: 12.850 m / 42.158 ft
Duration: 300.0 s

u Min: -5.8 m
u Max: 8.9 m
u Final: 8.9 m

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_SP03.PPF
Depth: 18.500 m / 60.695 ft
Duration: 900.0 s

u Min: 0.3 m
u Max: 22.1 m
u Final: 15.5 m

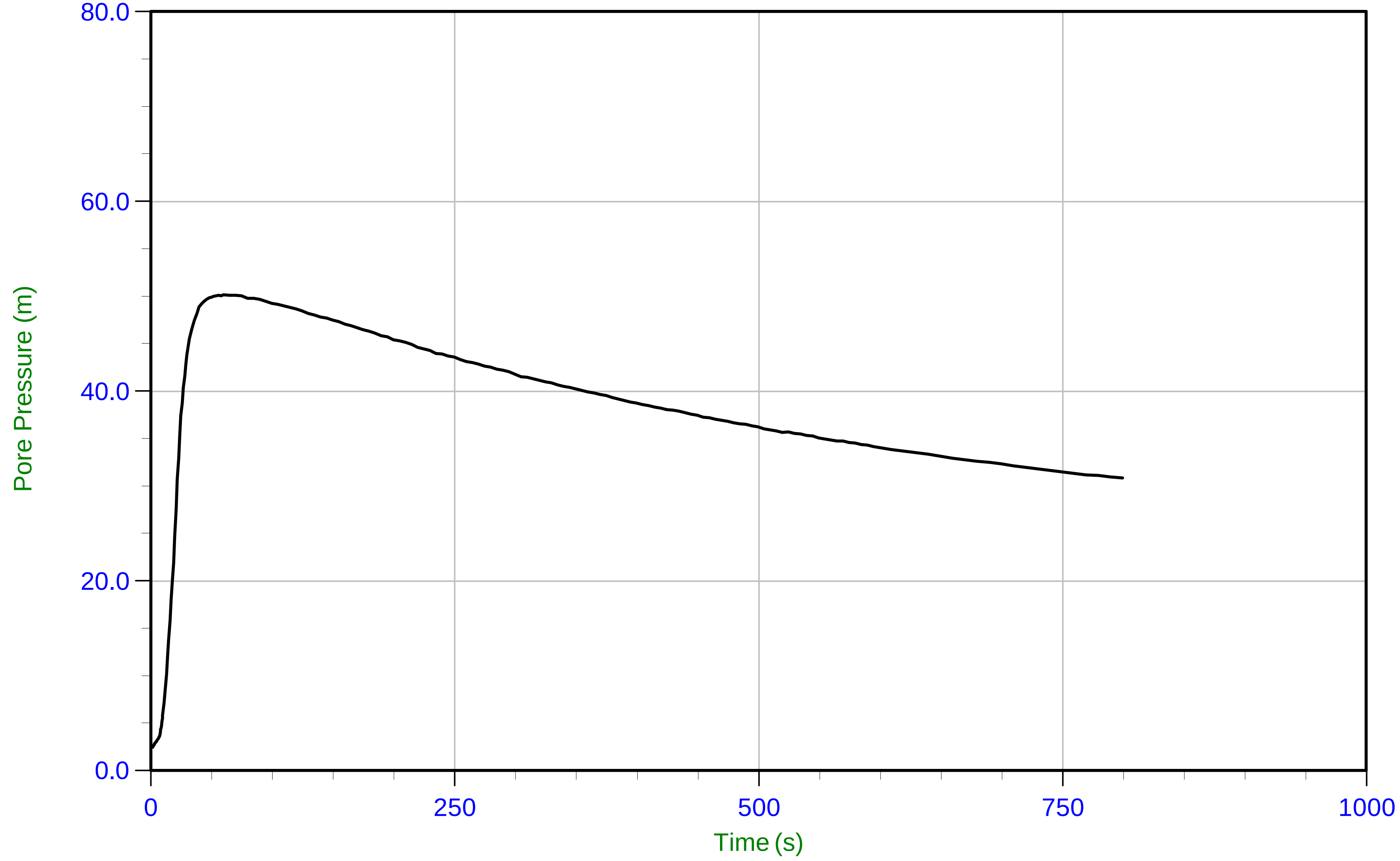
WT: 2.979 m / 9.772 ft
Ueq: 15.5 m

CONETEC

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35 Area=15 cm²



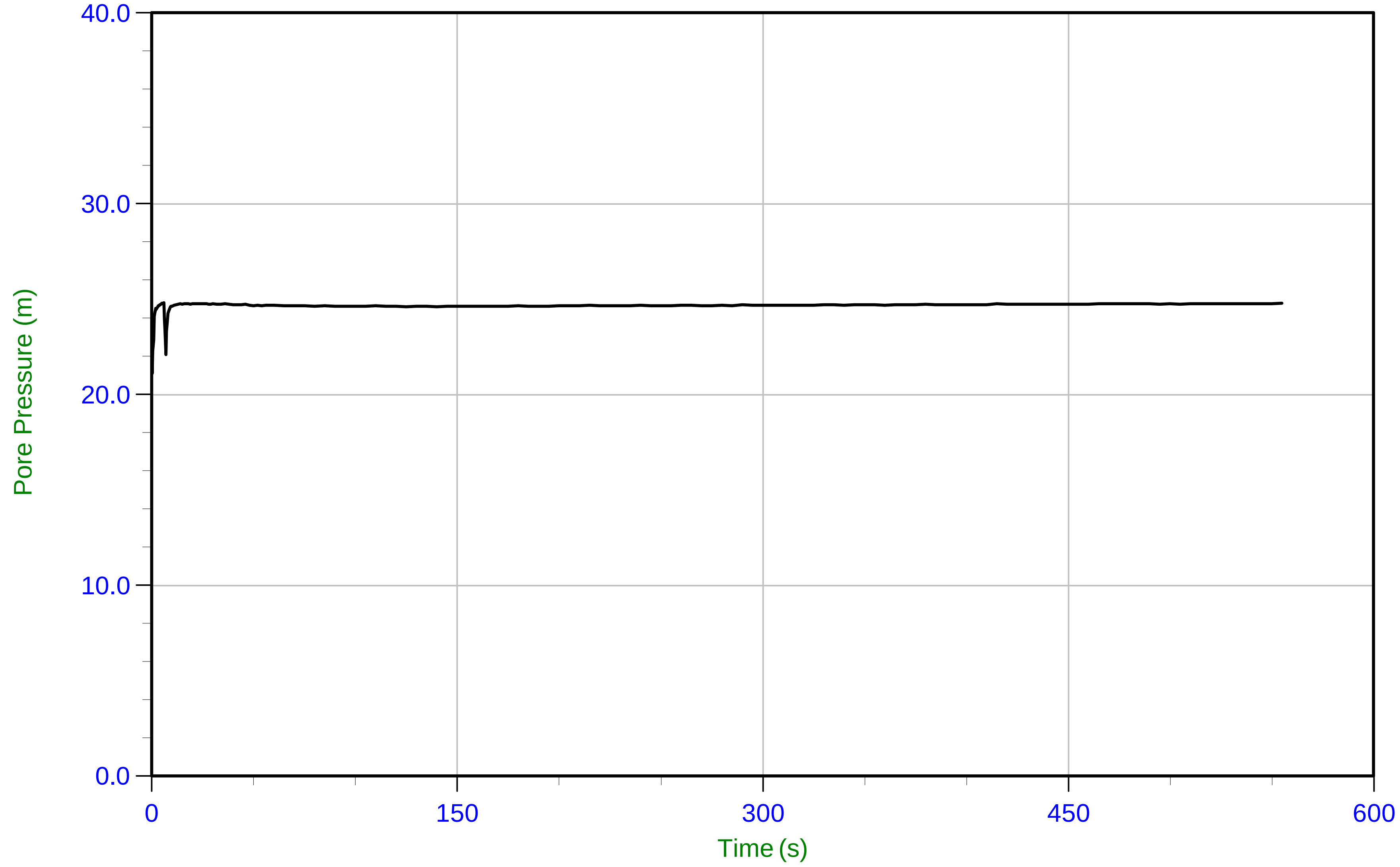
Trace Summary:

Filename: 22-05-23587_SP03.PPF
Depth: 24.500 m / 80.380 ft
Duration: 800.0 s

u Min: 2.5 m
u Max: 50.2 m
u Final: 30.8 m

WT: 2.979 m / 9.774 ft
Ueq: 21.5 m
U(50): 35.84 m

T(50): 454.1 s
Ir: 100
Ch: 1.5 cm²/min



Trace Summary:

Filename: 22-05-23587_SP03.PPF
Depth: 27.825 m / 91.288 ft
Duration: 555.0 s

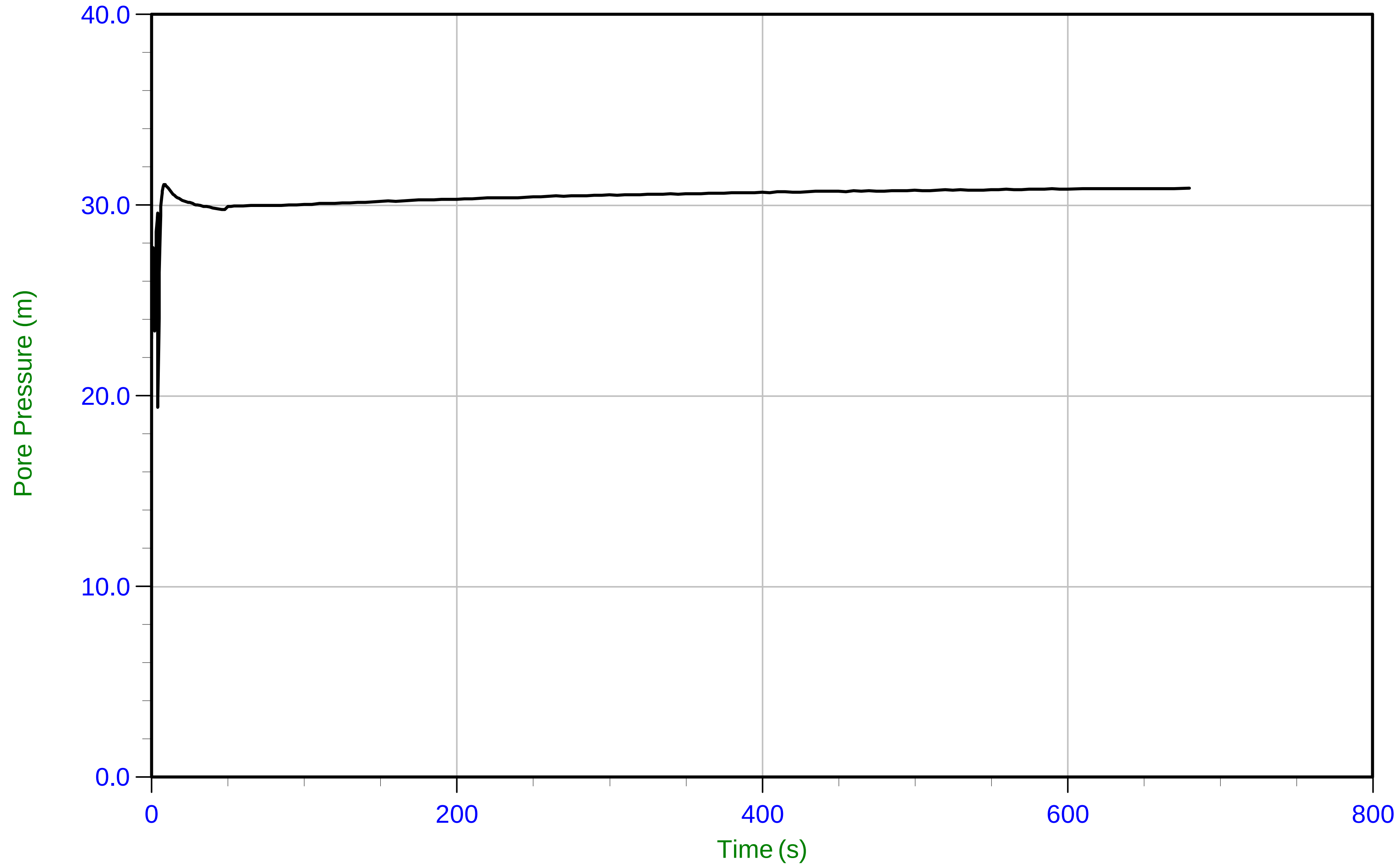
u Min: 8.1 m
u Max: 24.8 m
u Final: 24.8 m

