REPORT

Geotechnical Investigation

Proposed Residential Development 5957 and 5969 Fernbank Road
Ottawa, Ontario

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed residential development to be located at 5957 and 5969 Fernbank Road in Stittsville (Ottawa), Ontario.

The purpose of this geotechnical investigation was to assess the general subsurface conditions at the site by means of a limited number of test pits. Based on an interpretation of the factual information obtained, a general description of the soil, bedrock, and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared to construct a residential development to be located at 5957 and 5969 Fernbank Road in Stittsville (Ottawa), Ontario. The project limits for the proposed development are shown on Figure 1.

The following is known about the site and proposed development:

- The property is roughly rectangular in shape with a width and length of approximately 290 and 810 metres, respectively.
- The site has a relatively flat to gently sloping topography.
- The site primarily consists of undeveloped vacant land with some rows of trees. A residential house is located at 5957 Fernbank Road.
- The property is to be developed as a residential subdivision, with a mix of single family homes, semi-detached homes, and townhouse blocks.
- A City park will be located within the north central portion of the site (Block 203).
- A school (which is not part of this current investigation) will be located at the northeast corner of the site (Block 202).
- A stormwater management pond will be located at southeast corner of the site (Block 204).
- The proposed pond is irregular in shape and measures about 255 metres long by 75 to 120 metres wide in plan area.
- The elevation of the bottom of the pond will generally be at about 104.25 metres, with a submerged vegetation bench at an elevation of about 105.60 metres.
- The permanent water level in the pond will be at about elevation 105.75 metres and the 100-year ‘high’ water level will be at about elevation 107.34 metres.
- The side slopes of the pond will be inclined at 3 horizontal to 1 vertical to 5 horizontal to 1 vertical.
- Concrete inlet structures are proposed near the north-west and west-central portion of the pond, consisting of a bypass structure, a primary inlet structure, and separate bypass pipe and structure. The founding elevations for these structures are not known at this time.
- One concrete outlet structure is proposed at the south portion of the pond. This structure will consist of a 1,050 millimetre diameter pipe which will outlet to the existing pond.

Based on a review of published geological maps and previous subsurface investigations carried out in the vicinity of the site, the subsurface conditions on this site are expected to consist primarily of a deposit of glacial till over shallow bedrock. The depth to the bedrock surface is indicated to range between about 1 and 5 metres below the existing ground surface. Based on published geologic mapping, the bedrock is indicated to consist of limestone with dolomite interbeds of the Gull River Formation.

3.0 PROCEDURE

The fieldwork for the geotechnical investigation was carried out on December 14 and 15, 2017. During that time, 15 test pits (numbered 17-01 to 17-15, inclusive) were excavated at the approximate locations shown on the site plan, Figure 1.

The test pits were advanced using a track mounted hydraulic excavator supplied and operated by Cavanagh Construction of Ottawa, Ontario. The test pits were excavated to depths ranging from about 0.2 metres (refusal on the bedrock surface) to 4.5 metres below the existing ground surface.

The soils exposed on the sides of the test pits were classified by visual and tactile examination. Grab samples were obtained from the major soil strata encountered in the test pits. The groundwater seepage conditions were observed in the open test pits and the test pits were loosely backfilled upon completion of excavating and sampling.

The fieldwork was supervised by an experienced technician from our staff who logged the soils encountered and collected the soil samples. The soil samples obtained during the fieldwork were brought to our laboratory for further examination by the project engineer and for laboratory testing. The laboratory testing included grain size distribution testing and natural water content determination.

One soil sample from test pit 17-08 was submitted to Eurofins Environmental Testing Canada for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The test pit locations were selected, picketed in the field, and subsequently surveyed by Golder Associates personnel using precision GPS survey unit. The elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the test pits are shown on the Record of Test Pits in Table 1. The results of the basic chemical analyses on the selected soil sample from test pit 17-08 are provided in Appendix A.

In general, the subsurface conditions at this site consists of deposits of silts and sands over glacial till over limestone bedrock. Practical refusal to excavating was encountered in most of the test pits at depths varying from about 0.2 to 3.3 metres below the existing ground surface.

The following sections present a more detailed overview of the subsurface conditions encountered in the test pits.

4.2 Topsoil and Fill

A layer of fill exists at the ground surface at test pit 17-05. The fill at this location consists of sand containing brush (likely a windrow) with a thickness of about 0.5 metres.

A layer of topsoil exists at the ground surface, or below the fill, at all of the test pit locations, except test pit 17-09. The topsoil ranges from about 100 to 750 millimetres, but more typically between about 200 and 350 millimetres, in thickness.

A layer of rock fill was encountered below the topsoil at test pit 17-06. The rock fill consists of sand and gravel containing cobbles, and extends down to a depth of about 1.2 metres below the existing ground surface.

4.3 Sand and Silt

Discontinuous deposits of sand, silt, silty sand, and sandy silt, exist below the topsoil and fill in test pits 17-01, 17-03, 17-04, and 17-07 to 17-15. The sand and silt deposits were fully penetrated in most of the test pits, and were proven to extend to depths ranging from about 0.2 to 3.3 metres below the existing ground surface. These deposits were not fully penetrated in test pit 17-12, but were proven to extend to a depth of about 4.2 metres below the existing ground surface.

The measured water content on two samples of the sand and silt deposit ranges from about 10 to 11 percent.

The results of grain size distribution testing carried out on one sample of the sand and silt deposit are shown on Figure 2.

4.4 Glacial Till

A discontinuous deposit of glacial till was encountered below the topsoil and sand and silt layers in test pits 17-01, 17-02, 17-05, 17-06, 17-07, 17-11, 17-14, and 17-15. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a soil matrix of silty sand to sandy silt. The glacial till extends to depths ranging from about 0.9 to 4.5 metres below the existing ground surface.

The measured water content on three samples of the glacial till deposit range from about 9 to 22 percent.

The results of grain size distribution testing carried out on two samples of the glacial till are shown on Figure 3.

4.5 Bedrock

Refusal to excavating was encountered at all of the test pit locations, except test pits 17-12 and 17-14, at depths ranging from about 0.2 to 3.3 metres below the existing ground surface.

The following table summarizes the ground surface, depth to refusal, and refusal elevations as encountered at the test pit locations.

Test Pit Number	Ground Surface Elevation (m)	Bedrock Surface Depth (m)	Refusal Elevation (m)
17-01	111.1	0.9	110.2
17-02	109.5	1.1	108.4
17-03	111.8	1.3	110.5
17-04	112.0	1.3	110.7
17-05	113.6	1.6	112.0
17-06	111.3	1.5	109.8
17-07	108.9	1.8	107.1
17-08	109.5	0.4	109.1
17-09	111.6	0.2	111.4
17-10	110.6	1.9	108.7
17-11	109.7	2.1	107.6
17-12	107.2	> 4.2	< 103.0
17-13	105.9	3.3	102.6
17-14	106.8	> 4.5	< 102.3
17-15	108.6	1.6	107.0

4.6 Groundwater

Groundwater seepage and wet soil conditions were generally present at depths ranging from about 1.3 to 2.1 metres below the existing ground surface.

