GEOTECHNICAL INVESTIGATION
PROPOSED SITE SERVICING
TRENT RESEARCH AND INNOVATION PARK
PETERBOROUGH, ONTARIO

For:
City of Peterborough
500 George Street North
Peterborough, Ontario
K9H 3R9

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Ref. No. D16147

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1.0 INTRODUCTION

Downunder Geotechnical Limited was retained by the City of Peterborough, c/o D.M. Wills Associates Ltd., to conduct a geotechnical investigation for the site servicing of the proposed Trent Research and Innovation Park on Pioneer Road in Peterborough, Ontario.

The purpose of the investigation was to obtain information about the subsurface conditions at the site, and based on our interpretation of the data obtained provide recommendations on the geotechnical aspects of the proposed construction.

The recommendations provided are for the designers only and not to be relied upon by contractors bidding on this project. It is recommended that contractors bidding on this project review the factual data with a qualified geotechnical engineer and develop their own opinion of the subsurface soil and groundwater conditions at the site as well as the constructability concerns and details of the project. This report was prepared with the assumption that the design will be in accordance with all applicable standards and codes, regulations, and good engineering practice will be exercised. Further, the recommendations and opinions in this report are applicable only to the proposed project as described herein.

Any questions concerning the geotechnical aspects of the proposed project should be directed to Downunder Geotechnical Limited for further elaboration and/or clarification.

2.0 EXISTING SITE CONDITION

We understand that the Trent Research and Innovation Park project will consist of redeveloping the site to construct research buildings with internal roads, underground services and stormwater management ponds.

The existing site is composed of agricultural lands. Existing grades vary across the borehole locations at the site from elevation 219.1 to 233.7m. The site is bounded to the south by Pioneer Road, to the north by agricultural lands, to the east by agricultural lands and Douro 9th Line, and to the west by Trent University lands.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation was carried out between December 15 and 21, 2016. The fieldwork consisted of advancing twenty-two boreholes (Boreholes 1 to 22) to depths of 1.5 to 10.5m below existing ground surface. The approximate borehole locations are presented on Figure No. 1.

The boreholes were advanced with a track mounted direct push rig owned and operated by Strong Soil Search, equipped with standard soil sampling equipment, under the full-time supervision of an experienced technician. Soil samples were obtained by employing the Standard Penetration Test, in accordance with ASTM D1586. The Standard Penetration Test consists of freely dropping a 63 kilogram hammer a vertical distance of 0.76m to drive a 51 mm outside diameter split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m was recorded and is denoted as ‘N’-values. These recorded ‘N’-values give an indication of the consistency or compactness of the soil and are recorded on the Record of Borehole sheets in the Appendix.

Auger refusal was obtained in nineteen of the boreholes on inferred bedrock. Bedrock coring has not been carried out to date.

The soil samples were stored in air tight containers to minimize moisture loss and transported to our office for further examination and classification. The samples were visually inspected and logged for classification. A laboratory testing programme consisting of natural moisture content determinations, three Standard Proctor tests, five sulphate analyses, six unit weight determinations and ten grain size analyses was performed on representative soil samples. The results of the moisture content determinations and grain size analyses are presented on the Record of Borehole sheets and in the Appendix.

The borehole elevations were provided by D.M. Wills Associates Ltd. We understand that the elevations are referenced to the geodetic datum.

4.0 SUBSURFACE CONDITIONS

Descriptions of the sub-surface conditions encountered in the boreholes are presented on the Record of Borehole sheets in the Appendix. The following paragraphs are intended to supplement and complement these data.

4.1 Topsoil

All boreholes encountered topsoil at surface. The following table summarizes the topsoil thicknesses.

Borehole No.	Topsoil Thickness	Borehole No.	Topsoil Thickness
1	20cm	12	20cm
2	15cm	13	23cm
3	17cm	14	20cm
4	20cm	15	15cm
5	20cm	16	15cm
6	23cm	17	23cm
7	17cm	18	15cm
8	15cm	19	17cm
9	17cm	20	13cm
10	20cm	21	23cm
11	25cm	22	17cm

Measured moisture contents of the topsoil range from 20 to 26%.

4.2 Organic Stained Subgrade

Boreholes 11 to 13 encountered an organic stained sandy silt below the topsoil to a depth of about 0.7m below grade. This organic stained layer may be present elsewhere at the site due to potential past ploughing of the area.

Measured 'N'-values range from 6 to 9 blows/0.3m indicating a loose.