WT: 2.983 m / 9.786 ft
Ueq: 24.8 m

Cambium Inc

Job No: 22-05-23587
Date: 02/10/2022 14:32
Site: 272 Innsfil St.

Sounding: SCPT22-03
Cone: 642:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-05-23587_SP03.PPF
Depth: 34.125 m / 111.957 ft
Duration: 680.0 s

u Min: 19.4 m
u Max: 31.1 m
u Final: 30.9 m

WT: 3.014 m / 9.888 ft
Ueq: 31.1 m

Description of Methods for Calculated CPT Geotechnical Parameters

ConeTec

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 14

Revised November 26, 2019
Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

CONETEC

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of 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: $q_t = q_c + (1-a) \bullet u_2$ (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.

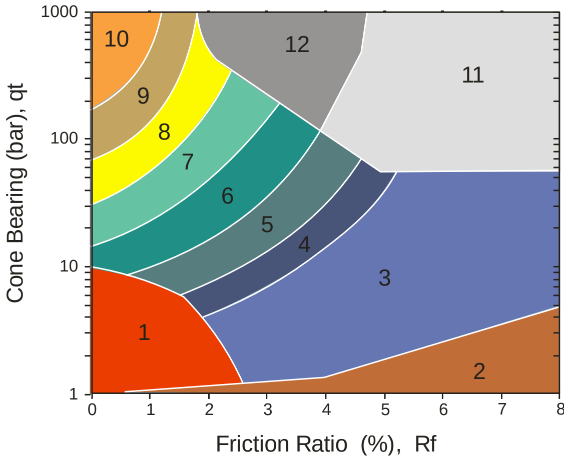
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the 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.

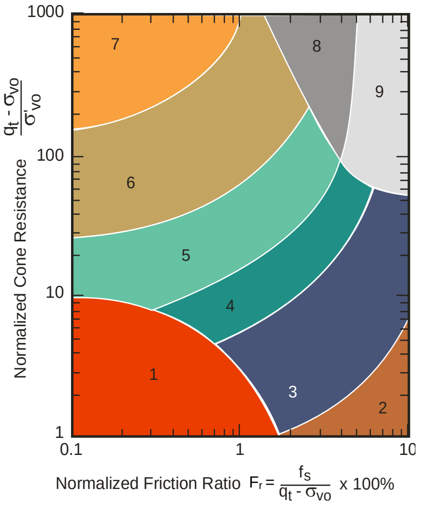


Zone		qt / N	Soil Behavior Type
1	🟥	2	sensitive fine grained
2	🟫	1	organic material
3	🟦	1	clay
4	🟦	1.5	silty clay to clay
5	🟩	2	clayey silt to silty clay
6	🟩	2.5	sandy silt to clayey silt
7	🟩	3	silty sand to sandy silt
8	🟨	4	sand to silty sand
9	🟫	5	sand
10	🟧	6	gravelly sand to sand
11	⬜	1	very stiff 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)