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.7 Corrosion

One soil sample from test pit 17-08 was submitted to Eurofins Environmental Testing Canada for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The results of this testing are provided in Appendix A and are summarized below.

Test Pit / Sample Number	Sample Depth (m)	Chloride (%)	SO_4 (%)	pH	Resistivity (Ohm-cm)
17-08 / 1	0.1 – 0.4	<0.002	0.02	7.28	9,090

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

Reference should be made to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

5.2 Site Grading

The subsurface conditions on this site generally consist of deposits of silts and sands over glacial till, underlain by bedrock. Refusal to excavating was encountered at depths ranging from about 0.2 to 3.3 metres below the existing ground surface.

No practical restrictions apply to the thickness of grade raise fill which may be placed on the site from a foundation design perspective.

As a general guideline regarding the site grading, the preparation for filling of the site should include stripping any topsoil, fill, and organic matter to improve the settlement performance of structures, services, and roadways. Topsoil, fill, and organic matter are not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only. In areas with no proposed structures, services, or roadways, these materials may be left in-place provided some settlement of the ground surface following filling can be tolerated.

Groundwater seepage was generally encountered at depths ranging from about 1.3 to 2.1 metres below the existing ground surface. More significant groundwater flow should be expected for excavations that extend below the groundwater level in these areas. Therefore, in these areas, consideration should be given (but not necessary from a geotechnical perspective) to setting the grading in order to limit the required depths of excavation (particularly for basements) since groundwater management requirements and costs increase with excavation depth below the groundwater level.

It is understood that drainage tiles may exist on the site. All drainage tiles should be removed. Within building footprints and below service trenches, the surrounding bedding/fill for the tile should also be removed (i.e., the existing trench for the drainage tile should be stripped to native soil or bedrock) and the resulting trench should be backfilled with Ontario Provincial Standard Specification (OPSS) Granular B Type II compacted to 95% of its maximum Proctor dry density.

Where the drainage tiles underlie landscaped areas or roadways, any suitable and compactable general fill may be used after removal of the drainage tiles, except within 1.8 metres depth of roadway surfaces where the fill used should match the native soils for frost heave compatibility.

5.3 Foundations

With the exception of the topsoil, the native undisturbed inorganic soils and bedrock on this site are considered suitable for the support of conventional wood frame houses on spread footing foundations. For design purposes, strip footing foundations, up to 1 metre in width, can be designed using a maximum allowable bearing pressure of 100 kilopascals for the overburden soils, consistent with design in accordance with Part 9 of the Ontario Building Code. For footings founded on or within bedrock, an allowable bearing pressure of 250 kilopascals may be used.

The post-construction total and differential settlements of footings supported on soil and sized using the above maximum allowable bearing pressure should be less than 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed before or during construction. Footings on bedrock should experience negligible settlements.

Some of the overburden soils on this site contain cobbles and boulders. Any cobbles or boulders in footing areas which have been loosened by the excavation process should be removed and the cavity filled with lean concrete.

At some locations on the property, and depending on the amount of proposed grade raise (i.e., filling), the inorganic subgrade elevation may be lower than the underside of footing elevation. At these locations, the subgrade may be raised to the footing elevation using suitable engineered fill. The engineered fill should consist of Ontario Provincial Standard Specification (OPSS) Granular B Type II. All fill material should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the material’s standard Proctor maximum dry density (SPMDD) using suitable vibratory compaction equipment. The engineered fill material must be placed within the full zone of influence of the house foundations. The zone of influence is considered to extend out and down from the edge of the perimeter footings at a slope of 1 horizontal to 1 vertical. The same bearing pressures provided above may be used on properly constructed engineered fill pads.

Where the subgrade at footing level changes from bedrock to overburden, differential settlement could result at this transition due to the different settlement properties of these materials. To limit the magnitude of the differential settlement, transition details (such as placing additional reinforcing steel in the foundation walls) may be required. Where sloping bedrock is encountered, stepped footings may also need to be considered. The structural engineering consultant should be contacted for input on these issues.

There may be portions of the site where the shallow sandy deposits will be exposed at footing/subgrade level. Prior to construction of footings or the placement of engineered fill within these areas, the surface of the native sandy material should be proof rolled to provide surficial densification of any loose or disturbed material.

Since these shallow sandy deposits, wherever present, are typically loose, they could be potentially liquefiable in an earthquake (i.e., potentially subject to temporary strength loss and post-earthquake settlements). That potential issue is not however considered relevant to the house design because:

- The potential post-earthquake differential settlements would be relatively small in relation to the expected collapse potential of a house (and the objective of earthquake-resistant design is only to avoid collapse and to provide for safe exit).
- The proof rolling of the sandy subgrade soils, as specified above, would densify any such soils in the immediate area of the footings and therefore the directly supporting soils would be non-liquefiable.

5.4 Seismic Design Considerations

The seismic design provisions of the 2012 Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or bedrock below founding level. Based on the 2012 Ontario Building Code methodology, this site can be assigned a Site Class of D.

Although the seismic Site Class is not directly applicable to structures designed in accordance with Part 9 of the OBC (i.e., conventional housing), this assessment is provided to address City of Ottawa requirements that relate to housing on Site Class E sites.

5.5 Frost Protection

The soils at this site are frost susceptible. For frost protection purposes, all exterior footings or interior footings in unheated areas should be provided with a minimum of 1.5 metres of earth cover. Isolated, exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover. Houses with conventional depth basements would satisfy these requirements.

5.6 Basement Excavations

Excavations for basements will be made through overburden deposits. Bedrock is also expected to be encountered for standard house foundations, but will depend on the proposed grading for the site.

No unusual problems are anticipated with excavating the overburden materials using large hydraulic excavating equipment, recognizing that significant cobble and boulder removal can be expected in areas of the site. Boulders larger than 0.3 metres in diameter should be removed from the excavation side slopes for worker safety.

If required, shallow depths of bedrock removal could be accomplished using mechanical methods (such as hoe ramming in conjunction with line drilling). Deeper excavations into bedrock would likely require blasting. Further details on blasting are provided in Section 5.9.1 of this report.

Above the water table, side slopes should be stable in the short term at 1 horizontal to 1 vertical (Type 3 soil in accordance with the Occupational Health and Safety Act of Ontario (OHSA)). Below the water table, side slopes of 3 horizontal to 1 vertical (Type 4 soil in accordance with the OHSA) will be required to prevent sloughing of the sandier soils.

Near-vertical excavation side slopes in the bedrock should be feasible.