4.3 Silty Sand to Sandy Silt Glacial Till

Below the topsoil and organic stained sandy silt, all boreholes encountered a brown to grey, silty sand to sandy silt glacial till to the termination depth of the boreholes at 1.5 to 10.5m.

The glacial till is a well graded, heterogeneous mixture of silty sand to sandy silt with varying amounts of gravel and trace clay. Due to its nature of formation the glacial till may contain cobbles and boulders. Grinding of the augers on inferred cobbles and boulders was noted in several boreholes during drilling.

Measured 'N'-values range from 4 to greater than 50 blows/0.3m, indicating a compactness of loose to very dense, but typically compact to very dense. The loose soils are generally in the upper 0.7m with some areas as deep as 2.1m below grade. Measured moisture contents ranged from 3 to 17%, but are typically 6 to 9%.

Grain size analyses were carried out on nine representative samples of the glacial till and three Standard Proctor tests. The results are presented in the Appendix, on the Record of Borehole sheets and on the grain size distribution curves, and further summarized below.

Grain Size Analyses		
	Gravel	8 to 31%
	Sand	29 to 41%
	Silt	26 to 55%
	Clay	2 to 13%
Standard Proctor		
	Maximum Dry Density	1861 to 2165 kg/m^3
	Optimum Moisture Content	7.7 to 10.2%
Bulk Unit Weight		21.4 to 24.1 kN/m^3

4.4 Sandy Silt

Interlayered within the glacial till in Borehole 9 a brown, wet, 2m thick sandy silt deposit was encountered at a depth of 3.5m below grade. This deposit has layers of sandy silt glacial till.

Measured 'N'-values were greater than 50 blows/0.3m, indicating a compactness of very dense. Moisture contents of 16 and 17% were measured.

A grain size analysis was carried out on a representative sample of the sandy silt. The results are presented in the Appendix on the Record of Borehole sheets and on the grain size distribution curves, and further summarized below.

Gravel	0%
Sand	19%
Silt	69%
Clay	12%

4.5 Inferred Bedrock

Nineteen of the boreholes at the site encountered auger refusal at depths of 1.5 to 7.0 m below grade. Inferred bedrock is assumed at these depths. The inferred surface of the bedrock at the borehole locations is summarized in the table below. The inferred bedrock contours are presented in Figure No.2. An additional geotechnical investigation is required to confirm the presence of bedrock by rock coring.

Borehole No.	Depth to Inferred Bedrock below grade (m)	Inferred Bedrock Elevation (m)
1	1.5	225.0
2	3.2	225.6
3	5.2	221.7
4	2.1	221.7
5	1.7	222.3
6	2.0	222.7
7	5.8	220.4
8	4.7	223.1
9	>6.7*	<219.4
10	3.1	218.0
11	3.2	217.4
12	3.3	217.1
13	2.6	217.1
14	2.9	216.2
15	3.1	217.1
16	2.4	216.7
17	2.4	216.9
18	>6.4*	<216.7
19	7.0	220.1
20	>10.5*	<223.2
21	4.0	219.8
22	3.2	220.4

*bedrock not encountered at termination depth of the borehole

From published geological mapping, the bedrock of the area consists of a limestone of the Verulam Formation.

4.6 Groundwater

Groundwater was encountered in seven of the boreholes during drilling. Groundwater levels were measured in open boreholes on completion and depth in the range of 1.2 to 5.8m, however these do not represent stabilized groundwater levels. Monitoring wells were installed in Boreholes 3, 8, 12, 15, 17 and 20.

The monitoring wells consisted of 50mm diameter PVC riser pipe and screen with above grade steel casing. The screen section consisted of 1.52 to 3.05m long No. 10 slotted PVC. Threaded points were installed at the bottom of each well, and all pipe sections were threaded. The annular space of the borehole around the screen was packed with clean silica sand. The upper section of the wells were completed with solid riser casing, with the annular space above the screen sealed with bentonite chips. Groundwater levels within the wells were measured over several weeks. Details of the monitoring well installations are presented on the Record of Borehole Sheets in the Appendix.

The following groundwater measurements were obtained and presented on the Record of Borehole sheets in the Appendix.

