Geotechnical Investigation Report - 65 Nettleton Drive, Penetanguishene, Ontario

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Prepared for:
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1.0 Introduction

Cambium Inc. (Cambium) was retained by Brittany Lowry (Client) to complete a geotechnical investigation in support of a proposed development located at 65 Nettleton Drive, in Penetanguishene, Ontario (Site).

The geotechnical investigation was required to confirm subsurface conditions at the Site and to provide geotechnical parameters and recommendations for supporting the proposed development to be conducted at the Site. This report presents and summarizes the methodology and findings of the geotechnical investigation conducted by Cambium at the Site. Based on the results of the investigation, geotechnical engineering recommendations relevant to the proposed development are provided.

1.1 Reviewed Documents

The following project documents were received and reviewed during the drafting of this report:

[1] Morgan Planning & Development – Orillia, Ontario
Severance Sketch, 65 Nettleton Drive, Part of Lot 116, Concession 1W of Penetanguishene, County of Simcoe; 1:400; File Number: 1332; October 20, 2023.

[2] Morgan Planning & Development – Orillia, Ontario
Topographic Survey CAD File for 65 Nettleton Drive – sent via email on January 31, 2024

[3] A.K. Fairbrother – Barrie, Ontario
Geological Survey: Plot Plan – For Henry Light, Part of Lot 34 Reg'd Plan No. 37, Penetanguishene, Ontario; September 8, 1982. – sent via email on October 26, 2023

[4] A.K. Fairbrother – Barrie, Ontario
Geological Survey: Sections – For Henry Light, Part of Lot 34 Reg'd Plan No. 37, Penetanguishene, Ontario; September 8, 1982. – sent via email on October 26, 2023

1.2 Standards and Guidelines

Applicable standards, guidelines and other normative documents utilized in preparing geotechnical engineering recommendations for this report are provided below.

[5] Canadian Foundation Engineering Manual – 4th Edition; Canadian Geotechnical Society; 2006.

[6] National Building Code of Canada – Natural Resources Canada; 2015

2.0 Site and Project Description

2.1 Site Description

The Site covers an area of approximately 0.6 acres and is bordered by Nettleton Drive to the south, a park to the north and residential development to the east and west. The site is currently vacant and undeveloped with medium tree coverage and a watercourse crossing the site. The topography at the Site is generally flat at the southern extents of the site with a decrease in elevation going from south to north with a maximum elevation difference of approximately 3.0 m.

A Site Location Plan is provided as Figure 1 of this report for reference.

2.2 Project Description

As per information provided by the Client [1], the property owner wants to separate the existing lot into two separate lots. The proposed development will consist of the construction of a residential dwelling on the lot that the owner is going to retain and a residential dwelling on the lot that is going to be severed.

3.0 Methodology

3.1 Borehole Investigation

Two boreholes were advanced throughout the Site on January 12, 2024, at predetermined locations confirmed with the Client to assess the subsurface conditions. The boreholes were designated as BH101-24 and BH102-24 and were terminated at a depth of 5.0 m below ground surface (mbgs).

BH102-24 was outfitted with a monitoring well to allow for subsequent groundwater level monitoring at the Site. A Borehole Location Plan is provided as Figure 2 of this report for reference.

Borehole drilling and sampling were completed 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 and solid stem augers with 50 mm O.D. split spoon samplers.

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 mbgs and at 1.5 m intervals below that depth.

Open boreholes were checked for groundwater and general stability prior to backfilling. The boreholes and monitoring wells were backfilled in accordance with O.Reg. 903, as amended and the Site was restored to a reasonable condition.

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, laboratory testing, and storage. Borehole logs are provided in Appendix A.

3.2 Site Survey

The elevations and coordinates for all boreholes and monitoring well locations were obtained using a GNSS receiver during a survey following the completion of drilling. The benchmark utilized for this survey (top of manhole located in the Rotary Park parking lot, north of the Site) was selected by the Cambium geotechnical analyst based on the provided topographic survey plan [2] and has a geodetic elevation of 179.27 metres above sea level (mASL).

3.3 Physical Laboratory Testing

Physical laboratory testing, including two particle size distribution analyses (LS-702, 705), was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters. Natural moisture content testing (LS-701) was completed on all retrieved soil samples. The physical laboratory testing results are presented in Appendix B and are discussed in Section 4.0.

4.0 Subsurface Conditions

The stratigraphy encountered in the boreholes is indicated on the attached borehole logs in Appendix A. It is 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.).

In general, the encountered subsurface conditions consist of surficial sand which may be possible fill or disturbed native material transitioning to undisturbed native deposits. The native deposits at the Site generally consisted of non-cohesive sand deposits. Bedrock was not encountered during this investigation.

4.1 Regional Geology

Ontario Geological Survey (OGS) quaternary geologic mapping indicates that the Site is projected to fall within the glaciomarine deposits, sand, gravelly sand and gravel nearshore and beach deposits.

Physiographic mapping (Chapman, L.J. & Putnam, D.F., 2007) shows the Site within the sand plains formations.

4.2 Possible Reworked or Disturbed Native Soil

A surficial layer of non-cohesive reworked or disturbed soil deposits were encountered and extended to depths from approximately 1.5 to 2.3 mbgs. The composition of the non-cohesive reworked/disturbed native materials was a sand material with varying amounts of silt and gravel in each borehole. The soils were predominantly brown in colour.

SPT N values within the non-cohesive fill deposits generally ranged between 2 and 50, indicating very loose to dense relative density. It was noted that in BH102-24, a boulder was encountered during the split spoon sampling after approximately 150 mm below ground surface, thus encountering a high SPT N value which is not indicative of the surficial soil conditions at BH102-24. Organics were noted within the possible fill/disturbed soils in each borehole; however, it is unclear whether the encountered organics are present within the deposit or represent caved material due to the drilling method (i.e., use of solid stem augers).

Natural moisture contents in the non-cohesive deposits ranged between 13.9% and 28.5% based on laboratory testing.

Assessments of organic matter content or other topsoil quality tests were beyond the scope of this study.

4.3 Native Sand Deposits

A sand material was encountered below the surficial inferred fill/reworked native soils in each borehole advanced at the site between depths of 1.5 mbgs and 2.3 mbgs. The native sand materials extended to the termination depths of 5.0 mbgs. The material was grey in colour with varying amounts clay and gravel observed within this formation.

SPT N values in the sand material ranged from 5 to 100 blows, indicative of a loose to compact relative density. Samples obtained in this layer were wet with natural moisture contents ranging from 9.3% to 49.7% based on laboratory testing.

Laboratory particle size distribution analyses were completed for two samples in the sand layer in order to assess the soil composition. The testing results are provided in Appendix B and are summarized in Table 1 based on the USCS.

Table 1 Particle Size Distribution – Native Sand Deposits

Sample ID	Depth (mbgs)	Description	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH101-24 SS4	2.3 – 2.7	Sand some Silt trace Clay trace Gravel	7	65	19	9	237
BH102-24 SS3	1.5 – 2.1	Sand some Gravel trace Silt trace Clay	10	77	9	4	16.0

4.4 Bedrock

Bedrock was not confirmed in any of the boreholes advanced by Cambium at the Site. The boreholes were terminated at a depth of 5.0 mbgs, corresponding to absolute elevations between 175.29 mASL and 175.70 mASL.

4.5 Groundwater

The encountered soils were predominantly described as being moist to wet throughout the borehole investigation.

Standing water (free water) was observed in each borehole upon completion of drilling, at depths of 0.6 mbgs and 0.1 mbgs in boreholes BH101-24 and BH102-24 respectively. Caving (sloughing) was only observed in BH101-24 at a depth of 2.3 mbgs. The groundwater level observations in the boreholes are not representative of the stabilized groundwater conditions and as such, the groundwater table elevation will vary.

One monitoring well was installed in BH102-24 to allow for subsequent groundwater level monitoring at the Site. The water levels measured in the installed well following the investigation are summarized in Table 2.

Table 2 Groundwater Observations During Monitoring Events

Date	Borehole	Ground Elevation (mASL)	Water Level in Monitoring Well (mbgs)	Water Level Elevation (mASL)	Bottom of Well Elevation (mASL)
January 31, 2024	BH102-24	180.70	-0.67	181.37	176.57
February 7, 2024	BH102-24	180.70	-0.58	181.28	176.57
February 16, 2024	BH102-24	180.70	-0.69[1]	181.39[1]	176.57
March 8, 2024	BH102-24	180.70	-0.04	180.74[1]	176.57

[1] groundwater measured in stickup pipe was frozen during measurement.

It is noted that the encountered and measured groundwater levels reflect the groundwater conditions in the boreholes at the time of the borehole investigation and subsequent monitoring events between January and March 2024. There is currently a creek crossing the centre of the site as shown in Figure 2 which may influence the existing groundwater conditions at the Site. Groundwater levels at the Site may be anticipated to vary between and beyond the borehole locations and to fluctuate with seasonal variations in precipitation and snowmelt.

Since all groundwater measurements to date were completed in the winter during significant changes in the weather from freezing to melting to significant precipitation events, they are likely not truly representative of stabilized groundwater levels at the site and it is recommended that further groundwater measurements are completed in the spring and summer to help determine the seasonally high and low groundwater levels at the site.

5.0 Geotechnical Considerations

This section of the report provides engineering information on, and recommendations for, the geotechnical design aspects of the project based on our interpretation of the borehole information, the laboratory test data, and our understanding of the project requirements. The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the Site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own independent interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like. Cambium will not assume any responsibility for construction-related decisions made by contractors on the basis of this report.

5.1 Site Preparation

Existing topsoil and organic material, any loose reworked/disturbed native materials and any deleterious material (i.e., imported fill material, construction debris, fibrous material, asphalt, brick fragments, etc.) encountered should be excavated and removed beneath proposed development areas prior to construction. Additionally, this material should be excavated and removed to a minimum distance of 3 m around the building footprint. Any topsoil and materials with significant quantities of organics and deleterious materials are not appropriate for use as fill.

The exposed subgrade should be proof-rolled and inspected by a qualified geotechnical engineer prior to placement of any granular fill or foundations. Any loose/soft soils identified at the time of the 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 soils can become unstable if wet or saturated. Such conditions are common in the spring and late fall. Under these conditions, temporary use of granular fill, and possible separating/reinforcing geotextiles, may be required to prevent severe rutting on construction access routes.

5.2 Groundwater Control and Dewatering

Groundwater (free water) was observed in all boreholes throughout the Site upon completion of drilling at depths as high as 0.9 mbgs. During the water level monitoring event conducted in between January and March 2024, the water level elevations measured in BH102-24 ranged from approximately 36 mm to 680 mm above the existing ground surface, which may indicate perched or confined groundwater conditions.

Based on the water level observations during drilling and subsequent monitoring events, excavation works at the Site can be expected to encounter groundwater and should be accompanied by advance dewatering to maintain a sufficiently dry excavation.

The groundwater level measured at the Site represents a momentary observation, not taken during a high groundwater season (i.e., typically between March to May). Therefore, it could reasonably be expected that the groundwater table will be higher than measured at other times of the year, and it is recommended to conduct excavation and foundation works during drier times of the year to avoid groundwater-related issues and to save costs related to dewatering efforts.

It is noted that the elevation of the groundwater table will vary due to seasonal conditions and in response to heavy precipitation events as well as the creek elevation at the middle of the Site. As the borehole investigation and follow-up well readings were conducted during winter, it is recommended that further well readings are conducted to capture ground water readings during drier season. Additionally, if a test pit investigation is conducted prior to construction, a better understanding would be gained as to whether the groundwater throughout the site is locally perched, and how it would affect deeper excavations. If the groundwater table does drop with drier seasons, to minimize predictable water issues and costs, it would be recommended that excavations and in-ground construction be performed in drier seasons.

Based on the current information, is anticipated that excavations for footings (to meet frost penetration depth requirements) will extend below the groundwater table.

Because most excavations will likely extend below the groundwater table and subgrade soils consist of mainly a non-cohesive sand material, groundwater seepage is anticipated at the Site.

Generally, this seepage should be manageable using filtered sumps and pumps, however, depending on the location, depth, size, and staging of the excavations, a Permit to Take Water (PTTW) or registry with the Environmental Activity and Sector Registry (EASR) of the Ministry of the Environment, Conservation and Parks (MECP) may be required.

In the event that the excavations for site services such as sewers are expected to extend below the water table, provisions will be required to maintain sufficiently dry excavations to permit safe working conditions. In this context, the groundwater level should be drawn down to at least 1 m below the base of the excavation, prior to the excavations reaching the base level, to reduce the potential for loosening of the excavation base due to seepage pressures. Further, care should be taken to direct surface water away from the open excavations. Excavations extending below the groundwater table through, or in, saturated non-cohesive deposits will require the use of positive dewatering in the form of perimeter trenching with filtered sumps and pumps, and/or well points.

Water takings in excess of 50,000 L/day are regulated by the MECP. Certain takings of groundwater and storm water for construction site dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP's EASR replaces the need to obtain a PTTW and a Section 53 approval. A Category 3 PTTW is required where the proposed water taking is greater than 400,000 L/day.

The dewatering system is the Contractor's responsibility and the rate and volume required for dewatering is dependent on the construction methods and staging chosen by the contractor. Further, the contractor will be responsible for obtaining any required discharge approvals.

It is recommended that groundwater monitoring events continue to especially during the drier months in order to determine if the groundwater levels drop significantly. The groundwater conditions must be below the base of the footings excavation during the construction phase of the project.

5.3 Excavations

Excavations will be required at the Site to construct footings for the proposed structures, which are currently assumed to be single detached dwellings. It is currently not known whether the structures will be constructed with basements, however, based on the findings during the field investigation and subsequent groundwater monitoring events, it is not recommended for basements to be constructed on either severed lot as the soil conditions are deemed to be too wet just below surface. Therefore, for the purposes of these recommendations it is assumed that excavation depths below the current ground surface should not exceed 1.6 mbgs unless proper advanced dewatering measures are in place and followed. A test pit investigation is recommended to conduct in drier months prior to construction to determine whether the groundwater throughout the site is locally perched, and how it would affect deeper excavations.

In the areas of the Site where unsupported excavations to the required depths are deemed feasible, the excavations must be carried out in accordance with the latest edition of OHSA and Ontario Regulation 213/91 (as amended).

For practical purposes, the overburden soils at the Site above the groundwater table and within continually dewatered depths can be considered Type 3 soils, as such, excavation side slopes should be no steeper than 1H:1V. Soils below the groundwater table are to be considered Type 4 soils in accordance with OHSA, and excavation side slopes are to be limited to 3H:1V.

It is noted that a workspace allowance of approximately 0.5 m should be maintained between building lines and the toe of the adjacent temporary excavation slopes (applies for slopes with a maximum inclination of 1H:1V).

Excavation slopes should be protected during construction from precipitation, runoff, or snow/ice melt and should be inspected regularly for signs of instability. If localized instability is noted during excavation or if wet conditions are encountered, the side slopes should be flattened as required to maintain safe working conditions or the excavation sidewalls must be fully supported (shored).

The crest area of unsupported excavation slopes is to be held free of any loading (i.e., by heavy machinery, stockpiled construction materials, etc.).

If basements are to be constructed below the groundwater elevation, the basements will need to be fully waterproofed and there would be a need for advance dewatering to be conducted prior to construction. A contractor whose expertise is in waterproofing should be consulted prior to construction. In addition, perimeter and subfloor drains connected to a sump pump with backups and battery for power outages are also required. Given the buildings are to be situated on a slope, adequate drainage and grading would be required to direct water away from the buildings.

5.4 Frost Penetration

Based on climate data and design charts, the maximum frost penetration depth below the surface at the Site is estimated at 1.6 mbgs. Exterior footings for the proposed structure should be situated at or below this depth for frost penetration or should be appropriately protected. Any services should be located below this depth or be sufficiently insulated.

5.5 Foundation Design

Based on the results of the borehole investigation, it is recommended to avoid the construction of basements at the Site, as basement levels extending beneath the current grades at the Site will likely encounter significant issues related to the ingress of groundwater into the basement level. In addition, very loose to loose soils were encountered until 3.0 mbgs and 2.3 mbgs in BH101-24 and BH102-24 respectively. The depth to competent native soils varies across the Site and Table 3 outlines the minimum depths at which it is expected that competent native soils will be encountered based on BH101-24 and BH102-24.

Table 3 Depth to Competent Native Soils

Borehole ID	Native Soil	Depth (mbgs)	Elevation (mASL)
BH101-24	Sand some Silt trace Clay trace Gravel	3.0	177.99
BH102-24	Sand some Gravel trace Silt trace Clay	2.3	178.40

Should incompetent (loose, soft and/or deleterious) soils be encountered at the proposed footing depths following excavation, these soils are to be sub-excavated down to competent soils under the guidance of a qualified geotechnical engineer and replaced with competent engineered fill as detailed in Section 5.7. The recommendations and bearing capacities provided in this report assume that any incompetent materials encountered at underside of footing depths will be sub-excavated and replaced in this manner.

Further, any large cobbles or boulders encountered at footing subgrade elevations are to be removed and replaced with engineered fill or other material approved by a geotechnical engineer.

If significant grade raises are to be completed at the site to raise site grades up to or above the elevation of Nettleton Drive, basements could be considered but loose soils below the building footprints would still need to be removed and replaced with engineered fill.

5.5.1 Foundations on Engineered Fill

In areas where the proposed founding levels are above the level of competent native soil, or where sub excavation is required, footings made to bear directly on a pad of engineered fill constructed per the recommendations in Section 5.7.1. From a preliminary perspective, footings placed on approved engineered fill and appropriately protected from frost may be designed for a preliminary allowable bearing capacity of 100 kPa at SLS and 150 kPa at ULS. Cambium should be retained to review the final grading plan, as the preliminary engineered fill bearing capacity values will change depending on grade raises, engineered fill thickness, material and the native subgrade soil the engineered fill pad is constructed on.

Settlement potential at the above-noted SLS loadings is less than 25 mm and differential settlement should be less than 20 mm.

If higher bearing capacities are required, Cambium should be consulted to provide further foundation recommendations (i.e., ground improvement, deep foundations, etc.). In addition, compact to dense soils were observed at varying depths and higher design bearing capacity values may be possible depending on footing elevations and location.

5.5.2 Helical Piles

The proposed structures can also be founded on insulated grade beams supported using helical piles advanced into competent soil strata at depth. A speciality helical pile contractor should be contacted to determine the number and spacing of piles, required torques and the suitability of the founding strategy. Helical piles consist of steel shafts with helical flights, which are rotated into the subsurface while torque is monitored. The helical piles must extend into competent soils, giving preliminary projected pile lengths at this Site of roughly 4.6 m to 5.0 m. The piles must be designed according to applicable standards. Given that the systems are often proprietary in nature, it is recommended to consult a speciality contractor/design engineer with experience in designing/constructing helical pile foundations.

It is recommended that if helical piles are to be used, a test trial using one to two helical piles should be conducted to ensure that if artesian conditions are present, the pile itself does not provide a conduit for the groundwater to migrate to the surface from the bearing soils.

As an alternative to helical piles, or other methods provided by specialist geotechnical contractors may also prove-cost effective to support foundations placed according to the finished floor elevation. Methods should be conceptualized and designed in consultation with specialized contractors.

5.6 Floor Slabs

To create a stable working surface, to distribute loadings, and for drainage purposes, an allowance should be made to provide at least 200 mm of OPSS.MUNI 1010 Granular A compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD) beneath all floor slabs. It is recommended that all floor slabs are situated at least 500 mm above the seasonal high groundwater elevation.

Within any interior areas that may be exposed to freezing conditions for extended periods of time, the floor slab may be susceptible to frost heaving, depending on the composition of the subgrade. The subgrade underlying these areas should be adequately insulated to prevent frost penetration.

5.7 Backfill and Compaction

Engineered fill, if required 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 100% of SPMDD, as confirmed by nuclear densometer testing.

Imported material for engineered fill should consist of clean, no-organic, soils, free of chemical contamination or deleterious material. The moisture content of the engineered fill will need to be close enough to optimum at the time of placement to allow for adequate compaction.

Foundation wall and any buried utility backfill material should consist of free draining imported granular material. Excavated silty clay to silt and clay materials, may not be suitable for re-use as backfill for foundation walls and for grading purposes. Geotechnical testing of the material will be required to confirm suitability and compaction parameters (i.e., Proctor testing to confirm optimum moisture content). The fines (silt and clay) content of materials utilized as backfill for foundation walls/grading should not exceed 35%, which will need to be confirmed by sampling from stockpiled material and conducting confirmatory grain size analyses.

Typically, backfill should be placed in maximum 300 mm thick lifts and should be compacted to a minimum of 98% of SPMDD. Backfill adjacent to the structural elements (i.e., foundation walls) should be compacted to 95% of SPMDD taking care not to damage the adjacent structures. The backfill material in the upper 300 mm below the pavement subgrade elevation should be compacted to 100% of SPMDD in all areas.

All existing vegetation, topsoil, organic and non-organic fills, and any loose soils shall be removed down to a competent base. Backfill areas must be approved by a qualified geotechnical engineer prior to placement of any new fill, to ensure the suitability of subgrade conditions.

5.7.1 Engineered Fill

Where the existing fill is treated as an engineered fill to support structural elements such as foundations and/or floor slabs the following is recommended for the construction of engineered fill:

I. Remove any and all existing vegetation, surficial topsoil / organics, organic fills or fills and any loose soils to a competent subgrade for a suitable envelope.

II. The area of the engineered fill should extend horizontally 1 m beyond the outside edge of the foundations then extend downward at an imaginary 1H:1V slope to the competent approved native soil. The exposed edges of the engineered fill should be sloped at a maximum of 3H:1V to avoid weakening of the engineered fill edges due to slope movement. If fill is required adjacent to sloped banks (i.e., slope steeper than 3H:1V), the fill shall be placed in stepped planes to avoid a plane weakness.

III. The subgrade or base of the engineered fill area must be approved by Cambium prior to placement of any new fill, to ensure that suitability of subgrade condition. The area(s) should then be proof rolled in conjunction with an inspection by Cambium to confirm that the exposed soils are native, undisturbed and competent, and have been adequately cleaned of ponded water and all disturbed, loosened, softened, organic and other deleterious material. Some of the localized near-surface loose/soft soils will also likely need to be removed prior to placement of engineered fill as directed by Cambium during proof-rolling.

IV. Materials for reuse as engineered fill must be approved by Cambium prior to placement. In this regard, approved disturbed native or the native soil which are near their optimum water contents and do not contain topsoil or organics or any other deleterious materials can be reused on Site as engineered fill. The materials for use as engineered fill must be maintained within about 2% of optimum water content for compaction. Based on the measured natural water contents, most of the native sandy soils are generally moist to wet and may require drying during engineered fill construction. Their actual water content will need to be assessed in comparison to the laboratory optimum water contents for compaction at the time of construction. If native soils from the site are not used as engineered fill, imported material for engineered fill should consist of clean, non-organic soils, free of chemical contamination or deleterious material that meet OPSS 1010.MUNI SSM or Granular B Type I material. The approved material should be placed in maximum 300 mm thick loose lifts, compacted to 100% of SPMDD. Any frost penetration into the fill material must be removed prior to placement of subsequent lifts of fill and reviewed by Cambium.

V. The engineered fill should be placed at least 600 mm above the elevation of the proposed underside of footing.

VI. Due to the potential negative effects of differential settlement between the engineered fill and the native soils, it is generally not recommended that individual footings be supported on both engineered fill and on native soils. In addition, differential settlement may occur between different footings if some of the footings are on native soils, and some are on engineered fill.

VII. Reinforcing steel bars should be included and placed within the footings and the top of the foundation walls. All tie reinforcing steel bars should be included and placed within the top of the foundation walls. All tie reinforcing steel bars should have at least 600 mm of overlap. The actual steel reinforcement design should be confirmed / designed by the project structural engineer.

VIII. Full time testing and inspection of the engineered fill will be required for it to be used as a founding material, as outlined in Section 4.2.2.2 of the Ontario Building Code.

IX. The final surface of the engineered fill should be protected as necessary from construction traffic, ponded water and freezing, and should be sloped to provide positive drainage for surface water during and following the construction period. During periods of freezing weather, additional soil cover should be placed above final subgrade to provide frost protection.

5.8 Subdrainage

The exterior grade around any buildings should be sloped from the walls to direct surface runoff away from the residential structures. In order to deal with seasonal perched water and/or the water table, perimeter subdrains consisting of geotextile-wrapped perforated pipe subdrains set in a trench of clear stone and connected to a sump or other frost-free positive outlet are recommended.

Subsurface walls should be adequately damp proofed above the water table and waterproofed below the water table.

5.9 Lateral Earth Pressure

Lateral earth pressure coefficients (K) are shown in Table 4 and may be used for the preliminary design of temporary and permanent structures at the Site. It is assumed that potential lateral loads will result from cohesion less, frictional materials, such as granular backfill and the encountered near surface native sand.

Table 4 Lateral Earth Pressure Coefficients

Stratum/Parameter	γ / γ ‘ [kN/m^3]	Φ [°]	c [kN/m^2]	K_o [-]	K_a [-]	K_p [-]
Sand *compact to very dense*	19 / 10.5	31	0	0.48	0.32	3.12
Engineered Fill (per recommendations provided above)	20.5 / 11.5	32	0	0.47	0.31	3.25

Where:	γ	=	bulk unit weight of soil (kN/m^3)
	γ'	=	submerged (effective) unit weight of soil (kN/m^3)
	φ	=	internal angle of friction (degrees)
	c	=	soil cohesion (kN/m^2)
	K_a	=	Rankine active earth pressure coefficient (dimensionless)
	K_o	=	Rankine at-rest earth pressure coefficient (dimensionless)
	K_p	=	Rankine passive earth pressure coefficient (dimensionless)

The coefficients provided in Table 4 assume that the surface of the granular backfill is horizontal against any proposed retaining wall, and the wall is vertical and smooth. Cambium should be contacted to provide updated lateral earth pressure coefficients should the assumptions differ to those noted.

5.10 Seismic Site Classification

The Ontario Building Code (OBC) specifies that the structures should be designed to withstand forces due to earthquakes. For the purpose of earthquake design, geotechnical information shall be used to determine the “Site Class”. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the OBC (2012). The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistances (N_{60} values).

The boreholes advanced on site were maximum of about 5.0 mbgs. Therefore, the site classification recommendation would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the area. It is assumed that the soils encountered in the samples retrieved remain continuous to a minimum depth of 30 m below the bottom of any foundations. In addition, average ‘N_{60}’ values for soils were assumed for the site. Based on the explored soil properties and in accordance with Table 4.1.8.4.A, it is recommended that Site Class “**E**” (soft soil) be applied for structural design at the Site.

Consideration could be given to carrying out shear wave velocity testing (“MASW”) to evaluate whether an improved seismic site class can be obtained. Further details regarding shear wave velocity testing could be provided upon request.

6.0 Report Limitations

6.1 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. It is important that onsite geotechnical supervision be provided at this site for excavation and backfill procedures, deleterious soil removal, subgrade inspections and compaction testing.

6.2 Changes in Site and Project Scope

This geotechnical engineering report is intended for planning and design purposes only.

Subsurface conditions can be altered by the passage of sufficient time, natural occurrences, and human intervention. In particular, consideration should be given to contractual responsibilities as they relate to control of groundwater seepage, disturbance of soils, and frost protection.

The design parameters provided, and the engineering advice offered in this report are intended for use by the owner and its retained design consultants. If there are changes to the project scope and development features, these interpretations made of the subsurface information, for geotechnical design parameters, advice, and comments relating to constructability issues and quality control may not be complete for the project. Cambium should be retained to conduct further review to interpret the implications of such changes with respect to this report.

7.0 Closing

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

Respectfully submitted,

Cambium Inc.

DocuSigned by:
E70D3B9336AE4BD...

Chris Malliaros, P.Eng.
Project Coordinator - Geotechnical

DocuSigned by:
0B68D45279A94B7...

Stuart Baird, M.Eng., P.Eng.
Director – Geotechnical, CQV, Building Sciences

SEB/cm

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8.0 Standard Limitations

Limited Warranty

In performing work on behalf of a client, Cambium relies on its client to provide instructions on the scope of its retainer, and, on that basis, Cambium determines the precise nature of the work to be performed. Cambium undertakes all work in accordance with applicable accepted industry practices and standards. Unless required under local laws, other than as expressly stated herein, no other warranties or conditions, either expressed or implied, are made regarding the services, work or reports provided.

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When preparing reports, Cambium considers applicable legislation, regulations, governmental guidelines, and policies to the extent they are within its knowledge, but Cambium is not qualified to advise with respect to legal matters. The presentation of information regarding applicable legislation, regulations, governmental guidelines, and policies is for information only and is not intended to and should not be interpreted as constituting a legal opinion concerning the work completed or conditions outlined in a report. All legal matters should be reviewed and considered by an appropriately qualified legal practitioner.

Site Assessments

A site assessment is created using data and information collected during the investigation of a site and based on conditions encountered at the time and particular locations at which fieldwork is conducted. The information, sample results and data collected represent the conditions only at the specific times at which and at those specific locations from which the information, samples and data were obtained and the information, sample results and data may vary at other locations and times. To the extent that Cambium’s work or report considers any locations or times other than those from which information, sample results and data was specifically received, the work or report is based on a reasonable extrapolation from such information, sample results and data but the actual conditions encountered may vary from those extrapolations.

Only conditions at the site and locations chosen for study by the client are evaluated; no adjacent or other properties are evaluated unless specifically requested by the client. Any physical or other aspects of the site chosen for study by the client, or any other matter not specifically addressed in a report prepared by Cambium, are beyond the scope of the work performed by Cambium and such matters have not been investigated or addressed.

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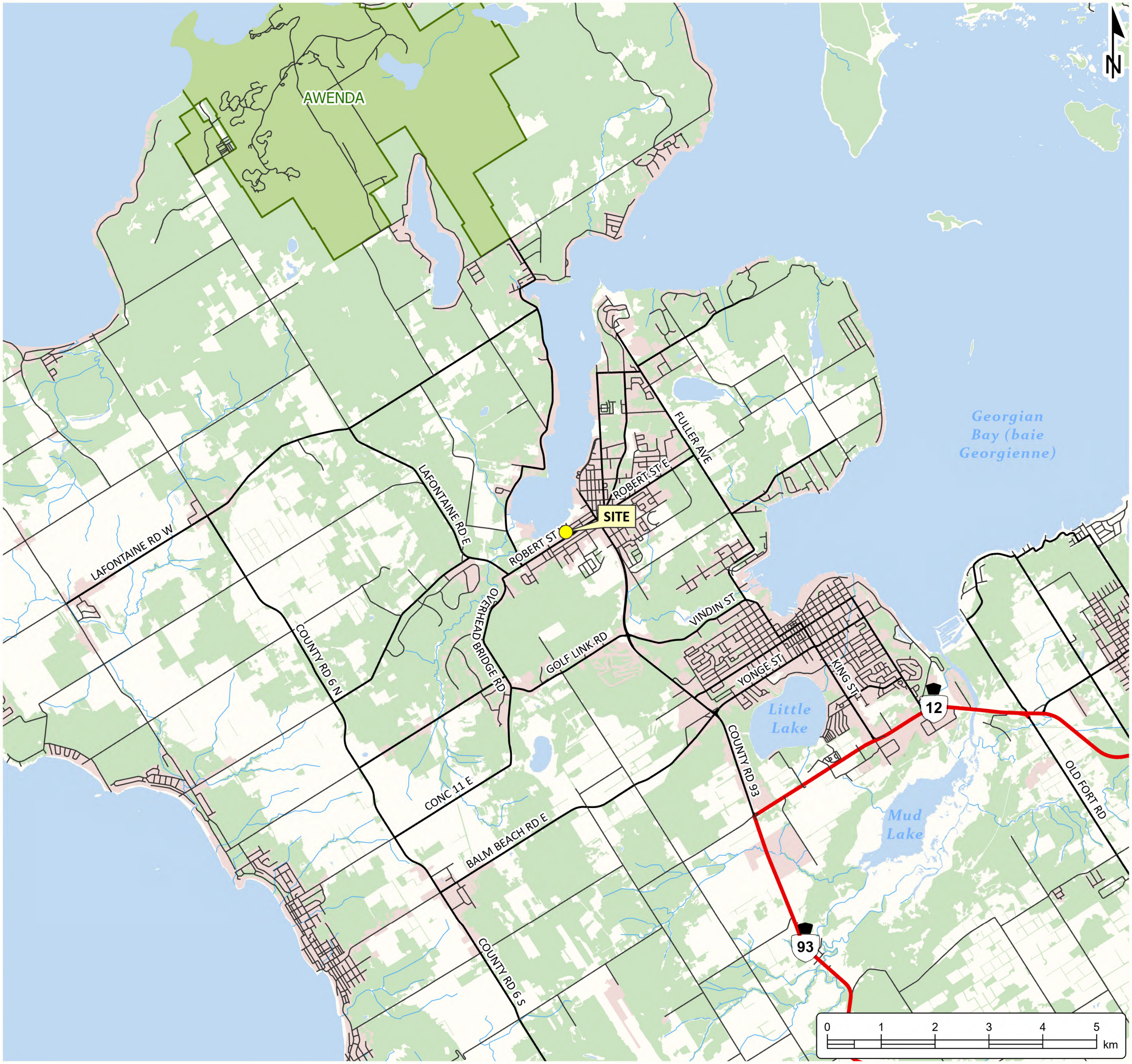
Appended Figures



GEOTECHNICAL INVESTIGATION

BRITTANY LOWRY
65 Nettleton Drive
Penetanguishene, Ontario

LEGEND

- Highway
- Major Road
- Minor Road
- Railway
- Watercourse
- Water Area
- First Nations Reserve
- Provincial Park
- Wooded Area
- Built Up Area

Notes:
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SITE LOCATION PLAN

Project No.: 19210-001	Date: February 2024 Rev.:
Scale: 1:100,000	Projection: NAD 1983 UTM Zone 17N
Created by: MAT / Checked by: CM	Figure: 1

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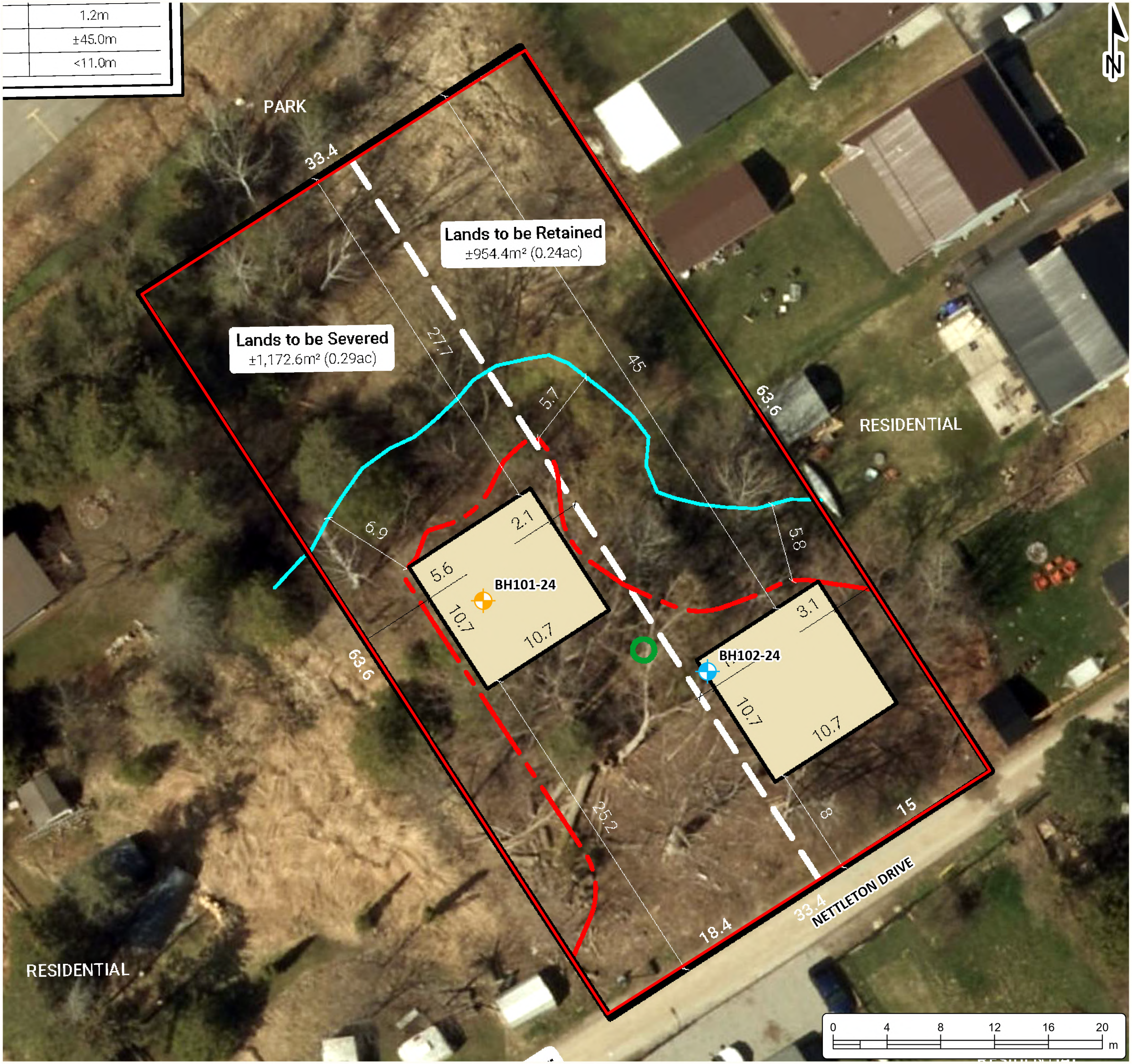

GEOTECHNICAL INVESTIGATION

BRITTANY LOWRY
65 Nettleton Drive
Penetanguishene, Ontario

LEGEND

- Borehole
- Monitoring Well
- Site (approximate)

Notes:
- Site plan overlay was created by Morgan Planning and Development, file no. 1332, dated October 20, 2023, drawing title: Severance Sketch.
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BOREHOLE LOCATION PLAN

Project No.: 19210-001	Date: February 2024 Rev.:
Scale: 1:400	Projection: NAD 1983 UTM Zone 17N

Created by: MAT	Checked by: CM	Figure: 2

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Appendix A

Borehole Logs

CAMBIUM

Client:	Brittany Lowry	**Project Name:**	65 Nettleton Drive, Penetanguishene	**Log of Borehole:**	BH101-24
Contractor:	Ontario Soil Drilling	**Method:**	Track Mounted Solid Stem Auger	Page:	1 of 1
Project No.:	19210-001	**Elevation:**	180.29 mASL	**Date Completed:**	January 12, 2024
Location:	65 Nettleton Drive, Penetanguishene	**UTM:**	17T **N:** 4957566 **E:** 583766		

SUBSURFACE PROFILE | **SAMPLE**

Elevation (m)	Depth	Description	Elevation / Depth	Number	Type	% Recovery	SPT (N)	% Moisture	SPT (N)	Well Installation	Log Notes
180.3	0	**(SP) SAND:** some gravel, some organics, brown, loose, moist; disturbed or reworked native		1	SS	70	9	13.9%	9		
179.3	1	- some cobbles, very loose, wet		2	SS	40	4	16.7%	4		
				3	SS	70	3	28.5%	3		
178.0	2.29	**(SW) SAND:** some silt, trace clay, trace gravel, brown, non-cohesive, wet, loose.	178 2.29	4	SS	40	5	23.7%	5		
		- compact		5	SS	40	11	16.5%	11		
		- very dense	175.26 5.03	6	SS	10	100	13.2%	100		
175.3	5	Borehole terminated @ 5 mbgs due to target depth achieved.									Borehole terminated in sand. Borehole caving observed at 2.3 mbgs. Groundwater encountered at 0.9 mbgs. Standing water observed at 0.6 mbgs.

Elevation scale: 180.3, 179.8, 179.3, 178.8, 178.3, 177.8, 177.3, 176.8, 176.3, 175.8, 175.3, 174.8, 174.3, 173.8, 173.3, 172.8
Depth scale: 0, 0.5, 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5, 5, 5.5, 6, 6.5, 7

Atterberg Limits (%) LL PL PI — 25 50 75
Shear Strength Cu, kPa nat V. rem V. — 20 40 60 80
% Moisture — 25 50 75
SPT (N) — 20 40 60 80

GRAINSIZE DISTRIBUTION

SAMPLE	GRAVEL	SAND	SILT	CLAY
SS4	7	65	19	9

Logged By: RG **Input By:** MH

Peterborough, Barrie, Oshawa, Kingston, Ottawa

CAMBIUM

Client: Brittany Lowry
Contractor: Ontario Soil Drilling
Project No.: 19210-001
Location: 65 Nettleton Drive, Penetanguishene

Project Name: 65 Nettleton Drive, Penetanguishene
Method: Track Mounted Solid Stem Auger
Elevation: 180.7 mASL
UTM: 17T **N:** 4957560 **E:** 583783

Log of Borehole: BH102-24

Date Completed: January 12, 2024

Elevation (m)	Depth (m)	Description	Elevation / Depth	Number	Type	% Recovery	SPT (N)	% Moisture	Log Notes
180.7	0	**(SP) SAND:** some gravel, some silt, brown, non-cohesive, dense, moist; disturbed or reworked native		1	SS	20	50	14.2%	Groundwater level measured in monitoring well at a depth of -0.69 mbgs (181.39 mASL) on February 16, 2024
		- very loose		2	SS	40	2	26.9%	
			179.18 / 1.52						
179.2	1.5	**(SW) SAND:** some gravel, trace silt, trace clay; brown, non-cohesive, wet, dense		3	SS	30	50	16.0%	
		- compact		4	SS	40	20	14.5%	
				5	SS	60	29	12.2%	
		- dense		6	SS	20	34	10.5%	
			175.67 / 5.03						
175.7	5	Borehole terminated @ 5 mbgs due to target depth achieved.							Borehole terminated in sand. Borehole caving observed at 3.4 mbgs. Groundwater encountered at 0.9 mbgs. Standing water observed at 0.3 mbgs.

Well Installation: Cap, Bentonite Plug, Riser, Sand Pack, PVC Screen, Cap

Atterberg Limits (%) LL PL PI; Shear Strength Cu, kPa nat V. rem V.

GRAINSIZE DISTRIBUTION SAMPLE	GRAVEL	SAND	SILT	CLAY
SS3	10	77	9	4

Logged By: RG **Input By:** MH

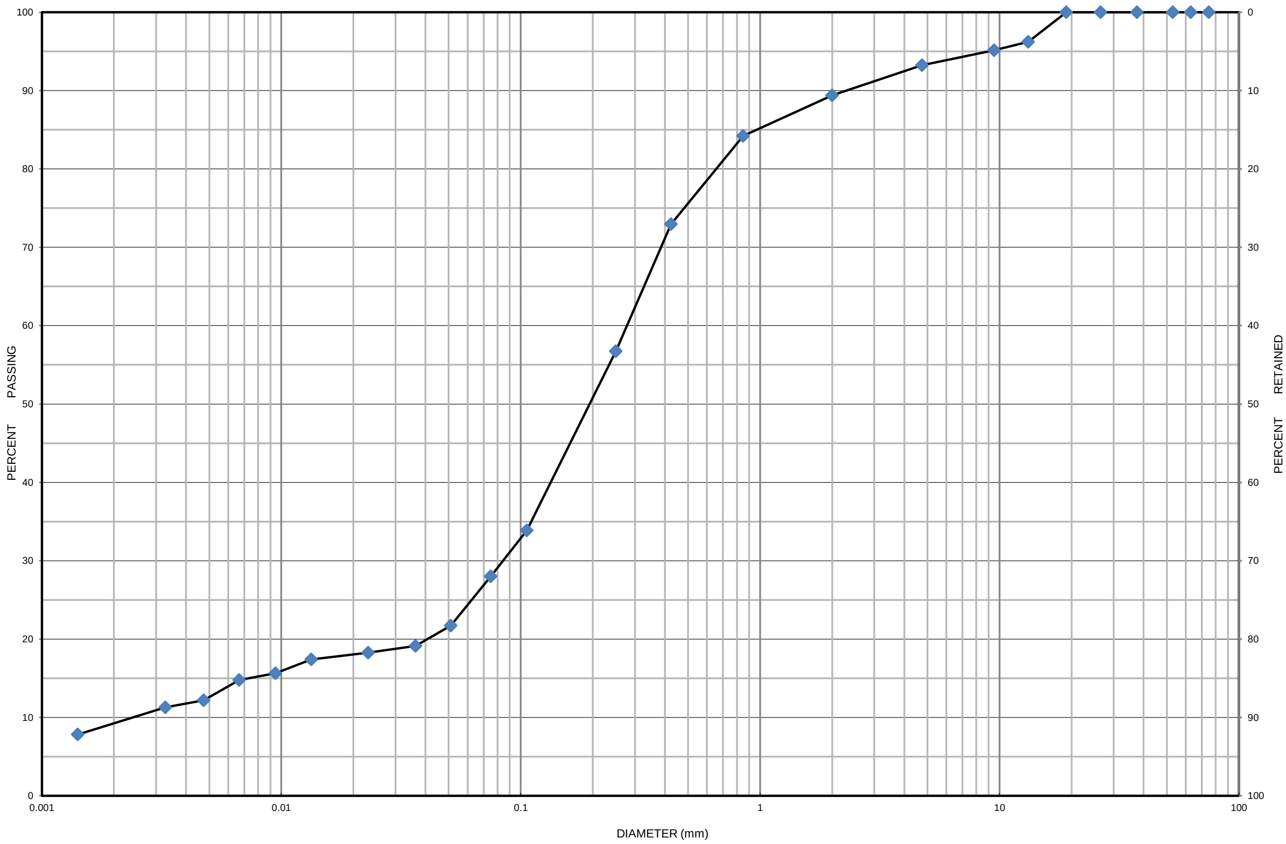
Appendix B

Physical Laboratory Testing Results

Grain Size Distribution Chart

Project Number:	*19210-001*	**Client:**	*Morgan Planning & Development Inc*		
Project Name:	*65 Nettleton Drive, Penetanguishene*				
Sample Date:	*January 12, 2024*	**Sampled By:**	*Rowan Galashan - Cambium Inc.*		
Location:	*BH 101-24 SS 4*	**Depth:**	*2.3 m to 2.7 m*	**Lab Sample No:**	*S-24-0152*

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 101-24	SS 4	2.3 m to 2.7 m	7	65	19	9	23.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Silt trace Clay trace Gravel	SM	0.2700	0.0840	0.0024	112.50	10.89

Additional information available upon request

Issued By: (Senior Project Manager)   *Date Issued:* *January 30, 2024*

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Form: L6V.2 - Grad.Hydo

Grain Size Distribution Chart

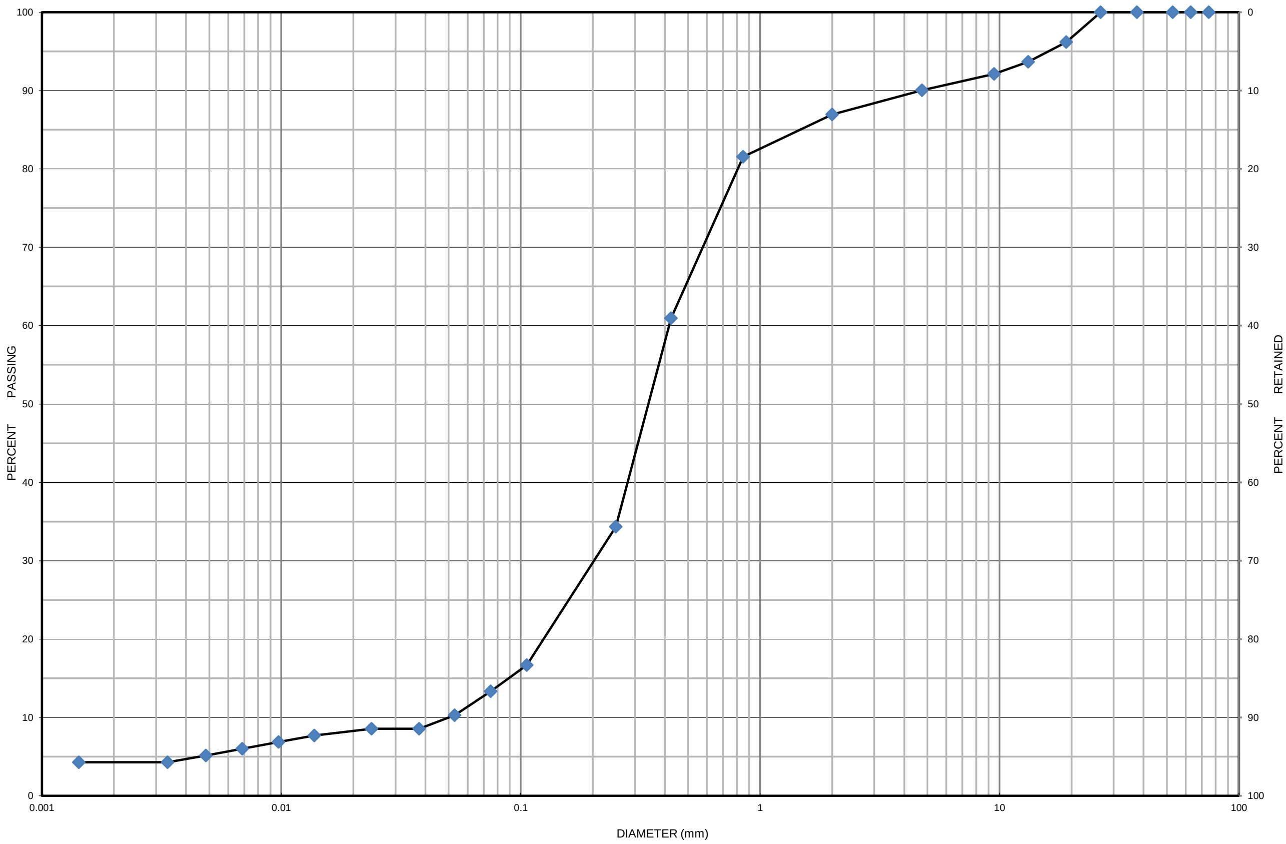
Project Number: *19210-001* **Client:** *Morgan Planning & Development Inc*

Project Name: *65 Nettleton Drive, Penetanguishene*

Sample Date: *January 12, 2024* **Sampled By:** *Rowan Galashan - Cambium Inc.*

Location: *BH 102-24 SS 3* **Depth:** *1.5 m to 2.1 m* **Lab Sample No:** *S-24-0153*

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 102-24	SS 3	1.5 m to 2.1 m	10	77	9	4	16.0

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Gravel trace Silt trace Clay	SM	0.410	0.200	0.050	8.20	1.95

Additional information available upon request

Issued By: (Senior Project Manager) *Date Issued:* *January 30, 2024*

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Moisture Content

Project Number:	19210-001	**Lab Number:**	S-24-0151
Project Name:	65 Nettleton Drive, Penetanguishene	**Date Tested:**	2024-01-25
Client:	Morgan Planning & Development Inc	**Tested By:**	A. Heffernan
Date Taken:	2024-01-12		

Borehole Number	Sample Number	Sample Depth (m)	Water Weight (g)	Water Content (%)	Additional Observations
101	1	0.00-0.61	41.8	13.9	
101	2	0.76-1.37	33.9	16.7	
101	3	1.52-2.13	79.5	28.5	
101	4	2.29-2.90	110.8	23.7	NR
101	5	3.05-3.66	40.1	16.5	
101	6	4.57-5.18	25.9	13.2	NR
102	1	0.00-0.61	36.9	14.2	
102	2	0.76-1.37	38.7	26.9	
102	3	1.52-2.13	58.3	16.0	NR
102	4	2.29-2.90	45.0	14.5	
102	5	3.05-3.66	39.9	12.2	
102	6	4.57-5.18	27.6	10.5	NR

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