Based on present groundwater levels, excavations deeper than about 1.3 metres will extend below the groundwater level. Groundwater inflow into the excavations should feasibly be handled by pumping from sumps within the excavations. The actual rate of groundwater inflow will depend on many factors including the contractor’s schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where significant volumes of precipitation, surface runoff and/or groundwater collects in an open excavation, and must be pumped out.

Where the subgrade is found to be wet and sensitive to disturbance, consideration should be given to placing a mud slab of lean concrete over the subgrade (following inspection and approval by geotechnical personnel) or a 150 millimetre thick layer of OPSS Granular A underlain by a non-woven geotextile, to protect the subgrade from construction traffic.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. Considering the size of the development and the groundwater information collected during the investigation, it is considered likely that a PTTW would be required for this project. Assistance with carrying out the PTTW application can be provided, if requested.

5.7 Basement and Garage Floor Slabs

In preparation for the construction of the basement floor slabs, all loose, wet, and disturbed material should be removed from beneath the floor slab. Provision should be made for at least 200 millimetres of 19 millimetre crushed clear stone to form the base of the basement floor slabs. The underslab fill should be compacted to at least 95 percent of the materials standard Proctor maximum dry density.

To prevent hydrostatic pressure build up beneath the basement floor slabs, it is suggested that the granular base for the floor slabs be positively drained. This can be achieved by providing a hydraulic link between the underfloor fill and exterior drainage system.

The groundwater level was generally observed to be at about 1.3 to 2.1 metres depth. Although not required from a geotechnical perspective, raising of site grades in areas with a high water table would be beneficial in reducing the water control measures for foundation construction. Similarly, since significant and sustained groundwater inflow into the foundation drainage system would ideally be avoided, the founding depths should be set above the groundwater level.

Where the groundwater level is encountered above subgrade level, a geotextile could be required between the clear stone underslab fill and the sandy subgrade soils, to avoid loss of fine soil particles from the subgrade soil into the voids in the clear stone and ultimately into the drainage system. In the extreme case, loss of fines into the clear stone could cause ground loss beneath the slab and plugging of the drainage system. Where a geotextile is required, it should consist of a Class II non-woven geotextile with a Filtration Opening Size (FOS) not exceeding about 100 microns, in accordance with Ontario Provincial Standard Specification (OPSS) 1860.

The garage backfill should be placed in maximum 300 millimetre thick lifts and be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable compaction equipment.

The granular base for the garage floor slabs should consist of at least 150 millimetres of Granular A compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable compaction equipment.

5.8 Basement Walls and Foundation Wall Backfill

The soils at this site are highly frost susceptible and should not be used as backfill directly against exterior, unheated, or well insulated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively, a bond break such as the Platon system sheeting could be placed against the foundation walls.

Drainage of the basement wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit. Conventional damp proofing of the basement walls is appropriate with the above design approach.

Where design of basement walls in accordance with Part 4 of the 2012 Ontario Building Code is required, walls backfilled with granular material and effectively drained as described above should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress with a base magnitude of $K_o\gamma H$, where:

K_o = The lateral earth pressure coefficient in the 'at rest' state, use 0.5;

γ = The unit weight of the granular backfill, use 21.5 kilonewtons per cubic metre; and,

H = The height of the basement wall in metres.

If Platon System sheeting or similar water barrier product is used against the foundation walls, then hydrostatic groundwater pressures should also be considered in the calculation of the lateral earth pressures.

5.9 Site Servicing

5.9.1 Excavations

Excavations for the installation of site services will be made through sand and silt deposits, glacial till and into the underlying bedrock.

No unusual problems are anticipated with trenching in the overburden using conventional hydraulic excavating equipment, recognizing that cobbles and boulders can be expected in the glacial till. Boulders larger than 0.3 metres in size should be removed from excavation side slopes for worker safety.

The soils above the groundwater table would generally be classified as a Type 3 soil in accordance with the Occupational Health and Safety Act of Ontario. As such, these excavations may be made with side slopes at 1 horizontal to 1 vertical. Where trenches for the installation of services extend below the water table, the excavation side slopes would need to be no steeper than 3H:1V (Type 4 soil). Alternatively, the excavations could be carried out using steeper side slopes with all manual labour carried out within a fully braced, steel trench box for worker safety.

Some groundwater inflow into the trenches should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations, provided suitably sized pumps are used.

The actual rate of groundwater inflow into the trench will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, and the time of year at which the excavation is carried out. There may also be instances where significant volumes of precipitation collect in an open excavation, and must be pumped out.

A PTTW is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the EASR as a prescribed activity. It is anticipated, due to the size of the project, that the contractor may have several trenches open at one time and that a PTTW may need to be obtained for the overall project.

If required, it is expected that the bedrock removal for this project will be carried out using drill and blast techniques. Mechanical methods of rock removal (such as hoe ramming) can likely be carried out for depths of about one metre; however, this work would likely be slow and tedious.

Near vertical trench walls in the bedrock should stand unsupported for the construction period.

If blasting is used, it should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

A pre-blast survey should be carried out of all of the surrounding structures. Selected existing interior and exterior cracks in the structures should be identified during the pre-blast survey and should be monitored for lateral or shear movements by means of pins, glass plate telltales and/or movement telltales.

The contractor should be limited to only small controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested.

Frequency Range (Hertz)	Vibration Limits (millimetres/second)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

These limits should be practical and achievable on this project.

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and within the structures themselves.

If excavations are made through the bedrock, the groundwater inflow from the bedrock could at first be relatively significant. That inflow may potentially diminish with time and continued pumping.

5.9.2 Bedding and Backfill

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials or silty/sandy soils on the trench walls could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the sand, and silty sand as trench backfill, provided that they are not too wet to handle, place, and compact. Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

It should be possible to use the bedrock as trench backfill, provided the bedrock is well broken and broadly graded (maximum size of 300 millimetres). The rock fill, however, should only be placed from at least 300 millimetres above the pipes to minimize damage due to impact or point load. The rock fill should be limited to a maximum of 300 millimetres in size.

5.10 Pavement Design

In preparation for pavement construction, all topsoil, fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the roadway areas.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. Perforated pipe sub-drains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres longitudinally, parallel to the curb in two directions.

The pavement structure for the interior 'local' roadways which will not experience bus or truck traffic should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	375

The pavement structure for the interior roadway(s) with bus and truck traffic should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

For arterial roadways, the subbase thickness should be increased to 600 millimetres.

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

The composition of the asphaltic concrete pavement should be as follows:

- Superpave 12.5 mm Surface Course – 40 millimetres
- Superpave 19 mm Base Course – 50 millimetres

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B for local roadways and Category D for collector roadways.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.11 Stormwater Management Pond

5.11.1 Temporary Pond Excavation

It is understood that the bottom of the excavation for the pond will generally extend about 3 to 6 metres below the existing ground surface, to about elevation 103.2 metres, which is about 2 to 4 metres below the groundwater level measured at the site. The proposed floor of the pond will generally be at about elevation 104.25 metres. At the inlet and outlet structures, additional subexcavation may be required locally to reach the founding elevations (which are not known at this time).

The general excavation for the pond will be through the topsoil, silt and sand deposits, and into the underlying glacial till. Based on the available information, the bedrock surface is below the proposed pond floor. However, bedrock may be encountered towards the north end of the pond.

No unusual problems are anticipated with excavating the overburden material using conventional hydraulic excavating equipment, although cobbles and boulders will be encountered within the glacial till. Temporary side slopes in the overburden should be stable in the short term at 1 horizontal to 1 vertical above the groundwater level and at 3 horizontal to 1 vertical below the groundwater level (i.e., Type 3 and Type 4 soils per the Occupational Health and Safety Act, respectfully) to the planned depths of excavation. Boulders larger than 300 millimetres in diameter must be removed from the excavation side slopes for worker safety.

Near-vertical excavation side slopes in the bedrock should be feasible.

The stand-up time for exposed side slopes in the overburden will be extremely short and the subgrade will be disturbed if left exposed. The rate of groundwater inflow from the overburden could be significant. Based on past experience at adjacent sites and particularly where excavations are deeper, some pre-draining of the overburden may be required. For example, several sumps could be constructed and pre-pumping of the overburden carried out. Pre-pumping for a couple of weeks prior to construction will likely be reasonable.

It is suggested that a test excavation be carried out at the location of the pond to help the contractor evaluate the groundwater management requirements.

Based on the available information, the floor of the pond is expected to consist of deposits of silts and sands and possibly glacial till. Bedrock may be encountered at the north end of the pond. The soils and bedrock at this site are expected to have high hydraulic conductivities. Groundwater inflow from the overburden will at first be relatively significant but should diminish with time and continued pumping.

Basal heaving of the excavation floor could result while excavating portions of the pond where there is limited overburden cover over the bedrock surface. Basal heaving occurs where the piezometric pressure in the bedrock exceeds the weight of the overburden cover between the excavation floor and the bedrock surface. Basal heaving would disturb the pond subgrade during construction, likely making it un-trafficable for equipment and difficult to shape. The rate of groundwater inflow would also increase significantly.

The following options could be considered to address the issue of basal heave and the resulting potential for subgrade disturbance during construction:

- The pond could be excavated ‘in the wet’, such that water would be maintained in the pond during excavation and the weight of that water would help resist the uplift pressures. However, the difficulties that would be experienced with shaping the pond floor and placing the clay liner if excavating ‘in the wet’ should be considered.
- Additional subexcavation of the overburden could be carried out to the bedrock surface. This would eliminate the potential for subgrade disturbance, however would deepen the pond depth, which may not be feasible for design.
- Pre-drainage of the overburden to at least 1 metre below the pond floor, by constructing several sumps and pre-pumping of the overburden and bedrock. This requirement would necessitate lowering the groundwater level in the underlying bedrock to about elevation 102.5 metres. The specific method of reducing the piezometric pressures in the overburden and/or bedrock should be selected and designed by the contractor. However, it is expected that this lowering could conceivably be achieved by installing and pumping from multiple wells around the perimeter of the pond, and possibly within the excavation itself. The groundwater level in the bedrock would need to be monitored, particularly in the area between the wells, and excavation for the pond construction should not proceed until the piezometric pressure has been sufficiently lowered.
- Soils on this site are sensitive to disturbance by construction traffic and ponded water. It would be impractical for equipment to travel within the excavation without first constructing haul roads to prevent softening and disturbance of the exposed subgrade. Therefore, excavation of the pond in one bench, with the equipment working from existing ground surface and not travelling within the excavation, may be necessary.

5.11.2 Groundwater Management

The excavation for the pond is expected to extend below the groundwater level. While the groundwater inflow during construction are expected to be high, the groundwater inflow into the excavation should feasibly be handled by pumping from several sumps within the excavation.

The actual rate of groundwater inflow to the excavation will depend on many factors including the contractor’s schedule and rate of excavation, the size of the excavation and the time of year at which the excavation is made. Also, there may be instances where significant volumes of precipitation, surface runoff and/or groundwater collects in the open excavation, and must be pumped out.

For construction dewatering that exceeds a rate of 50 m^3/day (50,000 L/day), but less than 400 m^3/day (400,000 L/day), a Ministry of Environment and Climate Change (MOECC) Environmental Activity and Sector Registration (EASR) is required, and must be supported by a water taking plan. For pumping that exceeds 400 m^3/day (400,000 L/day), a MOECC Permit To Take Water (PTTW) would be required.

Given the overall size of the project, the depth of excavation, the groundwater levels, and the potentially high hydraulic conductivity of the sand and silt deposits, it is likely that construction dewatering for the pond would exceed 400 m^3/day (400,000 L/day). It is therefore recommended that a PTTW be obtained from the MOECC for this project.

5.11.3 Permanent Pond Side Slopes

The permanent side slopes for the proposed storm water management pond will consist of deposits of sands and silts and possibly into the glacial till.

It is understood that the permanent pool water level in the pond will be at about elevation 105.75 metres, and up to about elevation 107.34 metres during a 100-year storm event. It is also understood that the permanent pond sides will be sloped at about 3 horizontal to 1 vertical to 5 horizontal to 1 vertical.

The stability of the proposed pond side slopes was evaluated for drained (long-term, static) and undrained (seismic) conditions, in which effective stress soil parameters were used. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modeling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is generally used to define a stable slope for static loading conditions. A factor of safety of 1.1 is used to define a stable slope for seismic loading conditions.

The results of limit equilibrium stability analyses indicate that the pond side slopes have a factor of safety of at least 1.5 against long term instability if inclined at 3 horizontal to 1 vertical (3H:1V), or flatter, which is acceptable. For seismic loading conditions, the analyses indicate that the slope should have a factor of safety of at least 1.1, which is also acceptable. During construction, some sloughing of the pond slopes should be expected when initially excavated due to the groundwater inflow. The rate of groundwater inflow will reduce with time and continued pumping. The side slopes should therefore first be cut within the pond footprint to a somewhat steeper inclination, such as 2 horizontal to 1 vertical, and only flattened to the final crest of slope once the rate of groundwater inflow has reduced and sloughing will no longer occur. This process will minimize the need to rebuild the slopes. Due to the mainly non-cohesive nature of the native soils at the site, some minor sloughing of the side slopes may still occur after construction but prior to the vegetation being established. Although the sloughing would not impair the short term use of the pond, it may require some additional maintenance/repair following construction.

To limit the potential for erosion (and/or sloughing) of the pond side slopes, the slopes should be covered with topsoil and vegetated as quickly as possible. Consideration could also be given to providing temporary erosion protection by the use of a biodegradable erosion control blanket.

5.11.4 Inlet and Outlet Structures

The proposed structures for the pond include concrete bypass structures, two inlet structures, and one concrete outlet structure. The founding elevations for the proposed structures are not known at this time. The design guidelines in this section of the report should be confirmed as the design progresses.

The excavation guidelines provided in Section 5.11.1 of this report are also generally applicable for the inlet and outlet structures.

The subsurface conditions in the area of the inlet and outlet structures consist of up to about 3.3 to greater than 4.2 metres of sandy silt, sand and silt, and silt and possibly glacial till overlying bedrock.

It is considered that the proposed inlet structures could be supported on spread footing placed within the sand and silt deposits, glacial till or bedrock.

The SLS bearing resistance for spread footings founded within overburden may be taken as 75 kilopascals. The ULS factored bearing resistance may be taken as 150 kilopascals. The ULS factored bearing resistance may be taken as 250 kilopascals for structures founded on the bedrock.

The post-construction total and differential settlements of footings sized using the above SLS net bearing resistance should be less than about 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction.

Structures should be provided with a minimum of 1.5 metres of earth cover. Insulation of the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection, if required.

The seismic design provisions of the 2012 Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or bedrock below founding level. Based on the 2012 Ontario Building Code methodology, this site can be assigned a Site Class of D for the structures.

The soils at this site are highly frost susceptible and, as such, should not be used as backfill against the structures. To avoid frost adhesion of the backfill to the structure and heaving of the foundations, the structure should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for Ontario Provincial Standard Specification OPSS Granular B Type I. However, that backfill material would be more permeable than the native soils and therefore could form a preferential conduit for seepage losses around the structure, which could then erode that material and form a direct conduit for the flow of retained water. It is therefore proposed that the surface of the backfill be capped with low permeability soil (no more than about 0.5 metres thick) wherever it could be in contact with the retained water.

The structure walls or head walls should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress, which may be determined as follows:

$$\sigma_h(z) = K\,(\gamma\, z + q)$$

Where:

- $\sigma_h(z)$ = The lateral earth pressure at depth ‘z’ (kPa);
- z = The depth below ground surface (m);
- K = The at-rest pressure coefficient, K_0, for non-yielding walls, use 0.5 or the active pressure coefficient, K_a, for yielding walls, use 0.33;
- γ = The unit weight of the backfill soil (kN/m^3), use 21.5 kilonewtons per cubic metre; and,
- q = The surcharge due to live loads on the ground surface above the structures (kPa).

If the walls allow lateral yielding, active earth pressures should be used in the design of the structure. If the walls do not allow for lateral yielding, at-rest earth pressures should be assumed for the design. The value of the surcharge due to live loading (q) should consider the potential construction loads from equipment or materials. A value of no less than 15 kilopascals could be reasonable.

Hydrostatic water pressures should also be considered for the portion of the foundation walls below the measured groundwater level.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The combined pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K\,\gamma\,z + (K_{AE} - K)\,\gamma\,(H-z)$$

Where: K_{AE} = The seismic earth pressure coefficient, use 0.7, and,

H = The total depth to the bottom of the foundation wall (m).

The above seismic earth pressure coefficient assumes that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

The K_{AE} value for yielding walls is applicable provided the wall can move up to 75 millimetres at this site.

In addition, the potential hydrodynamic pressures from the groundwater should be considered under seismic loading conditions for that portion of the foundation wall below the groundwater level. The additional hydrodynamic pressure may be calculated using the following expression:

$$p(z) = 1.5\,k_h\,\gamma_w\,(h^*z)^{1/2}$$

Where: $p(z)$ =The additional hydrodynamic water pressure, at depth z below the water table;

k_h =The design horizontal ground acceleration, 0.30,

γ_w =The unit weight of water, use 9.81 kN/m^3; and,

h =The total depth of water.

All of the above lateral earth pressure equations and parameters are given in an unfactored format.

5.11.5 Inlet and Outlet Pipes

As discussed previously in Sections 5.11.1 and 5.11.2 of this report, the stand-up time of the soils below the groundwater table will be extremely short and pre-drainage of the overburden may be required. Groundwater inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavation.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

It should generally be possible to re-use the layered sand and silt above the groundwater level as trench backfill. The sand and silt deposits below the water table may be too wet to handle and will likely need to be wasted or used as general landscape fill. Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the materials standard Proctor maximum dry density using suitable compaction equipment.

5.11.6 Re-use of Excavated Materials

Excavated materials from the pond may be re-used as landscape fill on other portions of the site. These soils may initially be too wet to easily handle and shape and compact into place. It may be necessary to allow them to drain/dry out somewhat prior to their re-use.

5.11.7 Access Road for Maintenance

A roadway will be provided along the north, east, and south sides of the pond to provide access to the inlet and outlet control structures for inspection and maintenance purposes.

In preparation for pavement construction, all topsoil and deleterious material (i.e., material containing organic material) should be removed from all pavement areas.

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material meeting the requirements of OPSS.MUNI 212 and 1010, respectively. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The surface of the subgrade should be crowned to promote drainage of the pavement granular structure.

Provided that the access road will be used fairly infrequently by light equipment, the pavement structure should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	50
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The composition of the asphaltic concrete pavement should be as follows:

- Superpave 12.5 Surface Course – 50 millimetres, PG 58-34

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where grade raise fill has been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.11.8 Pond Maintenance and Access

If the pond floor needs to be trafficable, the bottom of the pond should be lined with a material such as rip-rap, a synthetic geocell erosion layer, or interlocking concrete blocks to minimize disturbance to the subgrade and allow for maintenance vehicles to travel in the pond. A geotextile may also be required in addition to the materials mentioned above.

5.12 Corrosion & Cement Type

One soil sample from test pit 17-08 was submitted to Eurofins Environmental Testing Canada for chemical analysis related to potential corrosion of buried steel elements and sulphate attack on buried concrete elements. The results of this testing are provided in Appendix A.

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate a moderate potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures.

5.13 Pools, Decks and Additions

5.13.1 Above Ground and In Ground Pools

No special geotechnical considerations are necessary for the installation of above-ground or in-ground pools.

5.13.2 Decks

There are no special geotechnical considerations for decks on this site.

5.13.3 Additions

Any proposed addition to a house (regardless of size) will require a geotechnical assessment. Written approval from a geotechnical engineer should be required by the City prior to the building permit being issued.

6.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance from ponded water, construction traffic, and frost.

The test pits excavated and filled on site constitute zones of disturbance to the surficial soils. These could affect the performance of surface structures/foundations. The test pit locations were selected to be outside of proposed building areas (i.e., within the roadways); however, the site plan has changed since the investigation and there may be instances where a test pit underlies a proposed structure. In that case, the backfill soil in the test pit will need to be removed and replaced with engineered fill.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

At the time of the writing of this report, only preliminary details for the proposed development were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

7.0 CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report or if we can be of further service to you on this project, please call us.

Yours truly,

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
W. CAVERS
100077934
Sept 20/18
PROVINCE OF ONTARIO

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https://golderassociates.sharepoint.com/sites/16191g/deliverables/geotech report/1784392 rpt-001-rev3-geotech report 2018-09-20 final.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
17-01 (111.09 metres)	0.00 – 0.25	TOPSOIL – (ML) Sandy SILT; non-cohesive, dark brown, moist
	0.25 – 0.92	(ML) Sandy SILT, some gravel; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	0.92	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.30 – 0.75	W_n = 22%

17-02 (109.47 metres)	0.00 – 0.30	TOPSOIL – (SM) SILTY SAND; non-cohesive, dark brown, moist
	0.30 – 1.05	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	1.05	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.04 – 1.00	

17-03 (111.80 metres)	0.00 – 0.30	TOPSOIL – (ML) Sandy SILT; brown; non-cohesive, moist
	0.30 – 1.30	(SP) SAND, some non-plastic fines and gravel; grey, non-cohesive, moist
	1.30	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.10 – 0.25	
2	0.50 – 1.25	

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
17-04 (112.01 metres)	0.00 – 0.25	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.25 – 1.30	(SM) SILTY SAND, trace gravel; brown; non-cohesive, moist
	1.30	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Water seepage at 1.3 metres depth

Sample	Depth (m)	Lab Testing
1	0.30 – 0.50	

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
17-05 (113.55 metres)	0.00 – 0.50	FILL – (SP) SAND; brown, contains brush (WINDROW); non-cohesive, moist
	0.50 – 0.70	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive moist
	0.70 – 1.60	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL), non-cohesive, moist
	1.60	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	1.00 – 1.50	W_n = 9% MH see Fig. 3

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
17-06 (111.29 metres)	0.00 – 0.20	FILL/TOPSOIL – (ML) Sandy SILT; brown; non-cohesive, moist
	0.20 – 1.20	FILL – (SP/GP) SAND and GRAVEL; brown, contains cobbles (BLAST ROCK); non-cohesive, moist
	1.20 – 1.50	(ML) Sandy SILT, some gravel; grey (GLACIAL TILL); non-cohesive, moist
	1.50	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	1.30 – 1.50	

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-07 (108.94 metres)	0.00 – 0.25	TOPSOIL – (SM) SILTY SAND; dark brown; non-cohesive, moist
	0.25 – 1.10	(SM) SILTY SAND; brown; non-cohesive, moist
	1.10 – 1.80	(SM) SILTY SAND; brown; contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	1.80	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.50 – 1.00	
2	1.25 – 1.75	

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-08 (109.50 metres)	0.00 – 0.10	TOPSOIL – (SM) SILTY SAND; dark brown; non-cohesive, moist
	0.10 – 0.40	(SM) SILTY SAND; brown; non-cohesive, moist
	0.40	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.10 – 0.40	

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-09 (111.62 metres)	0.00 – 0.15	(SP) SAND, some non-plastic fines, trace gravel; brown; non-cohesive, moist
	0.15	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.00 – 0.15	

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-10 (110.63 metres)	0.00 – 0.15	TOPSOIL (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.15 – 1.85	(SM) gravelly SILTY SAND; grey; non-cohesive, moist
	1.85	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.30 – 0.65	
2	1.50 – 1.75	W_n = 10% MH see Fig. 2

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-11 (109.72 metres)	0.00 – 0.40	TOPSOIL (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.40 – 1.10	(SM) SILTY SAND; brown; non-cohesive, moist
	1.10 – 2.10	(SM) SILTY SAND; grey; contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	2.10	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry upon completion

Sample	Depth (m)	Lab Testing
1	0.50 – 1.00	
2	1.25 – 1.75	W_n = 9%

Test Pit Number Elevation (Metres)	Depth (metres)	Description
17-12 (107.17 metres)	0.00 – 0.35	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.35 – 1.20	(ML) Sandy SILT; brown; non-cohesive, moist
	1.20 – 4.20	(SP-ML) SAND and SILT, trace gravel; grey-brown, non-cohesive, wet
	4.20	END OF TEST PIT
		NOTE: Water seepage causing side walls to collapse

Sample	Depth (m)	Lab Testing
1	0.50 – 1.00	
2	1.50 – 2.50	W_n = 11% MH see Fig. 2

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
TP 17-13 (105.91 metres)	0.00 – 0.30	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.30 – 0.85	(SP/ML) SAND and SILT; brown; non-cohesive, moist
	0.85 – 2.95	(ML) Sandy SILT; grey-brown, non-cohesive, moist
	2.95 – 3.30	(ML) SILT, trace sand; grey; non-cohesive, moist
	3.30	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Water seepage at 3.0 metres depth

Sample	Depth (m)	Lab Testing
1	0.40 – 0.60	
2	0.95 – 1.30	
3	3.00 – 3.30	

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
TP 17-14 (106.80 metres)	0.00 – 0.20	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.20 – 1.50	(SM) SILTY SAND; brown; non-cohesive, moist
	1.50 – 4.50	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet
	4.50	END OF TEST PIT
		NOTE: Water seepage at 3.4 metres depth Could not excavate deeper than 4.5 metres Water seepage causing side walls to collapse