Borehole No.	Groundwater Level Measurements									
	Dec 22, 2016		Dec 28, 2016		Jan 5, 2017		Jan 11, 2017		Jan 26, 2017	
	Depth below grade	Elevation	Depth below grade	Elevation	Depth below grade	Elevation	Depth below grade	Elevation	Depth below grade	Elevation
3	3.0m	224.0	2.6m	224.3	1.2m	225.7	0.8m	226.2	0m	226.9
8	3.6m	224.2	3.5m	224.2	1.9m	225.9	1.7m	226.1	-	-
12	Dry	-	2.9m	217.5	Dry	-	2.3m	218.1	-	-
15	1.2m	219.0	0.3m	219.9	0.1m	220.1	0m	220.1	-	-
17	Dry	-	Dry	-	0.8m	218.5	0.4m	218.9	-	-
20	Dry	-	Dry	-	Dry	-	7.6m	226.3	-	-

It should be noted that the groundwater table may fluctuate seasonally and groundwater depths are based on short term monitoring.

5.0 DISCUSSION AND RECOMMENDATIONS

The following table summarizes the areas the boreholes were advanced within and the proposed sewer inverts.

Borehole No.	Proposed Sewer Invert Depth below Grade	Proposed Construction
1 to 14, 16, 21 and 22	4m	Sewers
19, 20	8m	Sewers
4 to 6	Stormwater Management Pond	
15, 17, 18		

These recommendations and comments are based on factual information and are intended only for use by the design engineers. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. Subsurface soil and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The anticipated construction conditions are discussed, but only to the extent that they may influence design decisions. Construction methods discussed express our opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect upon the construction.

5.1 Site Grading

We assume that the proposed grades will generally follow the existing grades of the site, with minor cutting of the high areas and filling of any low areas.

All infilling should be placed as engineered fill. The following placement procedure is recommended.

- The area to receive the engineered fill should be stripped of the existing topsoil and organic stained subgrade due to ploughing activities should be excavated from the planned cut and fill areas to expose the inorganic native subgrade. After stripping, the entire area must be inspected and approved by the geotechnical engineer. Spongy, wet or soft/loose spots, organics and any other deleterious materials should be sub-excavated to stable subgrade and replaced with compactable approved soil, compatible with subgrade conditions, as directed by the geotechnical engineer.

- The fill material should be placed in thin layers not exceeding approximately 200 mm when loose. Oversize particles (cobbles and boulders) larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used, to at least 98% of its Standard Proctor Maximum Dry Density (SPMDD).

- The on-site inorganic soils are generally acceptable for use as engineered fill, provided they are not mixed with the overlying topsoil and any organic inclusions are removed. Depending on the construction season, the on-site soils may require some reconditioning, wetting or drying. During freezing conditions the native tills must have a moisture content within 2% of the optimum value for compaction, otherwise the soils will not achieve the desired compaction as the moisture content of the soil cannot be reconditioned in freezing conditions.

- Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) are necessary for the construction of a certifiable engineered fill. Compaction procedures and efficiency should be monitored by a qualified geotechnical technician.

- The engineered fill or subgrade must not be frozen or contain ice and frozen material, and should be placed at a moisture content within 2 % of the optimum value for compaction. The engineered fill should not be performed during winter months when freezing ambient temperatures occur persistently or intermittently.

- Site activities may be carried out in the winter months and periods of cold weather. The following procedures are recommended for the placement of backfill during cold weather conditions:

    - The fill placement must be inspected by qualified field personnel on a full time basis under the supervision of a geotechnical engineer, with the authority to stop the fill operations at any time when conditions are considered to be unfavourable.
    - The intended area of fill must be clearly identified in the field prior to commencing the work.
    - When temperatures are between 0^0C and -5^0C, the fill material must be comprised of OPSS Granular 'B' - Type I or II, or other approved equivalent granular material. The engineered fill must be compacted in 200 mm thick loose lifts to a minimum of 98% of SPMDD.
    - Imported fill containing ice, snow or any frozen material cannot be accepted for use and should be returned to the source.
    - Overnight frost penetration into the fill mantle below proposed roadways or other structures must be prevented by using insulation blankets. Alternatively, any fill exposed and allowed to freeze must be completely removed prior to placing subsequent lifts. Breaking the frost in situ must not be accepted. Following removal of frozen materials the remaining exposed surface must be compacted to a minimum of 98% of SPMDD prior to placement of new fill.
    - During periods of cold where ambient temperatures are -5° C or less, placement of engineered fill shall stop and the existing fill materials must be protected from frost penetration.

  It should be noted that the placement of engineered fill materials during cold weather conditions requires extra effort beyond that typical in better climatic conditions. At any time where conditions are deemed unfavourable, the engineered fill operation must be suspended.