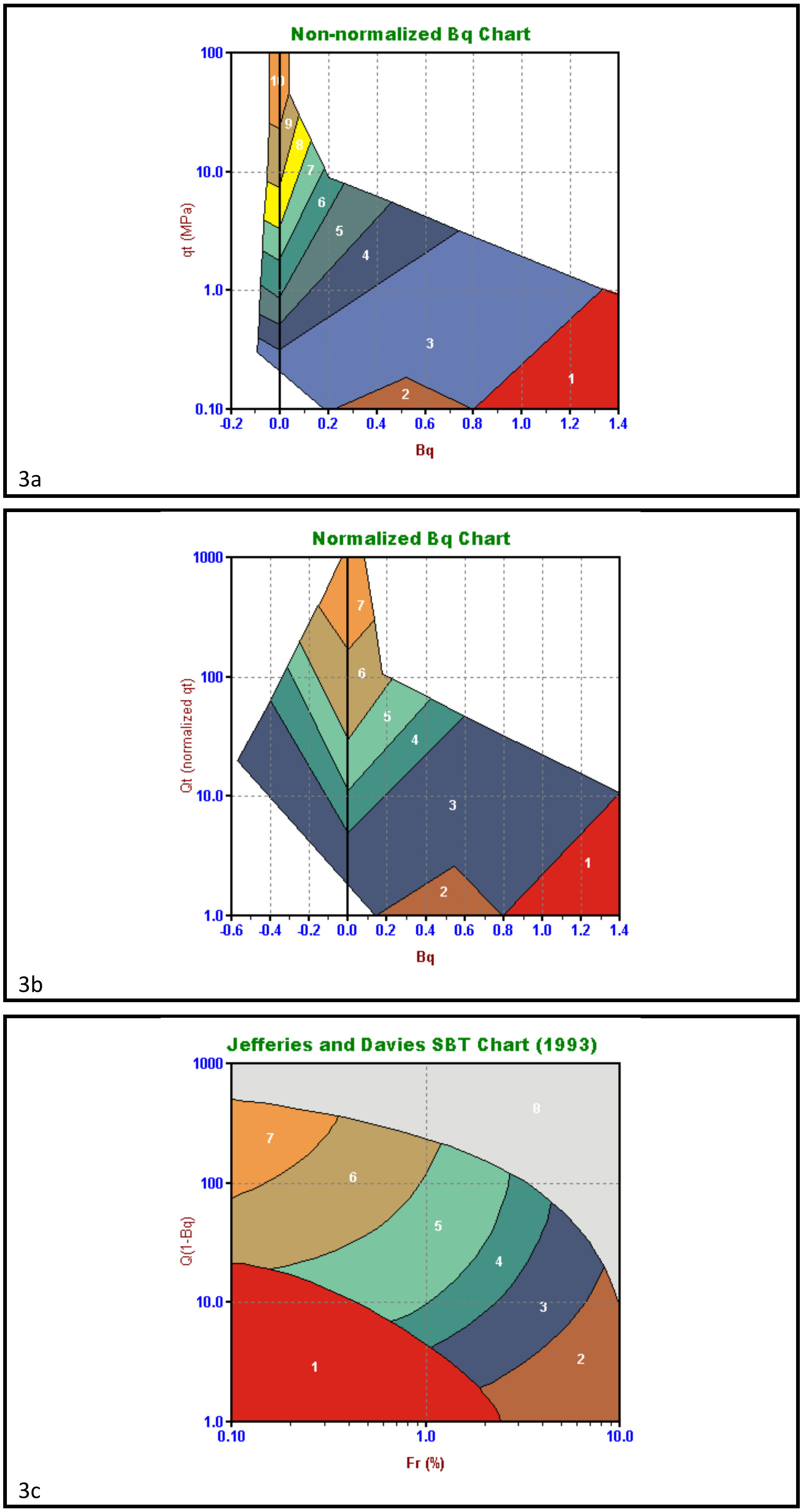


Figure 3. Alternate Soil Behavior Type Charts

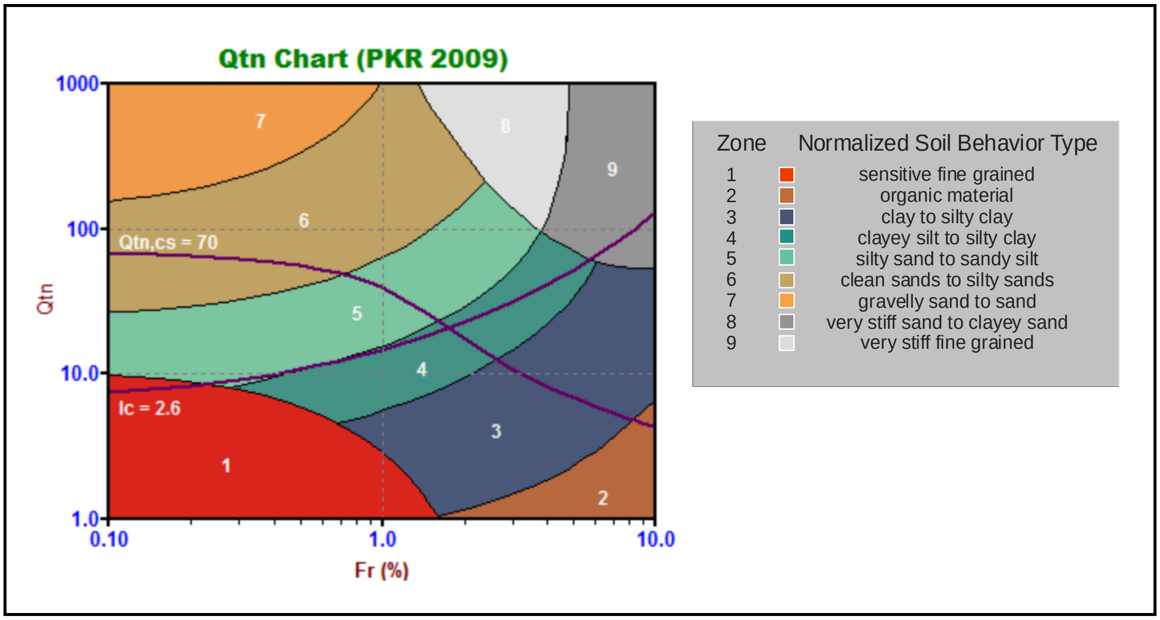


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

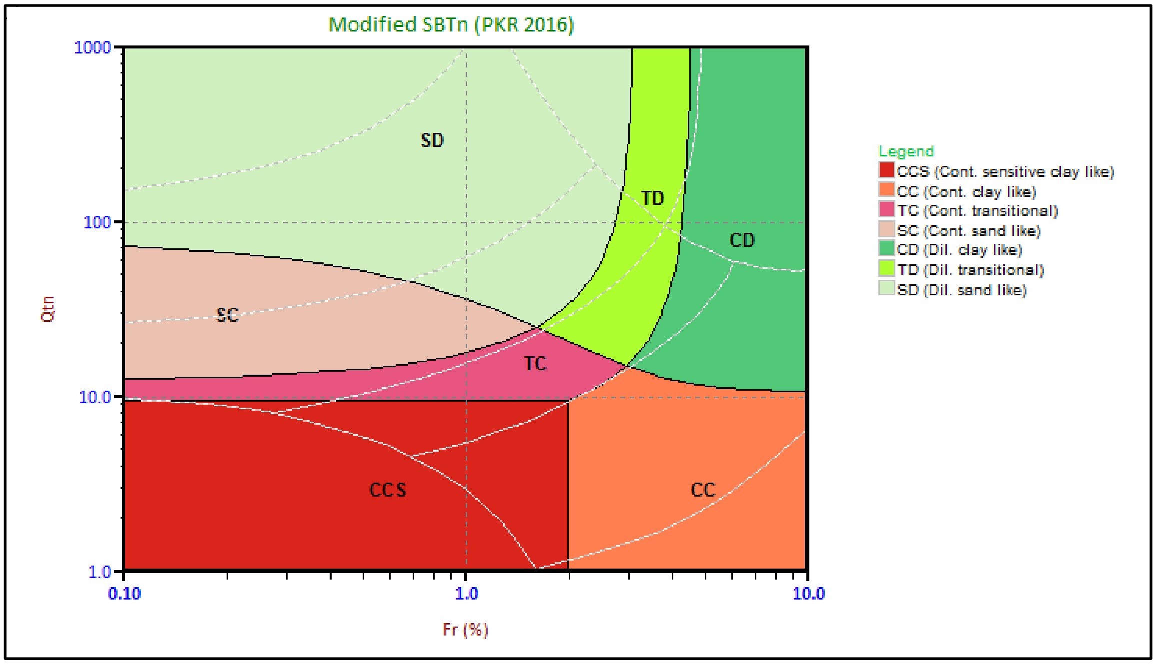


Figure 5. Modified SBTn Behavior Based Chart

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	$K_o = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	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	$(N_1)_{60} = C_n \cdot N_{60}$	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) $(N_1)_{60}Ic = C_n \cdot (N_{60}\, I_c)$ 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/P_c' 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%: $\alpha = 0, \quad \beta=1.0$ FC ≥ 35% $\alpha = 5.0, \quad \beta=1.2$ 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	$\frac{\underline{Su(Liq)}}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{\underline{Su(Liq)}}{\sigma_v'}$ 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

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No.	Reference
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Appendix E

2015 National Building Code Seismic Hazard Values

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.373N 79.694W **User File Reference:** 272 Innisfil St., Barrie, Ontario 2019-10-09 19:13 UT