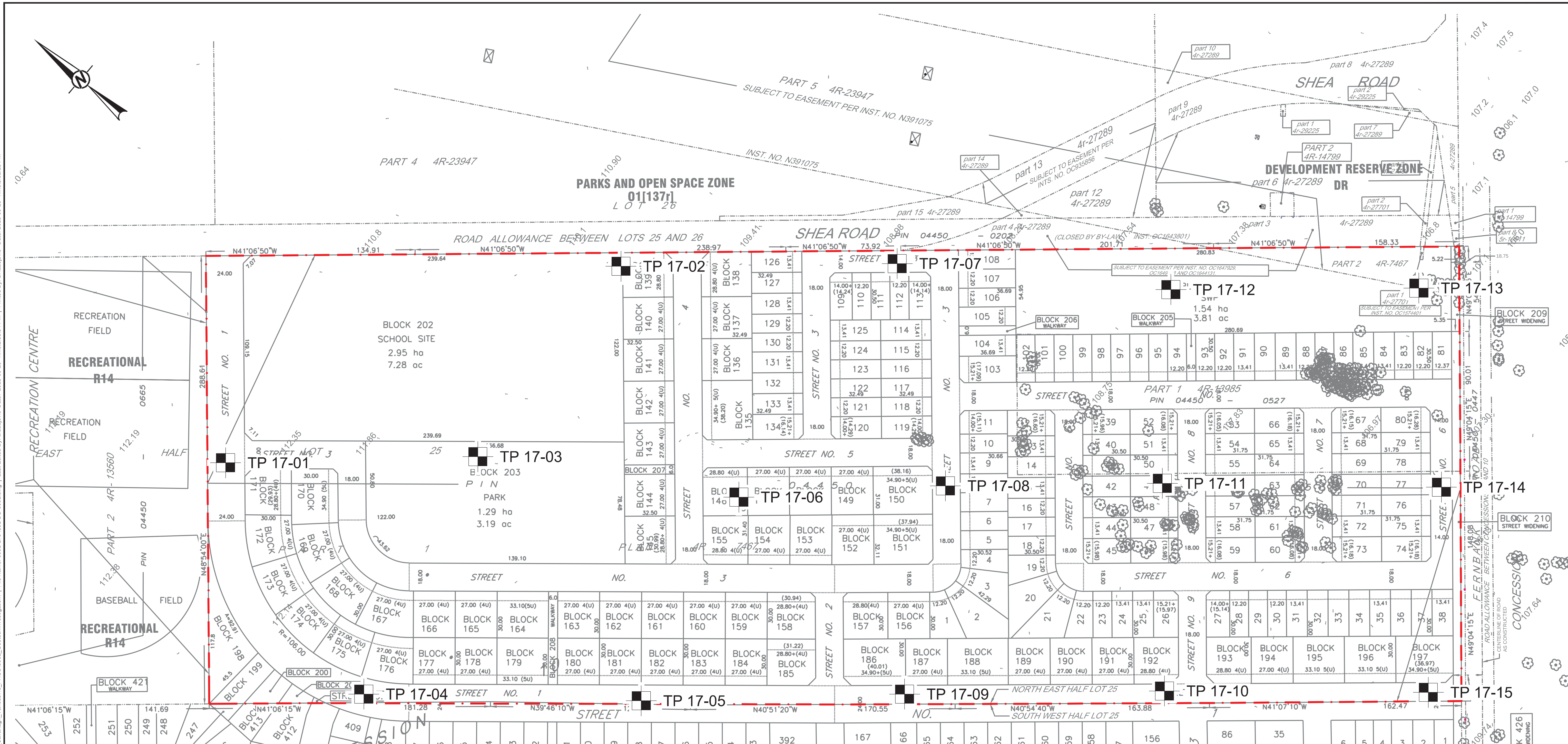
Sample	Depth (m)	Lab Testing
1	0.30 – 1.25	
2	2.00 – 2.50	

TABLE 1

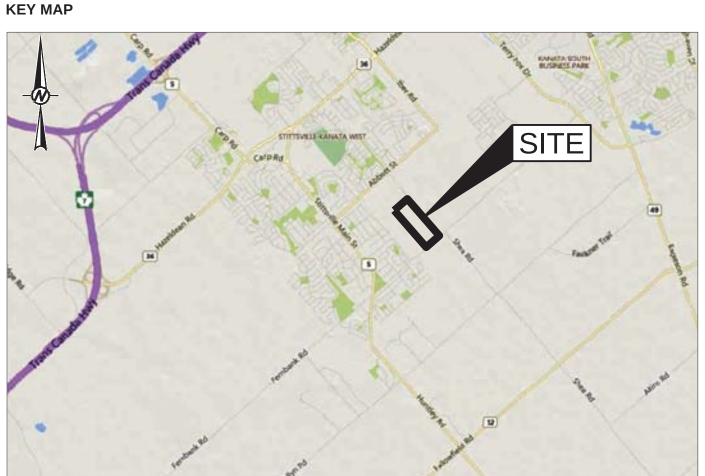
RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	Depth (metres)	Description
TP 17-15 (108.58 metres)	0.00 – 0.75	TOPSOIL – (ML) Sandy SILT; dark brown; non-cohesive, moist
	0.75 – 1.25	(SM) SILTY SAND, brown, non-cohesive, moist
	1.25 – 1.60	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	1.60	END OF TEST PIT – Refusal to excavation on BEDROCK
		NOTE: Test pit dry

Sample	Depth (m)	Lab Testing
1	0.50 – 1.00	
2	1.30 – 1.55	



KEY MAP



SCALE 1:125,000

LEGEND

APPROXIMATE TEST PIT LOCATION

SITE BOUNDARY

REFERENCE(S)

1. BASE SURVEY SUPPLIED BY THOMAS CAVANAGH CONSTRUCTION LIMITED ON SEPTEMBER 13, 2018, FILE NO. 161613153-131_Shea-Cavanagh_d12_c3d-Sept 6 18.dwg
2. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 50 100

1:2,500 METRES

CLIENT
1384341 ONTARIO LTD.

PROJECT
GEOTECHNICAL INVESTIGATION
5969 FERNBANK ROAD
OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

GOLDER

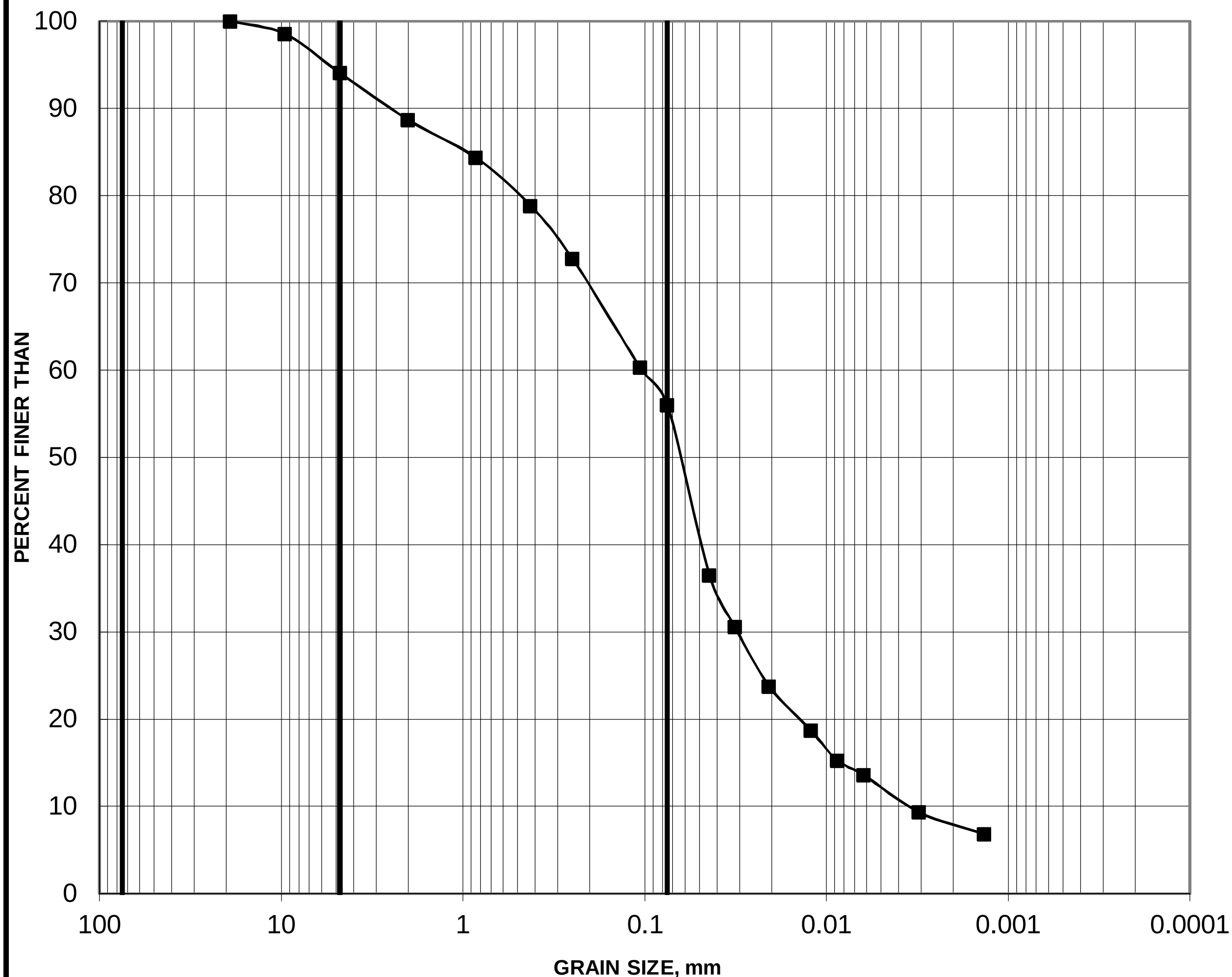
YYYY-MM-DD	2018-09-20
DESIGNED	----
PREPARED	JM/ABD
REVIEWED	WAM
APPROVED	TMS

PROJECT NO.	CONTROL	REV.	FIGURE
1784392	0001	A	1

GRAIN SIZE DISTRIBUTION

FIGURE 2

(ML) Sandy SILT



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

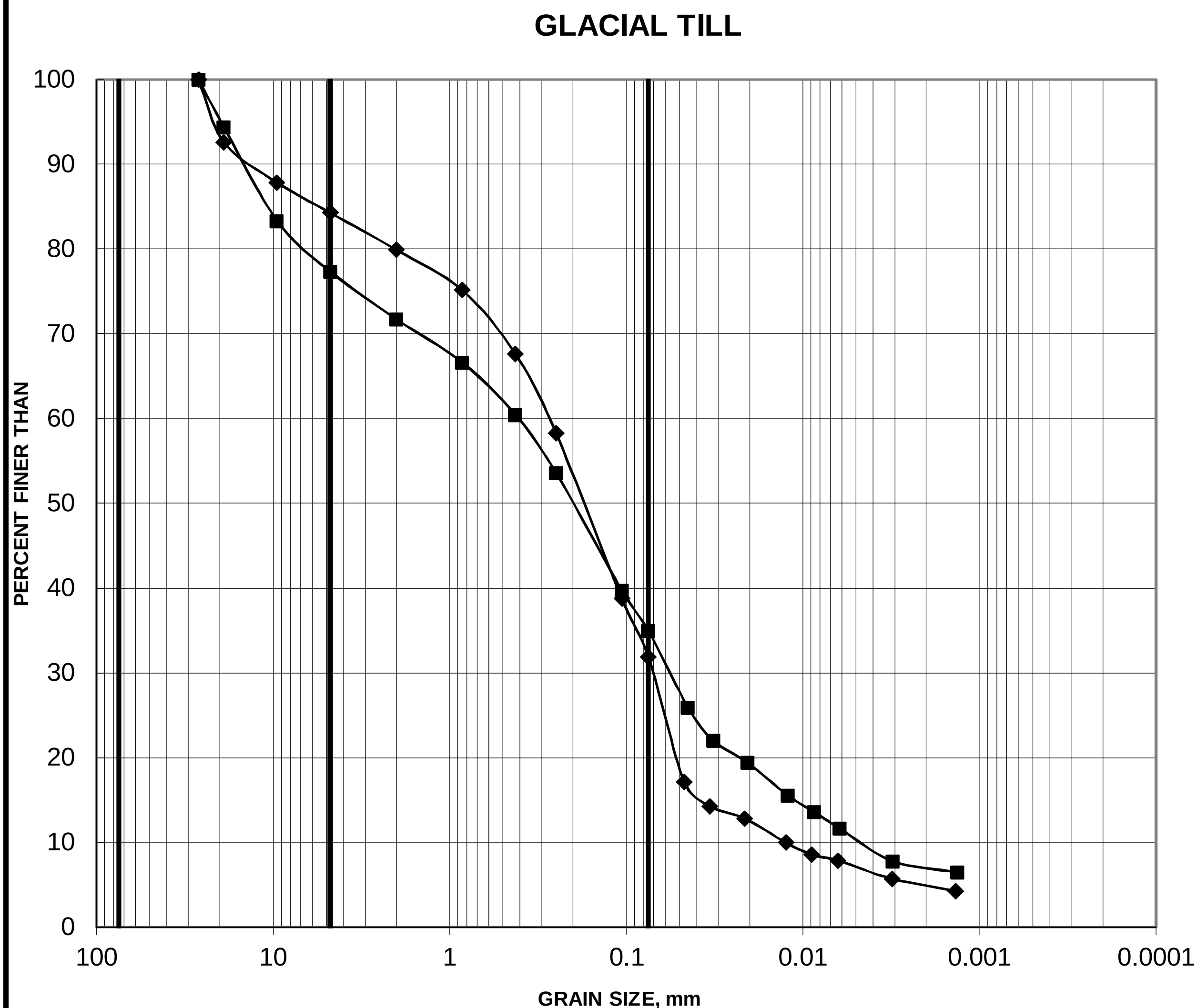
Borehole	Sample	Depth (m)
17-12	2	1.50-2.50

Project: 1784392 /1000

Golder Associates

Created by: CNM
Checked by: AC

GRAIN SIZE DISTRIBUTION	FIGURE 3



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
17-05	1	1.00-1.50
17-10	2	1.50-1.75

Project: 1784392 /1000

Golder Associates

Created by: CNM
Checked by: AC

APPENDIX A

Results of Chemical Analysis – Eurofins Environmental Testing, Report No. 1800603

Certificate of Analysis

Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1800603
Attention:	Mr. Alex Meacoe	Date Submitted:	2018-01-12
PO#:		Date Reported:	2018-01-19
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	1784392
		COC #:	827422

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1340841 Sample Matrix Soil Sample Type Sampling Date 2017-12-14 Sample I.D. 17-08 sa1 0.1-0.4m
Agri. - Soil	pH	2.00			7.28
	SO4	0.01	%		0.02
General Chemistry	Cl	0.002	%		<0.002
	Electrical Conductivity	0.05	mS/cm		0.11
	Resistivity	1	ohm-cm		9090

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range golder.com