Comments on the subgrade infiltration are provided under a separate report by D.M. Wills Associates Ltd.

5.2 Excavation

We understand that excavations for the proposed sanitary sewer will range from about 4 to 8m below grade. Based on available information we assume the following excavations will be required at the borehole locations. The inferred bedrock surface is also included for comparison purposes.

Borehole No.	Assumed Sewer Invert below Grade	Inferred Bedrock Surface below Grade	Borehole No.	Assumed Sewer Invert below Grade	Inferred Bedrock Surface below Grade
1	4m	1.5m	12	4m	3.3m
2	4m	3.2m	13	4m	2.6m
3	4m	5.2m	14	4m	2.9m
7	4m	5.8m	16	4m	2.4m
8	4m	4.7m	19	8m	7.0m
9	4m	>6.7m	20	8m	>10.5m
10	4m	3.1m	21	4m	4.0m
11	4m	3.2m	22	4m	3.2m

The proposed sewer excavations will be advanced through the glacial till deposits at the borehole locations. Bedrock will likely be encountered at eleven of the sixteen boreholes advanced along the proposed sewer alignments. It is recommended that rock coring be carried out to confirm the presence of the bedrock. It should be noted that the bedrock surface may vary along the alignment and that the current overburden boreholes cannot accurately assess the bedrock depths based on auger refusal.

All excavations should be carried out in accordance with the Ontario Health and Safety Regulations (OHSA). The various soils encountered can be classified as per OHSA as follows.

Stratigraphy	Excavation Soil Type
Sandy Silt	
Above Groundwater Table	3
Below Groundwater Table	4
Silty Sand to Sandy Silt Glacial Till	
Above Groundwater Table	3
Below Groundwater Table	3

In Type 3 soils a side slope of 1H:1V is required from the base of the excavation. In Type 4 soils a side slope of 3H:1V or flatter is required from the base of the excavation. Dewatering of the Type 4 soils to about 0.6m below the excavation base prior to excavation can reduce the side slope requirements to a Type 3 soil. The side slopes should be regularly inspected for instability and flattened if there are signs of erosion or instability. Excavations must be carried out as per the Occupational Health and Safety Act and Regulations for Construction Projects.

Cobbles and boulders should be expected within the glacial till.

Where space is restricted or side slopes cannot be constructed, temporary shoring may be used. Excavations using shoring may be designed using the soil parameters in Section 5.4.

Stockpiles of excavated materials should be kept at least 3m from the crest of the excavation to reduce the potential for slope instability. Care should also be taken to avoid overloading of any underground services/structures by stockpiles.

Excavation within the bedrock may require use of line drilling and/or hoe ramming. Blasting may be required as the quality of the rock is currently unknown. Chemical fracturing may also be considered as an alternative to explosives.

Testing for off-site soil disposal was not part of the scope of work for this project. Soil testing will be required prior to disposal.

5.3 Dewatering

Based on the subsurface conditions encountered at the boreholes, the groundwater table varies across the site from near ground surface to about 5 to 6m below grade. Excavations below the groundwater table within the glacial till should be manageable by pumping with temporary pumps. Excavations within the sandy silt below the groundwater table are not anticipated. Details on dewatering methods, quantities and permit to take water requirements are presented under a separate report by D.M. Wills Associates Ltd.

5.4 Shoring Parameters

Vertical shoring may be carried out at the site. The following soil parameters can be used for design purposes.

Soil Type	Bulk Unit Weight (kN/m^3)	Friction Angle	Lateral Earth Pressure Coefficients		
			K_a	K_0	K_p
Sandy Silt	20.0	30^0	0.33	0.50	3.33
Silty Sand to Sandy Silt Glacial Till	22.0	35^0	0.27	0.50	3.70

The groundwater table should be assumed to be at ground surface for design purposes. Where minor horizontal ground movements are acceptable the use of the active earth pressure coefficient (K_a) is suitable. For sensitive areas where horizontal ground movements must be minimized the at-rest earth pressure (K_0) coefficient should be used.

5.5 Sewer Bedding and Backfill

Bedding

The boreholes show that the service pipes will be laid predominantly within competent soil deposits provided that they are not disturbed during excavation and/or by groundwater seepage, or within the inferred limestone bedrock. The recommended minimum thickness of granular bedding for normal Class 'B' Type of bedding (i.e., compacted granular bedding material – OPSD-802) below the invert is 150 mm. The thickness of the bedding may, however, have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. Clear stone and high performance bedding should not be used as bedding material. Pipe cover materials should consist of compacted granular fill, such as Granular 'A' or 'B' Type I or II.