Requested by: Cambium Inc.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.081	0.050	0.032	0.011
Sa (0.1)	0.111	0.072	0.048	0.017
Sa (0.2)	0.108	0.072	0.049	0.019
Sa (0.3)	0.093	0.063	0.043	0.017
Sa (0.5)	0.077	0.052	0.036	0.013
Sa (1.0)	0.047	0.031	0.021	0.006
Sa (2.0)	0.025	0.016	0.010	0.003
Sa (5.0)	0.006	0.004	0.002	0.001
Sa (10.0)	0.003	0.002	0.001	0.000
PGA (g)	0.064	0.041	0.027	0.009
PGV (m/s)	0.064	0.040	0.026	0.008

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

Canada

Appendix F

Groundwater Monitoring Results (2020 Letter)

CAMBIUM

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Geotechnical

Building Sciences

Construction
Monitoring

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Peterborough
Kingston
Oshawa

Laboratory
Peterborough

Professional Engineers
Ontario

APGO

July 28, 2020

20667340 Ontario Inc.
Typhon Group Ltd.
#110 5409 Eglinton Avenue West
Etobicoke, Ontario, M9C 5Kc

Attn: Craig Mull

Re: Groundwater Monitoring Investigation, 272 Innisfil Street – Barrie, Ontario
Cambium Reference Number: 8707-001

Dear Mr. Mull,

