Geotechnical Investigation - 153 King Avenue East, Newcastle, ON

2019-07-10

Prepared for:
Tim Welch Consulting Inc.

 Reference No.: 9231-001

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1.0 Introduction

Cambium Inc. (Cambium) was retained by Tim Welch Consulting Inc. (Client) to complete a geotechnical investigation in support of the proposed addition at 153 King Avenue East, Newcastle, Ontario (Site). The Site is currently vacant with a gravel driveway to King Avenue East. The proposed addition will connect to the retirement building at 165 King Avenue East, Newcastle, Ontario, which is a 2-storey building with parking spaces along the eastern property line. It is understood that the proposed addition will consist of a 4-storey building with parking on the eastern property line, maintaining the existing driveway to King Avenue East.

The geotechnical investigation was required to confirm the existing subsurface conditions, groundwater conditions, soil bearing capacity as input into the design and construction of the proposed development and pavement design recommendations. A Site Plan, including borehole locations, is included as Figure 1 of this report.

This report presents the methodology and findings of the geotechnical investigation at the Site and addresses requirements and constraints for the design and construction of the development.

2.0 Methodology

2.1 Borehole Investigation

A borehole investigation was conducted on June 18, 2019 to assess subsurface conditions at the Site. Four (4) boreholes, designated as boreholes BH101-19 through BH104-19, were drilled in accessible areas throughout the Site. The boreholes extended to dense glacial till soils at depths ranging between 9.8 meters below ground surface (mbgs) and 11.1 mbgs. GPS coordinates and elevations of each borehole were obtained using a Topcon real-time kinematic (RTK) unit. The elevation of the boreholes were surveyed relative to a site benchmark, identified as the top of the catchbasin on the south side of King Avenue East in front of the Site, which was assigned an elevation of 100.00 m. Borehole locations are shown in Figure 1.

Drilling and sampling was completed using a truck-mounted drill rig operating under the supervision of a Cambium technician. The boreholes were advanced to the sampling depths by means of continuous flight 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 in to 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 consistency of cohesive soils and relative density of non-cohesive soils. Soil samples were collected at approximately 0.75 m intervals in the upper 3.0 m and in 1.5 m intervals below 3.0 m. 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.

Dynamic cone penetration testing (DCPT) was completed from 6.6 mbgs to a maximum depth of 11.1 mbgs in boreholes BH102-19 and BH104-19, to further evaluate soil consistency/relative density at depth. In the DCPT, a 51 mm diameter, 60 degree Apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 305 mm, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results may be difficult because no soil samples are obtained through this method, and the penetration resistances are not necessarily equivalent to N values or undrained shear strengths, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which may affect SPT N values. In some deposits, soil adhesion to the drill rod assembly may affect DCPT results, and therefore should be taken into account in the geotechnical assessments.

One (1) borehole, BH104-19, was outfitted as a monitoring well to understand static groundwater conditions at the Site. Borehole logs are provided in Appendix A. Site soil and groundwater conditions are described and geotechnical recommendations are discussed in the following sections of this report.

2.2 Physical Laboratory Testing

Physical laboratory testing, consisting of three (3) particle distribution analyses (LS-702,705), was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters. Moisture content testing was completed on all soil samples from the boreholes. Results are presented in Appendix B and are discussed in Section 3.0.

3.0 Subsurface Conditions

The subsurface conditions at the Site generally consist of sandy silt topsoil overlying a 400 mm to 800 mm thick layer of dark to light brown silt in boreholes BH101-19 through BH103-19. Borehole BH104-19 encountered a gravelly sand underlying the topsoil, which is likely backfill from the nearby gravel driveway. The silty soils were encountered underlying the gravelly sand fill. At 4.0 mbgs the silty soils transitioned to a clayey silt material that was light grey in colour. Between 8.5 mbgs and 9.0 mbgs, a clayey silt till with some gravel and a trace to some sand was encountered to termination depths, which ranged from 9.8 mbgs to 11.1 mbgs. All soil types overlying the glacial till were loose to very loose based on SPT N values between 0 (weight of hammer pushed spoon through soil) and 9. The glacial till soils were compact to very dense based on SPT N values ranging from 23 to greater than 50. The water level upon completion of drilling was measured between 2.7 mbgs and 3.4 mbgs, with grey soils first observed between 3.0 mbgs and 5.0 mbgs. The borehole locations are shown on Figure 1and the individual soil units are described in detail below.

3.1 Topsoil

A 75 mm to 350 mm thick layer of topsoil, with an average thickness of 156 mm, was encountered at the surface of all boreholes. The topsoil was a dark brown sandy silt with frequent organics in the form of roots and rootlets.

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

3.2 Gravelly Sand

A 400 mm thick layer of light brown gravelly sand with a trace of silt was encountered beneath the topsoil in borehole BH104-19. The gravelly sand has a compact relative density based on an SPT N value of 14 and was moist at the time of the investigation with a natural moisture content of 9% based on laboratory testing.

3.3 Sandy Silt

Underlying the topsoil and gravelly sand was a layer of silty soils that ranged from silt with varying amounts of sand and clay to clayey sandy silt. A trace of organic material was observed within the upper 1.0 m of soil. The silt was dark brown when first encountered and transitioned to a light brown, sandy clayey silt with depth. The sandy silt layer extended to 5.6 mbgs in borehole BH102-19 and 4.0 mbgs in the other boreholes. The relative density of the silty soil was very loose to loose based on SPT N values ranging from 3 to 9. At the time of the investigation the soil was described as moist to wet with a natural moisture content ranging from 18% to 32%.

Laboratory particle size distribution analysis was completed on one (1) sample of the silt soils from the Site. The analysis results, based on the Uniform Soil Classification System (USCS) scale, are summarized in Table 1 below with full results provided in Appendix B.

Table 1 Particle Size Distribution Results – Sandy Clayey Silt

Borehole	Depth (m)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture
BH104-19 SS4	2.3 – 2.7	Clayey Sandy Silt	0	24	53	23	21.2

3.4 Clayey Silt

Underlying the sandy silt was a layer of grey clayey silt that was first encountered at 4.0 mbgs in boreholes BH101-19, BH103-19 and BH104-19 and at 5.6 mbgs in BH102-19. The clayey silt has a very soft to firm consistency based on SPT N values ranging from 1 to 6. At the time of the investigation the clayey silt was generally about the plastic limit to wetter than the plastic limit, but was also observed as much wetter than the plastic limit in borehole BH101-19 at 6.4 mbgs. The natural moisture content of the clayey silt ranges from 24% to 44% based on laboratory results.

Laboratory particle size distribution analysis was completed on one (1) samples of the clayey silt soils from the Site. The analysis results, based on the USCS scale, are summarized in Table 3 below with full results provided in Appendix B.

Table 2 Particle Size Distribution Results – Clayey Silt

Borehole	Depth (m)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture
BH103-19 SS6	4.6 – 5.0	Clayey Silt	0	7	63	30	27.9

3.5 Glacial Till

Grey sandy silt glacial till soils containing some gravel and a trace to some clay were encountered in boreholes BH101-19 and BH103-19 at depths of 7.1 mbgs and 8.6 mbgs, respectively. Some cobbles were also observed in borehole BH101-19 at 11.0 mbgs. SPT N values ranging from 28 to greater than 50 blows indicate a compact to very dense relative density, which increased with depth. The glacial till soil was described as wet to saturated with a natural moisture content between 8% and 14%.

DCPT testing was used to confirm the presence of the dense glacial till soils in the other two boreholes. Based on increases in DCPT values, which are similar to the SPT N values at depth, it is presumed that the compact to dense glacial till soils observed in boreholes BH101-19 and BH103-19 were encountered at approximately 8.1 mbgs and 8.8 mbgs in boreholes BH102-19 and BH104-19, respectively.

A laboratory particle size distribution analysis was completed on one (1) sample of the glacial till soil from the Site. The analysis results, based on the USCS scale, are summarized in Table 3 below with full results provided in Appendix B.

Table 3 Particle Size Distribution Results – Glacial Till

Borehole	Depth (m)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture
BH101-19 SS9	9.1 – 9.6	Sandy Silt, some Clay, trace Gravel	8	33	42	17	8.6

3.6 Bedrock

Bedrock was not encountered during the drilling investigation. All boreholes terminated in dense to very dense glacial till soils at depths between 9.8 mbgs and 11.1 mbgs. Based on well records and geologic mapping, bedrock is expected to be at depths of between 18 m and 24 m below the ground surface.

3.7 Groundwater

On completion of drilling, borehole caving (sloughing) was observed at 10.4 mbgs and 9.8 mbgs in boreholes BH101-19 and BH103-19, respectively and groundwater seepage was encountered between 2.7 mbgs and 3.4 mbgs. Grey soils were first observed at 4.0 mbgs, 4.8 mbgs, 3.0 mbgs and 4.0 mbgs in boreholes BH10-19 through BH104-19, indicating the long-term presence of groundwater. On June 26, 2019 the water level in the monitoring well in borehole BH104-19 was measured at 1.87 mbgs. Based on these observations, the groundwater table at the Site was at approximately 1.9 mbgs shortly after drilling, which was during a seasonally wetter time of year.

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

4.0 Geotechnical Considerations

The following recommendations are based on 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 Site Preparation

It is understood that the proposed development will consist of a 4-storey brick building without a basement, which will be tied into the existing 2-storey building to the south. Given the very loose/soft soil conditions observed to 8.0 m depth and the depth of the groundwater table at 1.9 mbgs (seasonally) and likely closer to 3.0 mbgs based on soil colour, shallow foundation options including conventional strip and spread footings placed just below frost depth or insulated footings are not recommended for this Site.

The recommended foundation options at this Site include caissons or H-piles terminating in the dense to very dense glacial till soils observed below 9.5 m depth.

Existing topsoil, fill, asphalt, and any organic material identified or encountered shall be excavated and removed from beneath any areas of the Site to be developed.

4.2 Frost Penetration

Based on climate data and design charts, the maximum frost penetration depth below pavement at the Site is estimated at 1.2 m. Footings for the proposed structure should be placed at or below this depth for frost protection.

It is assumed that any pavement structure thickness will be less than 1.2 m; therefore, grading and drainage are important for good pavement performance and life expectancy. Any utilities should be located below this depth of be appropriately insulated.

4.3 Excavations

Temporary excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). The generally very loose to loose sandy silt soils and the soft to firm clayey silt soils encountered above the groundwater table may be classified as Type 4 soils in accordance with the OHSA, with unsupported sides no steeper than 3H:1V (3 Horizontal to 1 Vertical). Excavations below the groundwater table in these soils are not recommended without advance dewatering given the very loose to loose / soft to firm density / consistency..

Excavation side slopes should be protected from exposure to precipitation, associated ground surface runoff, 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).

4.4 Dewatering

Groundwater was encountered on completion of drilling at depths of 2.7 mbgs to 3.4 mbgs in all boreholes. Caving (sloughing) of the borehole walls upon completion of drilling was observed near termination depth in boreholes BH101-19 and BH103-19 at 10.4 mbgs and 9.8 mbgs. Grey and wet to saturated soils were generally observed at approximately 4.0 m depth in all of the boreholes. One (1) monitoring well was installed in borehole BH104-19 to measure the static groundwater level at the Site. On June 26, 2019 the water level measurements for the groundwater table depth was 1.87 mbgs.

Assuming that there are no plans for underground parking and excavations will not be required below 3.0 mbgs, no significant groundwater seepage in anticipated. Minor groundwater seepage should be controllable with filtered sumps and pumps. Registration on the Environmental Activity and Sector Registry (EASR) and a Permit to Take Water (PTTW) will likely not be required from the Ministry of the Environment and Climate Change (MOECC) as pumping rates are not expected to exceed 50,000 L/day.

It is recommended that the Client measure the static groundwater levels at the Site during different seasons to determine the driest time of year for construction. Cambium is able to provide this service, using the existing monitoring well on site, and can provide a quote to the Client, if required.

It should be noted that the groundwater table is influenced seasonal fluctuations and major precipitation events.

4.5 Backfill and Compaction

Excavated native sandy silt 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. The gravelly sand encountered in borehole BH104-19 may be stockpiled and tested for reuse as Granular B material. Some moisture content adjustments may be required depending upon seasonal conditions. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality.

Any engineered fill below structures shall be placed in lifts appropriate to the type of compaction equipment used, and be compacted to a minimum of 100% of standard Proctor maximum dry density (SPMDD), confirmed by nuclear densometer testing. Imported material for engineered fill should consist of clean, non-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.

Trench backfill and cover for utility trenches should consist of free-draining material meeting the specifications of OPSS Granular B or an approved equivalent and should be compacted to 98 percent of SPMDD, taking care to keep heavy compaction equipment from damaging the utility.

Placement of engineered fill should be verified by onsite compaction tests during construction.

4.6 Foundation Design

A deep foundation system is recommended given the scale of the proposed addition, the high groundwater table, and the loose/soft silt and clayey silt soils extending to significant depths at the Site. Such systems include driven steel H-piles or concrete caissons, extending into the very dense glacial till soils. These options are outlined in the subsections below.

4.6.1 Steel H-Pile Foundations

Steel H-piles drive to practical refusal in the very dense glacial till soils are appropriate to support the proposed structure. Some cobbles were not encountered in the till soils at 11 m depth in borehole BH101-19, otherwise in the subgrade soils during the drilling investigation were relatively fine grained. Advancement of the piles should be relatively straightforward; however, reinforced driving shoes may be required to protect the pile tips.

4.6.1.1 Axial Geotechnical Resistance

A minimum pile size of HP 310x110 is recommended give the depth to dense to very dense till soils and the anticipated structure loads. A factored axial resistance at ULS of 650 kN may be used for H-piles driven to practical refusal in very dense glacial till. An axial resistance of 500 kN at SLS is appropriate for HP310x110 piles founded in very dense glacial till.

Dense to very dense till was generally found below 9.5 mbgs. Based on the SPT and DCPT penetration resistances, pile refusal in very dense till would likely be another few metres below those depths.

Pile driving termination of set criteria will be dependent on the pile driving hammer, type, helmet and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. The driving criteria should be selected to ensure that the piles are not overdrive to avoid possible damage to the piles.

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

4.6.1.2 Resistance to Lateral Loads

Lateral loadings can be fully or partially resisted by the use of battered H-Piles. Otherwise, the resistance to lateral loading can be derived through passive earth pressures developed in the soils in which the piles are embedded. The depth to very dense glacial till soils varies from 9.1 mbgs to 10.6 mbgs. Lateral pile resistance may be considered in accordance with Section 6.4 of the Canadian Highway Bridge Design Code (CHBDC).

For lateral soil-pile interaction analysis, the horizontal subgrade reaction (lateral spring parameters) to the pile may be calculated from the following expression for cohesionless soils at the site:

$$k_s = n_h(z/d)$$

where k_s = Coefficient of horizontal subgrade reaction
n_h = Constant of horizontal subgrade reaction
d = pile width (310 mm for HP310x110)
z = depth

Where the soil is primarily cohesive, the coefficient of horizontal subgrade reaction and ultimate lateral resistance can be estimated from:

$$ks = 67\ Su/d$$

where Su = Undrained shear strength (kPa)

The values of n_h and Su for the various soil types encountered in the borehole investigation are summarized in Table 4.

Table 4 Soil Coefficients for Lateral Loads

Soil Type and Density	N_h (kN/m^3)	Su (kPa)
Very loose to loose sandy silt	1,500	
Very soft to firm clayey silt	-	30
Dense to very dense sandy silt glacial till	12,000	

4.6.1.3 Frost Protection

Pile caps should be provided with 1.2 m of soil cover or the equivalent insulation for frost protection.

4.6.2 End-Bearing Caissons

Concrete caissons extending into the very dense glacial till soils a minimum of 500 mm are also appropriate to support the structure and floor slab. The caissons can be designed for a bearing capacity of 1,300 kPa (ULS) and 1,000 kPa at SLS, conservatively assuming the caissons will be end bearing only given the very loose to loose density of the subgrade soils to significant depth. The caissons should be spaced no closer than three (3) times the diameter on centre.

In general, greater capacity could be obtained by increasing the depth of the caisson.

The value of a Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (typical) and a Resistance Factor of 0.4 (typical). The geotechnical axial reaction at SLS is considering maximum 25 mm of settlement.

The resistance to lateral loading developed by the soils in front of the caissons and the reductions due to group effects may be determined as similar to the driven H-Pile.

During the installation, a temporary liner must be installed to permit hand cleaning, if necessary, and to seal off any caving or groundwater seepage. A positive head of concrete inside the temporary steel liner must be maintained during withdrawal of the liner to prevent “necking” of the caissons.

4.7 Floor Slab

A slab on grade construction method may be not achievable due to the presence of significant compressible clayey silt deposits. Therefore, a structural concrete slab should be supported by the pile caps of the driven H-Piles or concrete caissons. Due to the relatively shallow groundwater level (1.87 mbgs), it is recommended that a moisture barrier consisting of a minimum thickness of 150 mm of 19mm clear stone should be placed below the slab. Consideration should be given to the use of geo synthetic fabric (geotextile) and mesh (geogrid) for the construction of the structural slab. To prevent migration of the fill materials from contaminating the moisture barrier, a geotextile fabric should be placed below the 19 mm clear stone.

4.8 Subdrainage

Given the groundwater conditions, perimeter subdrains consisting of geotextile wrapped perforated pipes set in a trench of clear stone are recommended around the building perimeter below the floor slab. Subdrains are also recommended beneath the floor slabs or the slab could be set on a 300 mm thick layer of crushed clear stone hydraulically connected to the perimeter subdrains.

4.9 Buried Utilities

Trench excavations will generally be within the very loose to loose silt soils, which can be excavated with side slopes no steeper than 3H:1V above the groundwater table. If utilities are to extend deeper than the groundwater table, the work should be completed while the Site is dewatered with filtered sumps and pumps. It is not recommended to excavate beyond approximately 3.0 mbgs where the groundwater table exists without an advanced dewatering system in place.

The bedding and cover material for any services should consist of OPSS 1010-3 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 95% of SPMDD, taking care not to damage the utility pipes during compaction. If groundwater is present during placement of bedding material for utilities, then 19 mm crushed clear stone shall be used in place of granular material to ensure adequate compaction under wet conditions.

4.10 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”. Based on the explored soil properties and in accordance with Table 4.1.8.4.A of the OBC (2006), it is recommended that Site Class “D” (stiff soil) be applied for structural design at the Site.

4.11 Pavement Design

The performance of the pavement is dependent upon proper subgrade preparation. All topsoil and organic materials should be removed to expose competent native material. The subgrade should be proof rolled and inspected by a Geotechnical Engineer. Any areas where rutting or appreciable deflection is noted should be subexcavated and replaced with suitable engineered fill as approved by geotechnical personnel and compacted to at least 98% of SPMDD.

The recommended minimum pavement structure design has been developed for two (2) traffic loading scenarios; light duty and heavy duty. The heavy duty design is appropriate for areas where some truck/bus traffic is anticipated while the light duty design is appropriate for areas where no truck/bus traffic is anticipated. A thicker layer of granular subbase has been recommended given the poor density of the subgrade soils. The recommended minimum pavement structure is provided in Table 5.

Table 5 Recommended Minimum Pavement Structure – Parking Area

Pavement Layer	Light Duty	Heavy Duty
Surface Course Asphalt	40 mm HL3 or HL4	40 mm HL3 or HL4
Binder Course Asphalt	50 mm HL8	90 mm HL8 (two lifts)
Granular Base	150 mm OPSS 1010 Granular A	150 mm OPSS 1010 Granular A
Granular Subbase	400 mm OPSS 1010 Granular B	400 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 150 mm thick lifts and compacted to at least 98% of SPMDD (ASTM D698) standard. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing.

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

4.12 Design Review and Inspections

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

We 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.

5.0 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 reviewer at 705 742-7900 ext. 336 or 332.

Respectfully submitted,

CAMBIUM INC.

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Appended Figures

Appendix A

Borehole Logs

Appendix B

Physical Laboratory Data