Backfill

Based on the visual and tactile examination of the soil samples, the inorganic on-site excavated soils could be re-used as backfill in service trenches. The moisture contents at the time of construction should be at or near optimum. The backfill should be placed in maximum 200 mm thick layers at or near (+ 2%) their optimum moisture content, and each layer should be compacted to at least 95% SPMDD. This value should be increased to at least 98% SPMDD within 0.6 m of the road subgrade surface.

The excavated soils may require reconditioning (e.g., wetting or drying) prior to reuse. Unsuitable materials such as organic soils, boulders, cobbles, frozen soils, etc., should not be used for backfilling. Clear stone and high performance bedding should not be used as backfill material.

We recommend that frost tapers be provided at backfilled trenches to ensure gradual transition from the frost-free materials to the frost susceptible natural soil, otherwise differential frost heaving may occur. Frost taper would not be necessary if the backfill material can be matched within the frost zone (i.e. within about 1.6 m depth below the pavement surface) with subgrade-type material.

5.6 Pavement Design

The existing topsoil and organic-stained subgrade must be removed prior to road base preparation. Using good engineering and construction practice, the following minimum pavement structure may be used:

Pavement Structure	Compaction	Heavy Truck Access
HL-3 Asphaltic Concrete HDBC Asphaltic Concrete	97 % Marshall Density	50mm 100mm
Granular 'A' Base	100 %	150mm
Granular 'B' Type II Sub-base	100 %	300mm

NOTE: HL-3 and HDBC asphaltic concrete to conform to Ontario Provincial Standard Specifications (OPSS) Form 310 and 1150.

To ensure the longevity of the pavement, the subgrade should be crowned and graded with a minimum 1 to 2% slope and the roadbed should be well drained at all times. We recommend that full-length perforated sub-drain pipes of 150 mm diameter be installed on both sides of the roadway for urban roadways or extending the granular base and sub-base to the full shoulder width to discharge to ditching for rural roadways. The sub-drain pipes should be surrounded by 20 mm size clear stone drainage zone of minimum 150 mm thickness, which should have non-woven geotextile (non-woven geotextile, with FOS of 75 – 150 µm) wraparound to minimize infiltration of fines in pipes which would reduce their effectiveness. All subdrains should be provided with a frost free outlet.

The placing, spreading and rolling of the asphalt should be in accordance with OPSS Form 310, or equivalent.

Construction traffic over exposed subgrade materials should be minimized, and temporary construction hauling routes should be established. If these routes coincide with future paved areas, adequately reinforced haul roads (increased thickness of granular base, use of geo-fabrics, etc.) should be constructed to reduce disturbance to the subgrade soils. These provisions are particularly important if the construction is scheduled during wet and cold seasons.

The subgrade should be adequately prepared to receive the sub-base course. Any disturbed, wet, frozen, organic and deleterious subgrade materials should be removed and the top of the subgrade should then be inspected and approved, by proof-rolling, by qualified geotechnical personnel. Cavities created by the removal of unsuitable materials should be backfilled with approved, inorganic fill materials similar to the existing subgrade material. All new fill should be placed in maximum 200 mm loose lifts within + 2 % of its optimum moisture content, and each lift compacted with suitable equipment to minimum 95 % SPMDD, before placing the next lift.

The uppermost zones of the roadfill, within 600 mm of the roadbed, should be compacted to minimum 98 % SPMDD. If construction of the roadfill is carried out in wet weather, the thickness of the sub-base course should be increased.

Special attention should be paid to proper grading of the subgrade surface. Depressions and undulations should be eliminated and, to permit quick drainage, the subgrade surface should be sloped towards ditches, sub-drains and/or catch-basins.

5.7 Stormwater Management Ponds

We understand that two stormwater management ponds are proposed at the site, one in the area of Boreholes 4 to 6, and one in the area of Boreholes 15, 17 and 18. The assumed pond invert and inferred bedrock surface are noted in the following table.