Cambium Inc. (Cambium) was retained by Typhon Group Ltd. (Client) c/o Innovative Planning Solutions to complete a groundwater monitoring program at 272 Innisfil St. Barrie, ON. Regular water level measurements were made between July 2019 and June 2020.

Following discussions with the Client, two (2) boreholes equipped with monitoring wells, identified as BH101-19 and BH102-19 were advanced in the proposed 15-storey residential building and commercial space area. The monitoring wells were installed on July 5 and 8, 2019 by Walker Drilling Ltd. under the supervision of a Cambium Geotechnical Analyst. The monitoring well locations are provided in Figure 1, with the relevant information summarized in Table 1.

Table 1 Monitoring Well Observations

Monitoring Well	Top of Standpipe Elevation (masl)	Ground Elevation (masl)	Well Depth (mbgs)	Bottom of Well Elevation (masl)	Screen Height (m)
BH101-19	228.87	228.98	8.75	220.23	3.0
BH102-19	229.80	229.87	8.89	220.98	3.0

As per the LSRCA requirements, Cambium has recorded the static water levels in the monitoring wells on a monthly basis over the duration of a year. In addition to the two (2) monitoring wells installed in 2019 (BH101-19 through BH102-19), four (4) existing monitoring wells (MW101 through MW104) were also measured during the monitoring program.

The following letter report includes a summary of the measured groundwater levels at the monitoring well locations.

GROUNDWATER OBSERVATIONS

Cambium visited the Site monthly between July 23, 2019 and June 17, 2020 to measure the groundwater levels within monitoring wells. The monitoring well measurements are provided within Appended Table 1 (metres above sea level (masl)) and Appended Table 2 (metres below ground surface (mbgs)).

July 28, 2020

Based on the monitoring well observations to date, the groundwater levels were found to vary from 2.85 mbgs to 4.93 mbgs, with elevations between 224.51 masl to 226.29 masl.

It should be noted that soil moisture and groundwater levels at the Site may fluctuate seasonally and in response to seasonal conditions and climatic events.

CLOSING

We trust that the information contained in this letter meets your current requirements. If you have questions or comments regarding, please do not hesitate to contact the undersigned at (705)719-0700 extension 405.

Best regards,

Rob Gethin, P.Eng.
Senior Project Manager

APPENDED: Figure 1. Site Plan
Appended Table 1 – Groundwater Elevations Measured During Monitoring Period
Appended Table 2 – Groundwater Depths Measured During Monitoring Period

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Kingston
Oshawa

Laboratory
Peterborough

Professional Engineers
Ontario

APGO

Appended Table 1 - Groundwater Elevations Measured During Monitoring Period (masl)

Date	BH101-19	BH102-19	MW101	MW102	MW103	MW104
July 23/19	225.62	225.92	-	-	-	-
July 29/19	-	-	225.37	225.66	225.92	225.85
Aug 14/19	225.51	225.83	225.28	225.64	225.84	225.75
Sept 17/19	225.41	224.95	225.21	225.52	-	225.63
Oct 25/19	225.42	225.90	225.22	224.51	225.71	225.61
Nov 19/19	225.55	225.84	-	-	225.83	225.74
Dec 12/19	225.57	-	-	224.99	-	225.75
Jan 10/20	225.64	225.94	-	224.95	225.85	-
Feb 24/20	-	225.98	-	225.07	-	225.78
Mar 12/20	-	-	-	224.98	-	226.03
Apr 15/20	225.97	226.28	225.75	226.11	226.29	226.20
May 8/20	225.95	226.11	225.68	225.55	225.98	225.84
Jun 17/20	225.85	225.99	225.66	225.53	225.87	225.74

Note

Wells with no measurements from September to March is due car parked over the well at time of measurement, snow/ice over monitoring well, etc.

Appended Table 2 - Groundwater Depths Measured During Monitoring Period (mbgs)

Date	BH101-19	BH102-19	MW101	MW102	MW103	MW104
July 23/19	3.36	3.96	-	-	-	-
July 29/19	-	-	3.23	3.39	4.13	3.98
Aug 14/19	3.47	4.05	3.32	3.41	4.21	4.08
Sept 17/19	3.57	4.93	3.39	3.53	-	4.20
Oct 25/19	3.56	3.98	3.38	4.54	4.34	4.22
Nov 19/19	3.43	4.04	-	-	4.22	4.09
Dec 12/19	3.41	-	-	4.06	-	4.08
Jan 10/20	3.34	3.94	-	4.10	4.20	-
Feb 24/20	-	3.90	-	3.98	-	4.05
Mar 12/20	-	-	-	4.07	-	3.80
Apr 15/20	3.01	3.60	2.85	2.94	3.76	3.63
May 8/20	3.03	3.77	2.92	3.50	4.07	3.99
Jun 17/20	3.13	3.89	2.94	3.52	4.18	4.09

Note

Wells with no measurements from September to March is due car parked over the well at time of measurement, snow/ice over monitoring well, etc.