Borehole No.	Assumed Pond Invert below Grade	Assumed Pond Invert Elevation	Inferred Bedrock Surface below Grade	Inferred Bedrock Elevation
Pond A				
4	1.8m	222.7m	2.1m	221.7m
5	1.3m	222.7m	1.7m	222.3m
6	2m	222.7m	2.0m	222.7m
Pond B				
15	2m	218.2m	3.1m	217.1m
17	1.1m	218.2m	2.4m	216.9m
18	4.9m	218.2m	>6.4m	<216.7m

The recommendations below are based on assumed pond invert elevations. If the pond inverts are different than above this report will need to be revised.

Pond A will be excavated into the silty sand to sandy silt glacial till and the invert may be in contact with the inferred bedrock surface. The invert of the pond should be kept as high as possible to avoid the bedrock surface which may vary across the pond area.

Pond B will be excavated into the silty sand to sandy silt glacial till. Provided the pond invert is above Elevation 218m, the bedrock should not be encountered. The groundwater table in the area is at about Elevation 219 to 220m.

The pond slopes will be constructed in fill and may consist of the existing on-site inorganic soils. The moisture content of the fill at the time of construction should be at or near optimum. The fill should be placed in maximum 200 mm thick layers at or near (+ 2%) their optimum moisture content, and each layer should be compacted to at least 95% SPMDD. The excavated soils may require reconditioning (e.g., wetting or drying) prior to reuse. Unsuitable materials such as organic soils, boulders, cobbles, frozen soils, etc., should not be used for backfilling.

The glacial till or inferred bedrock are not suitable as an impermeable barrier to create a wet pond. If a wet pond is desirable then a compacted clay liner or geosynthetic clay liner is required. For a compacted clay liner, the clay must be imported and consist of a clay material and have an in-situ hydraulic conductivity of less than 10^{-9}m/s. The clay must have the following characteristics:

- Free of rock greater than 50m.
- Free of topsoil, organics and root matter.
- Percent Fines (0.075mm) $\geq$ 50% by dry weight.
- Clay content (0.002mm) $\geq$ 20% by dry weight.

- Material passing 19mm sieve size $\geq$ 90% by dry weight.
- Plasticity Index $\geq$ 20%.
- Liquid Limit $\geq$ 30%.

The base of the ponds will be below the groundwater table the clay liner will be under hydrostatic pressure. The clay liner will need to be thick enough to resist this upward force or weighted down to resist this upward force. For the above noted assumed pond inverts, the pond bases will have up to 20 kPa upward force. This will require a 2.1m thick clay liner to resist upward groundwater pressures, however this will also require excavation of the bedrock. Therefore the pond invert should either be infilled with 0.6m of rip rap (or similar material such as clear crushed stone, crushed aggregate, etc.) above a 1m thick clay liner, or the pond invert raised by 1m (requiring a 0.9m thick clay liner). A slope stability analysis should be carried out for the proposed ponds once the pond details are defined.

5.8 Subsurface Corrosion

Five samples of the glacial till were submitted for sulphates at Caduceon Environmental Laboratories. The results are included in the Appendix and summarized in the following table.

Sample No.	Sulphate (%)
BH 2 SS 2	<0.001
BH 8 SS 4	0.001
BH 9 SS 2	<0.001
BH 12 SS 4	0.002
BH 22 SS 3	0.002

The measured sulphate content of the glacial till will not cause sulphate attack on concrete. In accordance with Table 12 of CSA A23.1, no special requirements for cementing materials are needed for concrete in contact with the subgrade.

5.9 Construction Inspection and Monitoring Program

It is recommended that a programme of geotechnical/material inspection and testing be carried out during the construction phase of the project to confirm that the conditions exposed in the excavations are consistent with those encountered in the boreholes and the design assumptions, and to confirm that the various project specifications and materials requirements are being met.

If blasting is required for bedrock excavation, a construction vibration study should be carried out to determine the blasting induced vibration zone of influence to the west and noise and vibration monitoring requirements.

5.10 Additional Investigation

An additional geotechnical investigation is required to confirm the presence and quality of the bedrock across the site in order to allow the contractor to estimate excavation requirements.

Slope stability analysis of the stormwater management pond slopes should be carried to confirm stable slope inclinations once details of the pond are defined and bedrock is confirmed.

6.0 CLOSURE

The attached Report Limitations are an integral part of this report.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
12 February 2017
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PROVINCE OF ONTARIO

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REPORT LIMITATIONS

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information herein in no way reflects on the environmental aspects of the project. Subsurface and groundwater conditions beyond the testhole may differ from those encountered at the testhole location, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the testhole.

The design recommendations in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of fill may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

This report was prepared for the City of Peterborough and its consultants. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Downunder Geotechnical Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX