Geotechnical Investigation

1098 Ogilvie Road
Ottawa Ontario

6770967 Canada Inc.

Table of Contents

Figure Index

Table Index

Appendix Index

1. Introduction

GHD was retained by 6770967 Canada Inc. (Client), represented by Mr. Francois Moffet, to undertake a Geotechnical Investigation for a proposed new residential development. The development is located at 1098 Ogilvie Road in Ottawa, Ontario, hereafter referred to as the Site.

The purpose of the investigation was to complete a preliminary assessment of the soil and groundwater conditions to allow the Client and his design consultants to better understand the below grade soil and conditions. It was proposed and agreed by the client that this preliminary assessment will be based upon drilling six boreholes to describe the subsurface stratigraphy at the borehole locations and based upon the data, provide preliminary recommendations concerning foundation type and associated design bearing pressures, groundwater conditions as well as provide comments on excavation, backfill, pavement design and construction field review.

A previous geotechnical investigation was conducted at the site by Paterson Group (Report Number PG2463-1, dated December 12, 2011). The borehole logs from this report were used as reference for this report and are provided in Appendix E along with the associated borehole location plan.

Based upon the large size of this site additional investigations are recommended to provide more extensive coverage of the development site.

This report has been prepared with the understanding that the design will be as described in Section 2 and will be carried out in accordance with all applicable codes and standards. Any changes to the project described herein will require that GHD be retained to assess the impact of the changes on the report recommendations provided herein.

The scope of work for this preliminary geotechnical assessment consisted of the following activities:

- Underground Service Clearances
- Fieldwork | The proposed scope included the advancement of boreholes out-fitted with monitoring wells at six locations spread across the Site.
- Lab Testing | Two grain size analysis (BH2/SS1, BH6/SS2) and moisture contents on selected soil samples. Chemical testing was completed two collected groundwater samples to allow comments on potential corrosive conditions from sulphates to buried concrete and to ductile iron.
- Reporting | Preparation of this Geotechnical Report which summarizes the findings of the fieldwork programs and presents preliminary recommendations for the design and comments for construction of the development.

2. Site and Project Description

At the time of the investigation, the site was a vacant lot with trees and overgrown vegetation. The Site is bounded by Ogilvie Road to the North, Cummings Avenue to the East, a parking lot to the South, and commercial buildings to the West. The site topography slopes down from North to South approximately 3 meters (m).

GHD understands that the proposed development will consist of the construction of the following structures:

- Tower 1 – 22-storey residential building
- Tower 2 – 24-storey residential building
- Link Building - A 6-storey connection between Tower 1 and Tower 2
- Tower 3 – 36-storey residential building with a partial 6-storey section
- 8-storey Hotel
- A 1-storey connection between the Hotel and Tower 3
- 4-levels of underground parking that will extend close to property limits
- Aboveground parking, access road, and landscaped area

GHD has not been informed of any special slab on grade floor loading requirements for this residential development. We understand that the lowest slab will be a slab on grade for the underground parking and local storage or mechanical/electrical equipment rooms. Therefore we are assuming a light floor loading (assumed to be less than 24 kilopascal (kPa) floor loading for slab on grade.

The location of the Site is shown on the Site Location Plan attached as Figure 1.

3. Field Investigation

The fieldwork program consisted of the advancement of six boreholes labelled as BH1 to BH6. Borehole location BH2 had an additional shallow probe drilled (labelled as BH2A) to allow installation of a shallow monitoring well. Boreholes were advanced to depths varying between 3.0 m to 15.8 m below the existing surface grade. Monitoring wells were installed in all boreholes. The monitoring well installed at the BH2A location was sealed within the upper 1.2 m of the overburden. All other wells had bentonite seals placed into the bedrock. The location of the boreholes are shown in the Borehole Location Plan attached as Figure 2 at the end of this report.

The borehole drilling fieldwork program was undertaken on September 23 to 25, 2019 with a truck mounted drill rig, under the supervision of GHD field staff.

Boreholes were advanced into the overburden using Standard Penetration Tests (SPTs) at regular intervals using a 50 mm diameter split-spoon sampler and a 63.5 kg hammer, free falling from a distance of 760 mm, to collect soil samples. The number of drops required to drive the sampler 0.3 m recorded on the borehole logs as "N" value. All boreholes except BH2A were advanced into bedrock using NQ diamond coring equipment, in order to confirm the existence of bedrock and comment on rock quality (ASTM D2113).

All boreholes were backfilled with bentonite hole plug and silica sand upon drilling completion. Auger cuttings were placed in drums and left on site for testing and future disposal.

The elevations of the boreholes were determined by GHD field staff using a laser level and related to an assigned benchmark on Site which was the top of spindle of a fire hydrant on Cummings Avenue to the east of the site. This benchmark was assumed to have an arbitrary elevation of 100 m. The elevations of the boreholes are not geodetic and are for use within the context of this report only.

3.1 Laboratory testing

Laboratory testing on recovered soil samples included two Grain Size Analysis and moisture contents on select samples. The results from the testing assisted in the subsoil descriptions provided below in Section 4 and on the borehole logs. The laboratory test results are also provided in Appendix B, at the end of this report.

Analytical testing was carried out on two groundwater samples to allow preliminary comments on corrosion potential within the subsurface to ductile iron and buried concrete. The results of the chemical analyses are discussed in Section 6.9.

4. Subsurface Conditions

In general, soils encountered at the borehole locations consisted of a layer of a silty sand fill material overlying bedrock. A thin native deposit of sandy to silty clay was encountered beneath the fill layer at the BH1 and BH6 locations.

General descriptions of the subsurface conditions are summarized in the following sections, with a graphical representation of each borehole on the Borehole Logs. Notes on Boreholes are provided in Appendix A, at the end of this report.

4.1 Surface Covers

The site has various shrub and tree cover with local areas of pavement and brush/tall grasses. The boreholes identified a thin topsoil with an approximate thickness of 50 to 75 millimeter (mm) covering Fill soils. At the BH1 location, there was an asphalt surface as part of an abandoned driveway from previous developments on the site.

The topsoil descriptions, and thicknesses within this report are for planning purposes only and should not be used for quality assessments or quantity take-offs.

4.2 Fill

A layer of fill was encountered at all borehole locations. The fill material consisted of a silty sand with trace to some gravel. The fill material was found to be compact, and in a damp to moist condition. The thickness of the fill layer varied from approximately 1.0 m to 2.6 m.

4.3 Sandy Clay

Underlying the fill layer at the borehole BH1 and BH2 locations, a native sandy clay deposit was encountered. In general this deposit was found to be very stiff and was recovered in a damp to moist condition.

4.4 Silty Sand

Underlying the fill layer at the borehole BH4 and BH6 locations, a native silty sand deposit was encountered. The deposit had varying amounts of clay and gravel. In general this deposit was found to be compact to very dense and was recovered in a damp to moist condition.

4.5 Bedrock

Practical refusal to auger advancement was encountered in all boreholes at shallow depths. Bedrock was confirmed by diamond coring methods in all boreholes except BH2A. The depth of bedrock ranged between 1.0 m to 2.7 m. The bedrock was found to be a black and grey sedimentary rock consisting shale of Billings formation with limestone interbeds at the borehole locations. The limestone interbeds increased in frequency with depth. The quality of this rock was generally highly weathered and fractured, very poor within the upper approximately 0.2 to 1.5 m of the bedrock. The quality improves becoming what is considered as fair to excellent rock based upon Rock Quality Designation (RQD) values of 52 to 100.

Strength testing of the bedrock was not completed as part of this preliminary investigation but typically Billings deposit is a weak bedrock with medium strong to strong limestone interbeds.

A photographic log of all the collected rock cores is provided in Appendix C.

5. Groundwater

Monitoring wells were installed in all boreholes as part of the scope of work. Groundwater levels were measured on October 16, 2019, at the monitoring wells. The following Table 5.1 shows the measured water levels.

Table 5.1 Groundwater Observations

Borehole No. (BH)	Depth of Water Below Existing Grade (m)	Elevation (m)*
BH1	2.42	97.95
BH2	2.44	98.37
BH2A	2.44	98.49
BH3	1.72	98.46
BH4	2.59	98.17
BH5	2.26	97.21
BH6	1.15	98.80
Notes: * Elevations are not geodetic		

The recorded groundwater levels show hydrostatic pressure in the bedrock at the Site. GHD recommends a hydrogeological investigation be performed at the site to investigate the hydrostatic pressures and expected groundwater infiltration into the excavation.

It should be noted that the groundwater table is subject to seasonal fluctuations and in response to precipitation and snowmelt events. Also, it would be expected that water may be perched within the fill materials or the very poor bedrock, especially during and following periods of precipitation and in the spring and fall or other wet seasonal periods.

6. Discussion and Recommendations

The recommendations in this report are based on GHD's understanding of the proposed development, which is outlined as follows:

- The proposed structures include low and high-rise buildings with four level underground parking garage structure (see Section 2 of this report for more details)
- A founding depth for the foundations of about 15 m below current ground surface is anticipated and the foundations will be conventional pad and strip type founded within the bedrock
- The slab-on-grade is assumed to be lightly loaded (less than 24 kPa).

Based on the subsurface conditions encountered in the boreholes, and assuming them to be representative of the subsurface conditions across the Site, the following recommendations are provided. Significant geotechnical considerations for design and construction of the proposed structure are:

- Bedrock Excavation **|** Based on the proposed founding depth of foundations for the structures, bedrock excavation will be required. The upper 0.2 to 1.5 m of bedrock was found to be weathered and fractured. The bedrock becomes fair to excellent quality with depth.
- Swelling of the Bedrock **|** Bedrock consists of shale of Billing Formation; this rock is subject to expansion if exposed to air. GHD recommends immediately covering any exposed bedrock surface (horizontal or vertical) with a concrete mud slab to minimize the risk of swelling of the bedrock.
- Adjacent Structures - Construction Activity Induced Vibrations **|** The excavation operations of bedrock will impart vibrations affecting the nearby below grade and above grade structures including the adjacent roadways. The client, designers and contractors should implement measures to reduce risk and severity of vibrations and damage to adjacent structures. The excavation methods must follow the City of Ottawa guidelines.
- Excavation **|** The excavation faces through the overburden depth will need to be adequately shored or sloped. Upper levels of weathered bedrock should be planned to be back sloped at 1:1. The underlying more sound bedrock should be able to be cut at near vertical.
- Dewatering **|** The recorded groundwater levels at the borehole locations show hydrostatic pressure in the bedrock at the Site. GHD did not complete a hydrogeological assessment of this site as part of the scope of work. A hydrogeological assessment is recommended to estimate the extent of dewatering activities and whether a Permit to take water (PTTW) or submission on the Ontario Environmental Activity and Site Registry (EASR) will be required.
- Pre-Construction surveys should be carried out and contractors should incorporate excavation methods to minimize damage due to vibration to the adjacent structures.

6.1 Site Preparation

Site preparation within the new building footprint will involve the removal of existing vegetation, topsoil, asphalt and any existing fill materials to expose the bedrock.

In the proposed pavement areas the site preparation will involve removal of existing vegetation, topsoil, and asphalt. Boreholes show variance in the fill and some existing fills may remain in place under the proposed pavement areas as long as they are verified as competent and meet environmental quality requirements for the site use. Further investigations are recommended to better assess and recommend management plans for the re-use or off-site disposal of some fill. Efforts may be required to improve the compactness of some of the fill.

Any fill left in place should have the exposed subgrade surface compacted followed by proof rolling and examination by geotechnical personnel. This would be part of a program to assess the competency and any identified local anomalies (over size materials) or soft spots should be subsequently excavated, replaced with suitable fill, and compacted. Field verification should be carried out by qualified geotechnical personnel during construction. Detailed recommendations regarding the pavement subgrade preparation is provided in Section 6.12 of this report.

Note that additional soil removals may be required due to possible environmental contamination at the site. This is discussed in a separate report.

Bedrock removal is expected for underground services, underground parking and footing excavations. The excavation operations of bedrock is expected to impart vibrations. Contractors must use techniques and methods to prevent settlement of adjacent ground, structural damage to adjacent buildings and minimize aesthetic impacts (e.g., paint/drywall cracks, pavement cracking). It is recommended that the specifications require that pre-condition surveys of the adjacent structures be completed. Specifications and Tenders submitted by contractors of their proposed methods of excavations, blasting, vibration monitoring, and soil and groundwater management plans in the form of written plans are recommended to be requested by the owner's design consultant team prior to construction to allow adequate time for review and discussion.

The rock at this site is subject to expansion if exposed to air. GHD recommends that any exposed bedrock surface (vertical or horizontal) be covered with a concrete mud slab to minimize the risk of swelling of the bedrock.

6.2 Excavation and Dewatering

All excavations should be completed and maintained in accordance with the Occupational Health and Safety Act (OHSA) requirements. The following recommendations for excavations should be considered to be a supplement to, not a replacement of, the OHSA requirements.

Based on the results of the investigation, overburden soil material within excavation would be considered as 'Type 3 Soils', as defined by the OHSA Regulations for Construction.

Bedrock removal will be required since footing excavations are expected to penetrate to approximately 15 m for the underground parking.

The soil overburdens and the upper level of bedrock that is heavily weathered bedrock (upper 0.2 to 0.7 m) are considered to be Type 3 Soils as per the Occupational Health and Safety Act and should be sloped back at 1:1 or supported by a shoring system. The less weathered rock may either be shored or should be planned to be cut back at a 30 degree from vertical. Sound rock may be planned to be excavated at near vertical.

A rock protection system of rock bolts-mesh-shotcrete may be a third option and can be discussed in more detail in the Final Geotechnical report.

Typically decisions will be required as excavations proceed to address issues such as local fractures, shear zones or weathered areas. These may require treatments ranging from rock bolting to rock bolting with mesh and shotcrete. The Tender and Specifications should allow for unit price submissions from contractors during the Tender and have allowances in the contract.

The excavation of the bedrock will require the use of line drilling in combination with pneumatic or hydraulic breakers such as hoe rams or heavy excavation equipment equipped for rock excavation such as rock grinders. Excavation may involve controlled blasting techniques and/or line drilling. Local by-laws should be consulted to confirm that blasting will be allowed in this area. Line drilling on a closely spaced pattern may also be an option to assist excavation methods and prevent over breakage issues, especially around the perimeter or to create local excavations for elevator pits, footings, etc.

If waterproofing is required, the excavation method and degree of roughness may dictate the excavation method. For example, the use of cutter heads/grinders would assist to have a smooth rock face.

Excavations must be properly planned in advance to ensure the foundations of the adjacent structures and roadways are not undermined during excavation. Any excavation methodology is subject to the laws and blasting restrictions that are in effect for the area.

The excavation operations of bedrock will impart vibrations affecting the surrounding buildings. It is recommended that the specifications require pre-construction condition surveys as well as submittal of plans for excavations, blasting, vibration monitoring, and soil and groundwater management plans.

It is recommended that the client's design team include in the specification package, requirements for the successful contractor to submit written Plans for Excavation as well as Soil and Groundwater Management for review by the client design team.

Surface water and groundwater seepage is expected in the excavated areas, especially within the overburden and weathered rock. Water quantities will depend on seasonal conditions, depth of excavations, and the duration that excavations are left open. Conventional construction dewatering techniques should be taken during construction, such as pumping from sumps and or ditches. Contractors will need to use techniques and methods to minimize disturbance to soils.

GHD did not complete a hydrogeological assessment of this site as part of the scope of work. A hydrogeological assessment is recommended to estimate the extent of dewatering activities and whether a Permit to take water (PTTW) or submission on the Ontario Environmental Activity and Site Registry (EASR) will be required.

Method of disposal of the groundwater in the excavations will depend on the environmental quality of the Site groundwater. The results of the groundwater analysis and recommendations are discussed in a separate report.

6.3 Foundations

The Ontario Building Code (OBC 2012) requires buildings to be designed using Limit States Design values (LSD) of Serviceability Limit States (SLS) and Ultimate Limit States (ULS). It is expected that the foundation of the proposed residential buildings will be bearing on bedrock at an approximate depth of 15 meters below ground surface (mbgs) and will be supported by conventional spread footings.

Based on the recorded conditions within the boreholes, and the founding level of the four level underground, the recommended bearing pressure of the bedrock at this deep depth is 2000 kPa under factored ULS conditions. The factored ULS value includes the geotechnical resistance factor (Φ) of 0.5.

If specific parts of structures or stand-alone elements are founded at higher elevations and will be within the weathered bedrock may need bearing pressures as low as 500 kPa under factored ULS conditions.

For all footings set on bedrock, there is no corresponding SLS value, as settlement is considered to be nil for the footings founded on bedrock.

The minimum founding sizes should be 0.75 m for pad footings and 0.5 m widths for strip footings on bedrock.

Based on the existence of mud seam recorded in the coring of the bedrock, it is our recommendation that rock probing be completed at the time of construction to evaluate the bedrock for mud seams within the footing areas. This "probing" may consist of contractors being required to drill a 50 mm diameter hole, 1.5 m below the base of the footing subgrade. These probe holes should then be assessed by the Geotechnical Engineer to confirm the absence/presence of mud seams. If mud seams are verified then remedial options may include deepening the footings down to the underside of the mud seam if the mud seam is deemed significant by the Geotechnical Engineer. Structural engineers should determine the remedial approach for foundation support if this over excavation is required. Remedial approach options may include replacing the excavated rock with bulk concrete backfill, or extending the foundation walls or piers or other structural solutions. Designers/Owners should account for this work and unit rates for over excavation and remedial approach in the Tender and Specification documents. If the mud seam are greater than 1 m below underside of footing level and/or thin enough then the Geotechnical assessment during construction may allow the mud seam to be left in-place.

Excavations for footings and other adjacent structures (sump pits, storm water tank, sewer trenches, etc.) set within bedrock at various levels, including step footings, should be positioned such that they do not encroach within the 1V:1H zone of influence of an adjacent footing. Step footings should be designed in a manner that the average slope of the benching is no steeper than 1V:2H along the length and the height of the bench is less than 0.3 m.

6.4 Floor Slabs

Conventional slab-on-grade construction is considered suitable for the proposed buildings. We understand that the building will have light floor loadings only, i.e., considered to be less than 24 kPa. Higher loading requirements will require additional consultation and analysis.

Preparation of the subgrade as discussed in Section 6.1 and 6.2 would the placement of the skim slab of concrete to minimize risk of swelling. Removal of any unsuitable overburden or bedrock materials will be required to expose suitable subgrade and/or the design subgrade level. Any local weakened areas should be excavated and replaced with suitable fill and compacted. Field verification should be carried out by geotechnical personnel during construction.

A layer consisting of Granular 'A' at least 200 mm thick should be placed immediately below the floor slabs to support the slab-on-grade. This layer should be compacted to 100 percent of its SPMDD and placed on approved subgrade surfaces.

If floor coverings are to be used on slab-on-grades then, a vapour barrier is recommended to be incorporated beneath the slab and should be specified by the architect. Floor toppings may also be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer's specifications and requirements should be consulted and procedures outlined in the specifications should be followed.

The slabs should not be tied into the foundation walls. The placement of construction and control joints in the concrete should be in accordance with generally accepted practice.

6.5 Frost Protection

All exterior footings associated with the heated building must be provided with at least 1.5 m of soil cover or its equivalent in insulation, in order to provide adequate protection against detrimental frost action. This cover depth requirement must be increased to 1.8 m for footings for unheated or isolated structures such as signs, entrance canopy, or piers.

Billings shale is considered frost susceptible bedrock, therefore, the exposed surfaces to support foundations must be protected by Contractors against freezing. It is recommended that the bedrock surface be covered following approval with a lean concrete mud slab.

6.6 Seismic Site Classification

In accordance with OBC-2012, the building and its structural elements must be designed to resist a minimum earthquake force based upon the borehole drilling program that was undertaken as part of this Preliminary Geotechnical Investigation, this Site is recommended to have a Site Classification 'C', with respect to Table 4.1.8.4.A of the National Building Code of Canada 2010.

Higher site class may be available, but would require confirmation by additional investigation using geophysical methods, in order to measure the shear wave velocity within the soil and rock mass.

6.7 Lateral Earth Pressure

If open cut and oversize excavations are used such that backfill is placed against permanent basement walls then these should be designed to withstand lateral earth pressures. There may also be retaining walls at grade changes with adjacent properties. The walls should be designed for lateral pressures resulting from the following sources:

- Unit weight of the backfilled soil
- Temporary and permanent vertical loads on the completed ground surface

6.7.1 Static Conditions

The following soil parameters can be used for designing of the retaining walls for lateral earth pressures.

Table 6.1 Soil Parameters and Earth Pressure Coefficients

Soil	Density 'γ' (kN/m^3)	Angle of internal Friction	Rankin Earth Pressure Coefficients[(1)] [(2)]		
		φ	K_a	K_0	K_p
Compacted granular backfill such as an OPSS "Granular BI or BII" type product	21	32	0.31	0.47	3.3
Existing Fill	19	30	0.33	0.50	3.0
Native Soils	18	28	0.36	0.53	2.8
Notes: [(1)] Assumes level/flat backfill surface [(2)] For Temporary soils support shoring is required, designers should refer to the CFEM for design assistance					

For yielding walls the active earth pressure coefficients K_a is recommended to be used.

For non-yielding wall the at-rest K_o should be used.

The resultant of the applicable static or at-rest force is assumed to act at 1/3H above the base of the wall where H is the height of the wall for the permanent wall with free drain backfill material.

It is noted that for the temporary shoring system that will support the existing fill and upper weather bedrock Section 26.10.3 of CFEM 2006 should be used by designers regarding the distribution of the forces. The soils encountered in the boreholes consist mainly of granular soils. If the shoring must support existing structures then the stiffness of the shoring system must be addressed by the designers and K_o is recommended. The contractor must also ensure installation procedures minimize risk of lateral movements especially where structures are being supported by the shoring system. Hydrostatic pressures should be considered in the design of the shoring systems

These statements are based on the assumption that there is a perimeter drainage system installed at the base of the retaining walls draining under gravity to a frost free outlet, to prevent the build-up of hydrostatic pressure behind the wall; hydrostatic pressures may not be included in the design.

6.7.2 Dynamic Condition

For a seismic event, the Mononobe-Okabe (M-O) equations, shown in Section 24.9 of CFEM-2006 are recommended for design implications for permanent earth retaining walls. In these formulas, there are both geotechnical and geometric components. The geotechnical parameters of imported granular backfill are provided in the above table.

Assuming:

- k_h = (PGA) – Corrected PGA is 0.288 for Site Class C for this area. A copy of the 2015 NBCC Seismic Hazard Calculation is provided in Appendix C.
- k_v = typically a range of 2/3 x k_h to 1/3 k_h is considered but a value closer to 2/3 x k_h is recommended.

The total active thrust under seismic loading (P_{ae}) is recommended to be expressed as follows:

$P_{ae} = ½ K_{ae} \gamma H^2 \times (1- k_v)$

This includes both the active pressures under static (P_a) as well as the increased force due to seismic or simply as follows:

$P_a = ½ K_a \gamma H^2$

Therefore, the seismic force (P_e) is simply the difference between the P_{ae} and P_a, or:

$P_e = P_{ae} – P_a$

The active force under static conditions is assumed to act at a point of (0.3 x H) above the base and the seismic force is assumed to act near (0.6 x H) above the base, where H is the height of the wall. Therefore the point of applying P_{ae} may be calculated from the following:

$h = [(0.33H\times P_a) + (0.6H \times P_e)]/ P_{ae}$

In reducing the above formula for this site, we recommend values of $K_{ae} \times (1-k_v) = 0.48$ and $K_a = 0.31$. This condition applies for open cut excavations and granular backfill behind the walls, assuming there is permanent drainage system in place.

6.7.3 Basement Walls against Bedrock

If the bedrock is cut vertical in the sound bedrock at depth, the rock pressures may be less. This will be discussed under further mandates for Final Geotechnical Reports.

6.8 Permanent Drainage

6.8.1 Underfloor Drainage

Under floor drains are recommended for structures with underground levels.

6.8.2 Perimeter drainage

Perimeter drainage around the exterior of the walls of the proposed buildings is recommended. The drain should be connected to a frost-free outlet for year round drainage.

6.9 Corrosion Potential of Soils

Analytical testing is being carried out one groundwater sample collected to determine corrosion potential of the groundwater at the site. The selected groundwater samples were tested for pH, chlorides, sulphates, and conductivity. The test results are summarized in the following table.

Table 6.2 Corrosion Parameter Results

Sample ID	BH5
pH	7.33
Conductivity (µS/cm)	1140
Sulphate (mg/L)	8
Chloride (mg/L)	26

The American Water Works Association (AWWA) publication 'Polyethylene Encasement for Ductile-Iron Pipe Systems' ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. Soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils are considered to be potentially corrosive to cast iron pipe. Therefore protective measures, such as sacrificial cathode protection should be considered.

Table 3 of the Canadian Standards Association (CSA) document A23.1-04/A23.2-04 'Concrete Materials and Methods of Concrete Construction/Methods of Test and Standard Practices for Concrete' divides the degree of exposure into the following three classes:

Table 6.3 Classes of Exposure

Degree (Class) of Exposure	Sulphate (SO_4) in Groundwater Sample (mg/L)
Very Severe (S-1)	>10,000
Severe (S-2)	1,500 – 10,000
Moderate (S-3)	150 – 1,500

A review of the analytical test results shows the sulphate content in the tested sample was found to be less than 150 mg/L. Based upon the test results, the degree of exposure of the subsurface concrete structures to sulphate attack is low. Therefore, normal General Use (GU) hydraulic cement can be used for the below grade concrete structures.

6.10 Building Backfill

Where it is required to have the placement and compaction of the granular materials and these will support the floor slabs, foundations, pavement or any interior backfill then these materials must be treated as Engineered Fill.

6.10.1 Engineered Fill

The fill operations for Engineered Fill must satisfy the following criteria:

- Engineered Fill must be placed under the continuous supervision of the Geotechnical Engineer.

- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade proof rolled, and approved. Any deficient areas should be repaired.
- Prior to the placement of Engineered Fill, the source or borrow areas for the Engineered Fill must be evaluated for its suitability. Samples of proposed fill material must be provided to the Geotechnical Engineer and tested in the geotechnical laboratory for Standard Proctor Maximum Dry Density (SPMDD) and grain size, prior to approval of the material for use as Engineered Fill. The Engineered Fill must consist of environmentally suitable soils (as per industry standard procedures of federal or provincial guidelines/regulations), free of organics and other deleterious material (building debris such as wood, bricks, metal, and the like), compactable, and of suitable moisture content so that it is within -2 percent to +0.5 percent of the Optimum Moisture as determined by the Standard Proctor test. Imported granular soils meeting the requirements of Granular 'A', or 'B' Type II OPSS 1010 criteria would be suitable.
- The Engineered Fill must be placed in maximum loose lift thicknesses of 0.2 m. Each lift of Engineered Fill must be compacted with a heavy roller to 100 percent SPMDD.
- Field density tests must be taken by the Geotechnical Engineer, on each lift of Engineered Fill. Any Engineered Fill, which is tested and found to not meet the specifications, shall be either removed or re-compacted and retested.

6.10.2 Exterior Foundation Wall Backfill

Where applicable and/or if necessary, any backfill placed against the foundation walls should be free draining granular materials meeting the grading requirements of OPSS 1010 for Granular 'B' Type I specifications up to within 0.3 m of the ground surface. The upper 0.3 m should be a low permeable soil to reduce surface water infiltration. Foundation backfill should be placed and compacted as outlined below.

- Free-draining granular backfill should be used for the foundation wall.
- Backfill should not be placed in a frozen condition, or placed on a frozen subgrade.
- Backfill should be placed and compacted in uniform lift thickness compatible with the selected construction equipment, but not thicker than 0.2 m. Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures.
- At exterior flush door openings the underside of sidewalks should be insulated, or the sidewalk should be placed on frost walls to prevent heaving. Granular backfill should be used and extended laterally beneath the entire area of the entrance slab. The entrance slab should slope away from the building.
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to at least 98 percent of its SPMDD.
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 percent of its SPMDD.
- In areas on the building exterior where an asphalt or concrete pavement will not be present adjacent to the foundation wall, the upper 0.3 m of the exterior foundation wall backfill should be a low permeable soil to reduce surface water infiltration.

- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall.

6.11 Underground Services

6.11.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials that may be associated with the development. Note: service cuts in Billings shale will need the rock cut faces to be covered with a concrete mud slab to minimize risk of swelling of the bedrock.

- Bedding for buried utilities should be OPSS Granular 'A', and placed in accordance with City of Ottawa specifications.
- The cover material should be a sand material or Granular 'A' and the dimensions should comply with City of Ottawa standards.
- The bedding material and cover materials should be compacted as per City of Ottawa standards and to at least 95 percent of its SPMDD.
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.11.2 Service Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches under pavement areas, the backfill should be placed and compacted in uniform thickness compatible with the selected compaction equipment and not thicker than 200 mm. Each lift should be compacted to a minimum of 95 percent SPMDD.
- To reduce the potential for differential settlement and frost heave the excavation sides should have frost tapers as per OPSD 800 series which essentially indicates that there should be a back slope of 10:1 (H:V) within the frost zone below finished grade.

6.12 Pavement Sections

Access driveways and parking areas are expected to be constructed over existing fill or bedrock. In order to prepare the site for the pavement area, it is necessary that the area be stripped of any existing cover materials such as surficial topsoil and associated root-mat other deleterious materials deemed unsuitable by geotechnical personnel to expose a suitable subgrade. The exposed subgrade should be proof rolled in the presence of a Geotechnical Engineer. Any areas where "soft spots", rutting, local anomalies, or appreciable deflection are noted should be excavated and replaced with suitable fill and use of geotextiles may be warranted for strength improvement. The fill should be compacted to at least 95 percent of its SPMDD.

The pavement sections described in the table below are recommended for areas subjected to parking lot and access road. Pavement materials and workmanship should conform to the appropriate Ontario Provincial Standard Specifications (OPSS).

Table 6.4 Recommended Pavement Structure

Pavement Layer	Minimum Thickness	Heavy Duty (Access Roads)
HL3 Asphalt	50 mm	40 mm
HL8 Asphalt	n/r	50 mm
Granular 'A' Base Course	150 mm	150 mm
Granular 'B', Type II Sub-Base Course	300 mm	450 mm

In order to accommodate the recommended thicknesses, designers will need to review grades and determine where stripping or filling is necessary. Pavement materials and workmanship should conform to the appropriate OPSS.

Minimum Performance Grade (PG) at 58 – 34 should be used at this Site.

Drainage of the pavement layers is important. The subgrade surface and each layer of the pavement section should be provided with a suitable cross fall (approximately 2 percent) to prevent water from ponding on the pavement surface and beneath the pavement layers. Surface runoff should be directed to storm sewers, or allowed to flow into ditches.

Where the new pavement abuts existing and the subgrade levels vary between the two areas, then a frost transition should be integrated into the subgrade with a 10:1 slope in the subgrade. Sufficient field-testing should be carried out during construction to assess compaction of each lift of the pavement layers. This should be accompanied by laboratory testing of the granular and asphalt materials. All granular base course materials should be compacted to 100 percent of its SPMDD.

Annual or regular maintenance will be required to achieve maximum life expectancy. Generally, the asphalt pavement maintenance will involve crack sealing and repair of local distress.

It should be noted that the pavement sections described within this report represent end-use conditions only, which includes light vehicular traffic and occasional garbage or service trucks. It may be necessary that these sections be temporarily over-built during the construction phase to withstand larger construction loadings such as loaded dump trucks or concrete trucks.

6.13 Construction Field Review

The recommendations provided in this report are based on an adequate level of construction monitoring being conducted during construction phase of the proposed building. GHD requests to be retained to review the drawings and specifications, once complete, to verify that the recommendations within this report have been adhered to, and to look for other geotechnical problems. Due to the nature of the proposed development, an adequate level of construction monitoring is considered to be as follows:

- Prior to construction of footings, the exposed foundation subgrade should be examined by a Geotechnical Engineer or a qualified Technologist acting under the supervision of a Geotechnical Engineer, to assess whether the subgrade conditions correspond to those encountered in the boreholes, and the recommendations provided in this report have been implemented.

- A qualified Technologist acting under the supervision of a Geotechnical Engineer should monitor placement of Engineered Fill underlying floor slabs.
- Backfilling operations should be conducted in the presence of a qualified Technologist on a part time basis, to ensure that proper material is employed and specified compaction is achieved.
- Placement of concrete should be periodically tested to ensure that job specifications are being achieved.

7. Limitation of the Investigation

This report is intended solely for Ottawa Community Housing Corporation and other party explicitly identified in the report and is prohibited for use by others without GHD's prior written consent. This report is considered GHD's professional work product and shall remain the sole property of GHD. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to GHD. Client shall defend, indemnify and hold GHD harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of Geotechnical Engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. 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.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, GHD is the Geotechnical Engineer of record. It is recommended that GHD be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the test hole locations only. The subsurface conditions confirmed at these test locations may vary at other locations. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction, which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

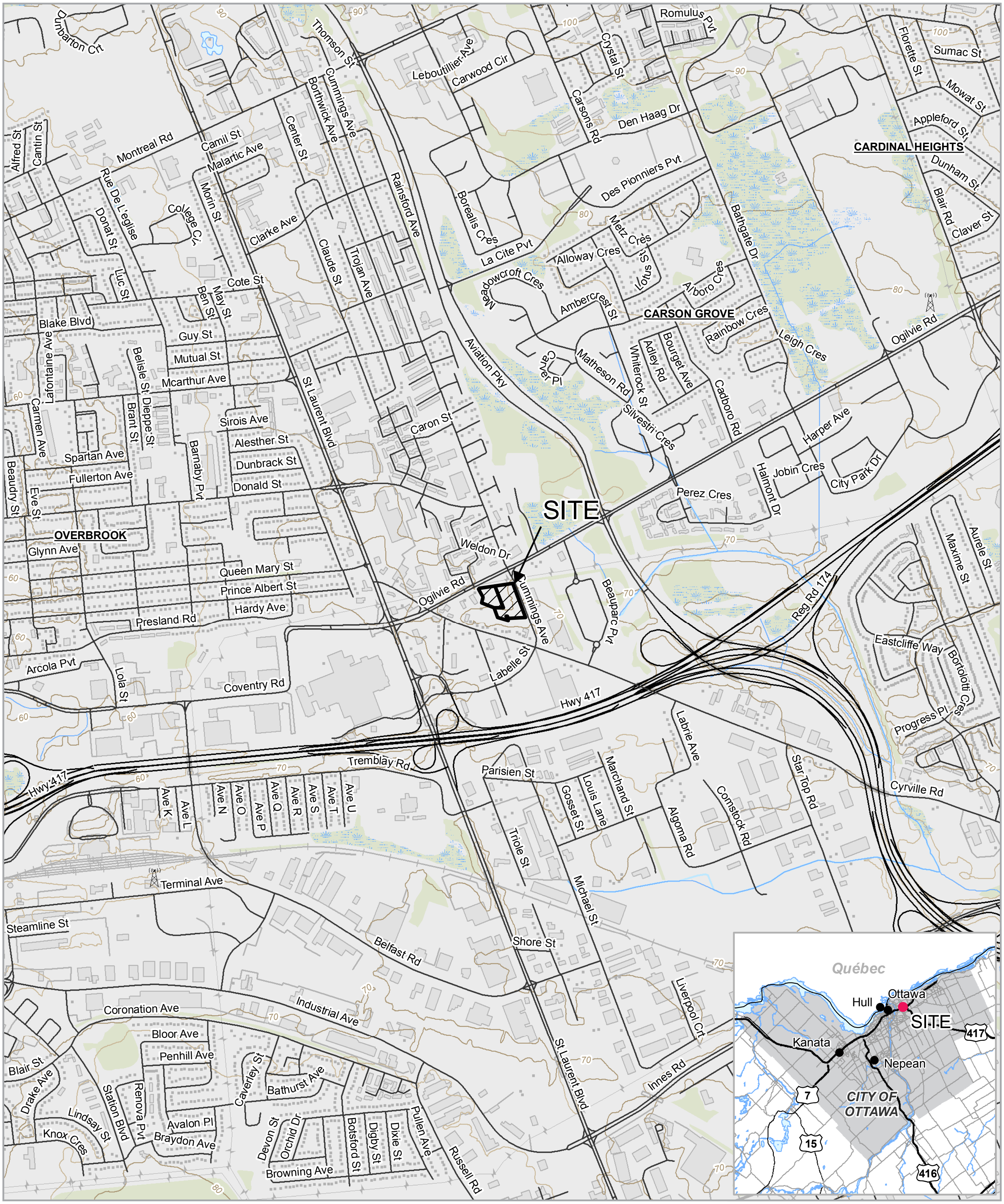
All of Which is Respectfully Submitted,

GHD

Ryan Vanden Tillaart, EIT

LICENSED PROFESSIONAL ENGINEER
11201061-A
Nov. 7, 2019
B. VAZHBAKHT
100171356
PROVINCE OF ONTARIO

Bahareh Vazhbakht, P. Eng.



Paper Size ANSI A

0 140 280 420 560

Meters

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 18N

N

GHD

6770967 CANADA INC.
1098 OGILVIE ROAD AND 1178 CUMMINGS AVENUE
OTTAWA, ON
GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT

SITE LOCATION MAP

Project No. **11201061-A1**
Revision No. **-**
Date **Aug 12, 2019**

FIGURE 1

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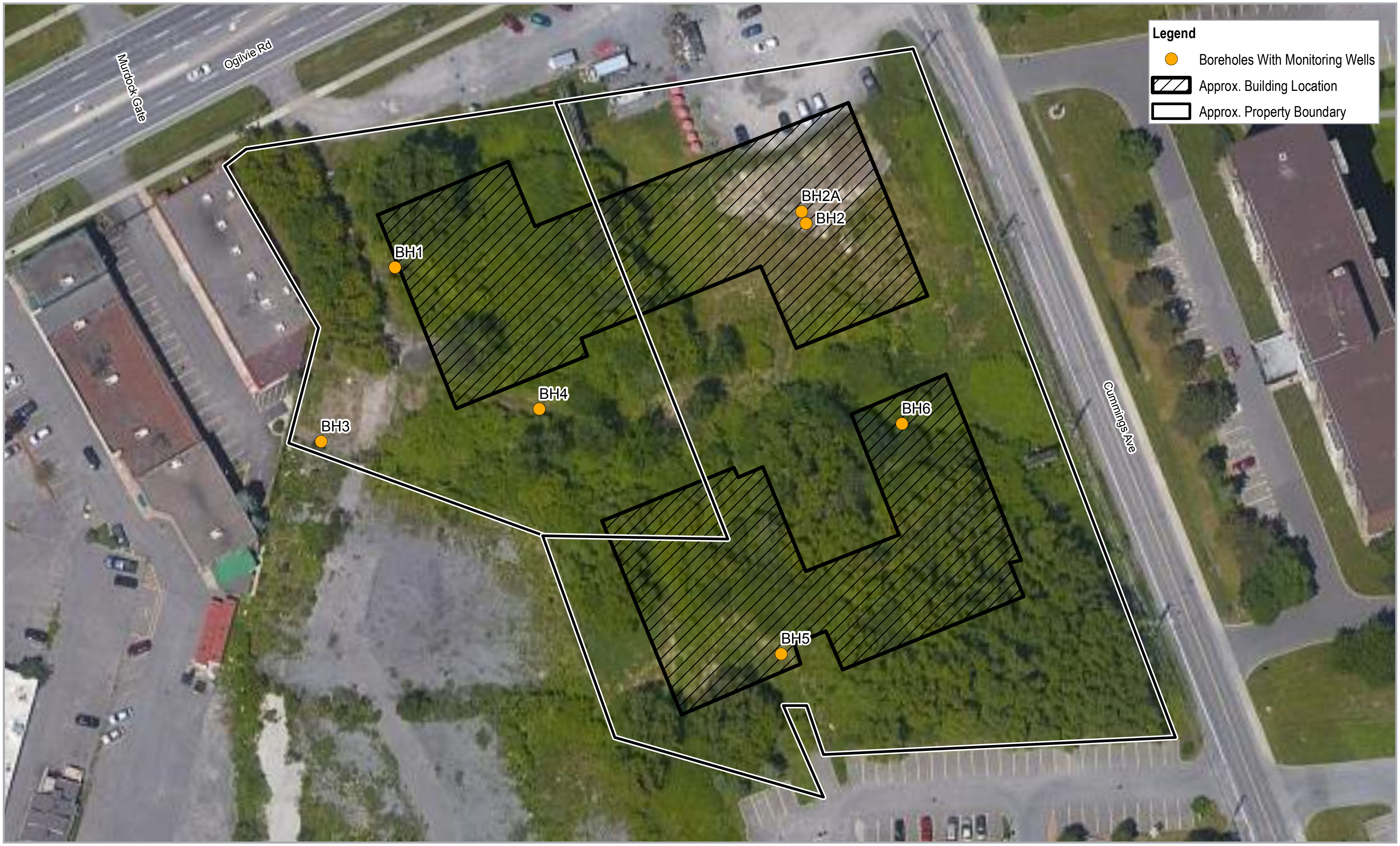
Print date: 12 Aug 2019 - 16:14

Data source: MNRF NRVIS, 2017. Produced by GHD under licence from Ontario Ministry of Natural Resources and Forestry, © Queen's Printer 2019.



Paper Size ANSI A

0 7 14 21 28

Meters

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 18N

N

6770967 CANADA INC.
1098 OGILVIE ROAD AND 1178 CUMMINGS AVENUE
OTTAWA, ON
GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT

BOREHOLE LOCATION PLAN

Project No. **11201061-A1**
Revision No. **-**
Date **Oct 9, 2019**

FIGURE 2

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Print date: 09 Oct 2019 - 11:27

Data source: Image ©2019 Google, Imagery date: 2018/06/08 .

Appendices

Appendix A
Borehole Logs and Notes on Boreholes

REFERENCE No.: 11201061 ENCLOSURE No.: 1

BOREHOLE No.: BH1
ELEVATION: 100.37 m

BOREHOLE LOG

CLIENT: 6770967 Canada Inc.
PROJECT: 1098 Ogilvie Road
LOCATION: 1098 Ogilvie Road, Ottawa, ON
DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht
DATE (START): 23 September 2019 DATE (FINISH): 24 September 2019

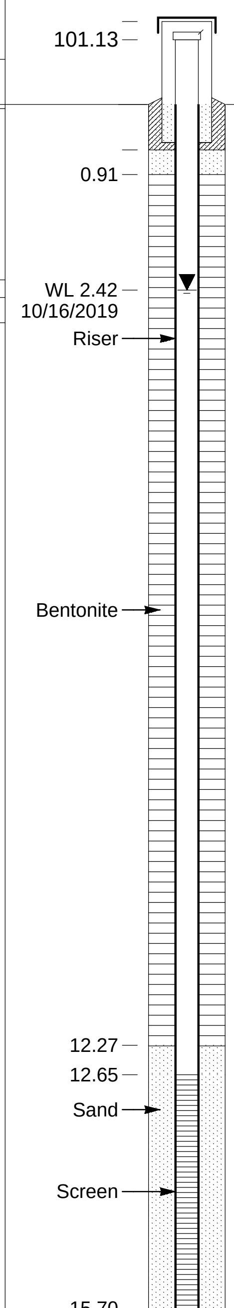
LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock
	100.37	GROUND SURFACE
	100.3	**ASPHALT**
		FILL - Gravelly silty sand, some gravel, compact, brown, damp
	98.1	**SANDY CLAY** - very stiff, grey, moist. Spoon refusal encountered at 2.5 mbgs
	97.9	**WEATHERED BEDROCK - SHALE** Auger refusal encountered at 2.8 mbgs
	97.5	**BEDROCK - SHALE** interbedded limestone, highly weathered and fractured surface, black and grey, fair quality becoming excellent with depth
		Mud seam
		Thicker limestone bedding
		Mud seam
	84.7	Borehole terminated at 15.7 mbgs

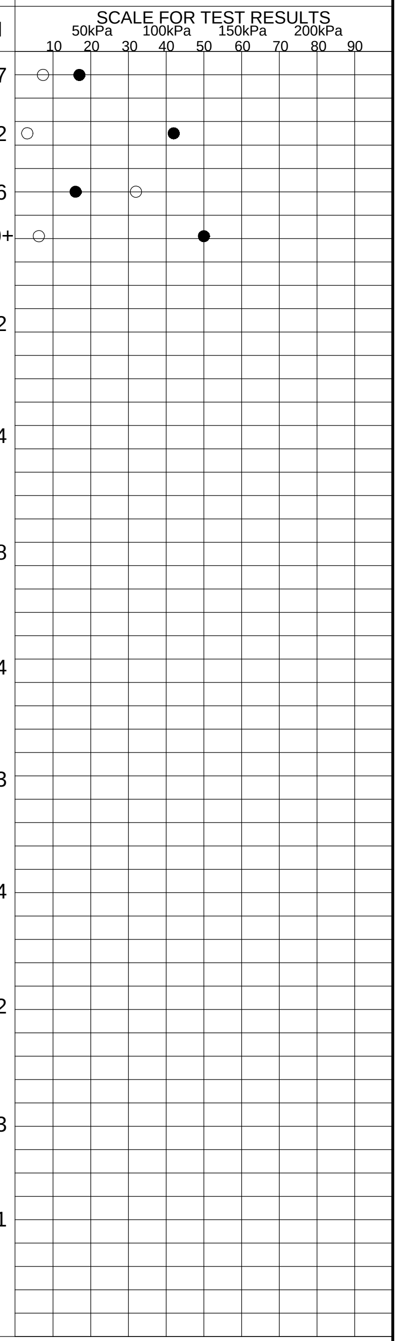
Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

MONITOR WELL



SAMPLE DATA

Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
SS1	67		17
SS2	71		42
SS3	42		16
SS4	89		50+
RC1	100		52
RC2	100		64
RC3	100		88
RC4	100		84
RC5	100		93
RC6	100		84
RC7	100		92
RC8	100		93
RC9	100		91



NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11201061 ENCLOSURE No.: 2

BOREHOLE No.: BH2
ELEVATION: 100.81 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: 6770967 Canada Inc.
PROJECT: 1098 Ogilvie Road
LOCATION: 1098 Ogilvie Road, Ottawa, ON
DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht
DATE (START): 24 September 2019 DATE (FINISH): 24 September 2019

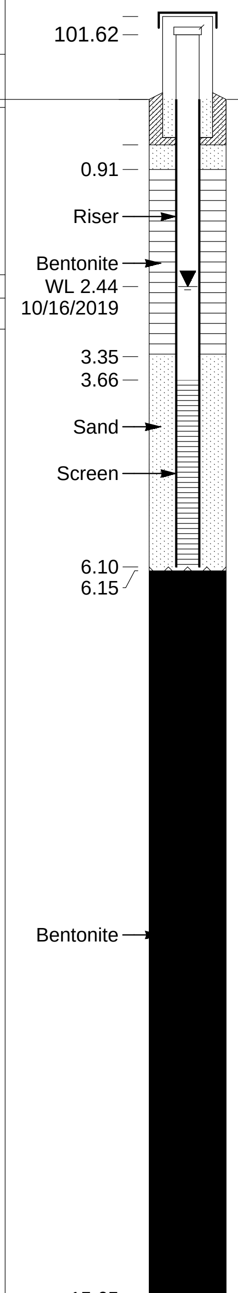
LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

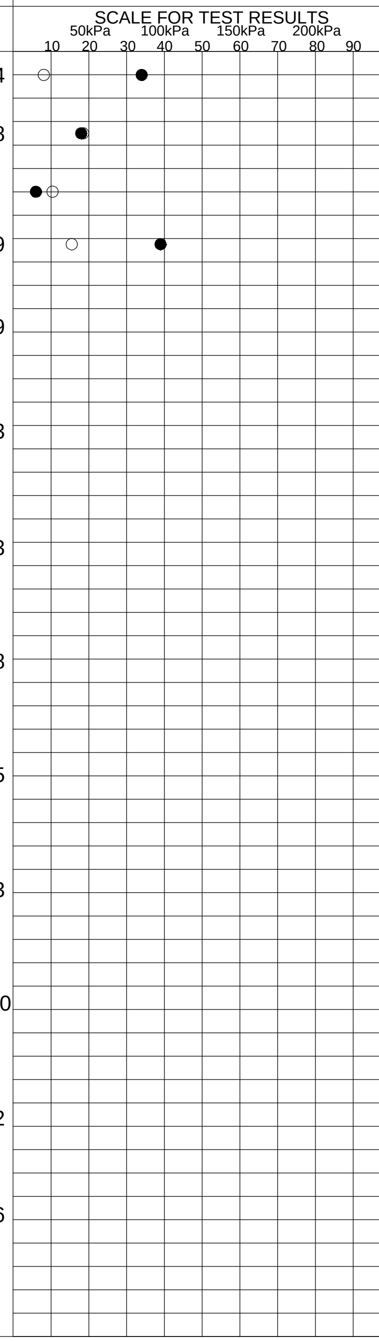
Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	100.81	GROUND SURFACE				
	100.7	**TOPSOIL**	SS1	83		34
		FILL - Sand and Silt, compact, dark grey, damp	SS2	21		18
			SS3	38		6
	98.5	**SANDY CLAY** - trace gravel, very stiff, dark grey, moist	SS4	33		39
	98.2 / 97.8	**WEATHERED BEDROCK - SHALE** Spoon refusal encountered at 2.7 mbgs. Auger refusal encountered at 3.0 mbgs	RC1	100		19
		BEDROCK - SHALE interbedded limestone, highly weathered and fractured surface, black and grey, very poor quality becoming excellent with depth	RC2	100		53
			RC3	100		93
			RC4	100		88
		Mud seam Mud seam	RC5	100		65
			RC6	100		93
		Thicker limestone bedding	RC7	100		100
			RC8	100		92
			RC9	100		96
	85.2	Borehole terminated at 15.7 mbgs				

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

MONITOR WELL



SCALE FOR TEST RESULTS



NOTES:
mbgs: meters below ground surface

BOREHOLE No.: **BH2A**

ELEVATION: 100.93 m

BOREHOLE LOG

CLIENT: 6770967 Canada Inc.

PROJECT: 1098 Ogilvie Road

LOCATION: 1098 Ogilvie Road, Ottawa, ON

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht

DATE (START): 24 September 2019 DATE (FINISH): 24 September 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		MONITOR WELL	SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK		State	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	100.93		GROUND SURFACE	101.79			%	ppm	N
3.0	97.9		Auger to 3.0 mbgs for direct well install	0.30; Riser; Bentonite; 1.17; 1.47; Sand; Screen; WL 2.44 10/16/2019; 3.00					

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11201061 ENCLOSURE No.: 4

BOREHOLE No.: BH3

ELEVATION: 100.18 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: 6770967 Canada Inc.

PROJECT: 1098 Ogilvie Road

LOCATION: 1098 Ogilvie Road, Ottawa, ON

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht

DATE (START): 23 September 2019 DATE (FINISH): 23 September 2019

LEGEND

- ☒ SS Split Spoon
- GS Auger Sample
- ▨ ST Shelby Tube
- ▼ Water Level
- ○ Water content (%)
- ├─┤ Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK	MONITOR WELL	State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	100.18	GROUND SURFACE	100.92					
	100.1	**TOPSOIL**						
		FILL - Sand, trace to some gravel, compact, brown, damp	Riser	☒	SS1	50		29
1.0				☒	SS2	50		19
	98.6	Spoon refusal encountered at 1.6 mbgs	1.52; WL 1.72 10/16/2019 ▼	☒	SS3	100		50+
2.0	97.9	**WEATHERED BEDROCK - SHALE** Auger refusal encountered at 2.3 mbgs	2.29; 2.59; Sand; Screen					
3.0					RC1	100		30
4.0	96.4	**BEDROCK - SHALE** interbedded limestone, highly weathered and fractured surface, black and grey, very poor to poor quality	3.81					
5.0		Borehole terminated at 3.8 mbgs						

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11201061 ENCLOSURE No.: 5

BOREHOLE No.: BH4

ELEVATION: 100.76 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: 6770967 Canada Inc.

PROJECT: 1098 Ogilvie Road

LOCATION: 1098 Ogilvie Road, Ottawa, ON

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht

DATE (START): 23 September 2019 DATE (FINISH): 23 September 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Monitor Well	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	100.76	GROUND SURFACE	101.50				
	100.7	**TOPSOIL**		SS1	75		17
	100.2	**FILL** - Silty sand, some gravel, compact, brown, damp	Riser				
1.0	99.7	**SILTY SAND** - some gravel, very dense, brown, damp	1.24	SS2	83		69
	99.3	**WEATHERED BEDROCK - SHALE** Auger refusal encountered at 1.5 mbgs	Bentonite; 1.91				
2.0			2.29; WL 2.59 10/16/2019	RC1	100		44
3.0		**BEDROCK - SHALE** interbedded limestone, highly weathered and fractured surface, black and grey, very poor to poor quality Mud seam	Sand; Screen	RC2	100		47
4.0	97.0	Borehole terminated at 3.8 mbgs	3.81				

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

REFERENCE No.: 11201061 ENCLOSURE No.: 6

BOREHOLE No.: BH5

ELEVATION: 99.47 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: 6770967 Canada Inc.

PROJECT: 1098 Ogilvie Road

LOCATION: 1098 Ogilvie Road, Ottawa, ON

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht

DATE (START): 25 September 2019 DATE (FINISH): 25 September 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10, 20, 30, 40, 50, 60, 70, 80, 90

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
	99.47	GROUND SURFACE				
	99.4	**TOPSOIL**	SS1	83		25
1.0	98.4	**FILL** - Silty sand, some gravel, compact, brown, damp; Spoon refusal encountered at 1.0 mbgs	SS2	100		50+
	98.1	**WEATHERED BEDROCK - SHALE** Auger refusal encountered at 1.4 mbgs	RC1	100		30
3.0		**BEDROCK - SHALE** interbedded limestone, highly weathered and fractured surface, black and grey, very poor quality becoming excellent with depth	RC2	100		88
5.0		Mud seam	RC3	100		90
6.0			RC4	100		90
8.0			RC5	100		63
9.0			RC6	100		100
11.0			RC7	100		98
12.0		Thicker limestone bedding	RC8	100		100
14.0			RC9	100		97
15.0			RC10	100		96
16.0	83.7	Borehole terminated at 15.8 mbgs				

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

MONITOR WELL: 100.23; 0.61; Riser; Bentonite; WL 2.26 10/16/2019; 2.74; 3.05; Sand; Screen; 6.10; 6.15; Bentonite; 15.80

NOTES:
mbgs: meters below ground surface

BOREHOLE LOG 11201061-A1-BH LOGS.GPJ INSPEC_SOL.GDT 24/10/19

BOREHOLE No.: BH6

ELEVATION: 99.92 m

BOREHOLE LOG

CLIENT: 6770967 Canada Inc.

PROJECT: 1098 Ogilvie Road

LOCATION: 1098 Ogilvie Road, Ottawa, ON

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B.Vazhbakht

DATE (START): 24 September 2019 DATE (FINISH): 25 September 2019

LEGEND

- SS Split Spoon
- GS Auger Sample
- ST Shelby Tube
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

SCALE | STRATIGRAPHY | MONITOR WELL | SAMPLE DATA

Depth BGS (meters)	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
0.0	99.92	GROUND SURFACE
	99.8	**TOPSOIL**
	98.9	**FILL** - Silty sand, some clay, trace gravel, loose, brown, damp
	98.1	**SILTY SAND** - some clay, trace gravel, very stiff, dark grey, damp. Spoon refusal encountered at 1.8 mbgs
	97.9	**WEATHERED BEDROCK - SHALE** Auger refusal encountered at 2.0 mbgs
		BEDROCK - SHALE interbedded limestone, highly weathered and fractured surface, black and grey, very poor quality becoming excellent with depth
~12.0		Thicker limestone bedding
~12.5		Mud seam
~15.0		Mud seam
15.5	84.4	Borehole terminated at 15.5 mbgs

Monitor well: 100.61 (top of riser); 0.61; WL 1.15 10/16/2019; Riser; Bentonite; 12.19; 12.50; Sand; Screen; 15.54

Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
SS1	67		9
SS2	50		8
SS3	80		50+
RC1	100		16
RC2	100		72
RC3	100		92
RC4	100		84
RC5	100		91
RC6	100		89
RC7	100		100
RC8	100		97
RC9	100		100
RC10	100		90

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey sols is measured by the value of undrained shear strength (Cu).

Classification (Unified system)

Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	>300 mm		

Terminology

"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

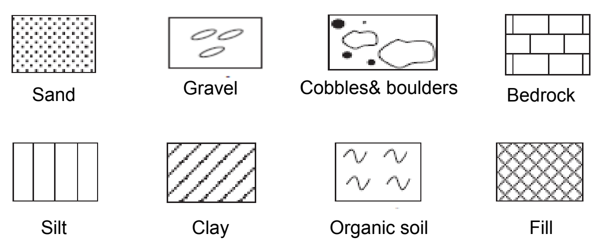
Relative density of granular soils	**Standard penetration index "N" value** (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	**Undrained shear strength (Cu)** (P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation

"RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND



Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environmental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
N_c: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressure meter
k: Permeability
ABS: Absorption (Packer test)

LABORATORY TESTS:

I_p: Plasticity index
W_l: Liquid limit
Wp: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
γ: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor

Appendix B
Laboratory Testing Results

Moisture Content of Soils
(ASTM D2216)

Client: 6770967 Canada Inc.
Project: 1098 Ogilvie Road and 1178 Cummings Avenue
Location: Ottawa, On

Lab No.: G-19-009
Project No.: 11201061

Apparatus Used for Testing

Oven no.: 1 Scale no.: 1

Sample No.	BH1-SS3	BH2-SS4	BH6-SS2	BH5-SS2	BH2-SS1	BH2-SS2	BH2-SS3	BH4-SS2
Container no.	S37	S40	S26	S12	S13	S11	S14	S12
Mass of container + wet soil (g)	71.6	82.3	80.2	73.7	84.5	61.1	85.4	94.5
Mass of container + dry soil (g)	59.5	74.2	68.4	70.2	79.8	54.9	79.4	90.0
Mass of container (g)	21.7	21.8	21.4	21.5	21.5	21.4	21.8	21.4
Mass of dry soil (g)	37.8	52.4	47.0	48.7	58.3	33.5	57.6	68.6
Mass of water (g)	12.1	8.1	11.8	3.5	4.7	6.2	6.0	4.5
Moisture content (%)	32.0	15.5	25.1	7.2	8.1	18.5	10.4	6.6

Sample No.	BH6-SS1	BH6-SS3	BH1-SS1	BH1-SS4	BH1-SS2			
Container no.	S39	S35	S29	S15	S25			
Mass of container + wet soil (g)	80.8	81.0	98.8	73.7	95.7			
Mass of container + dry soil (g)	74.4	74.0	93.5	70.6	93.4			
Mass of container (g)	21.3	21.5	21.5	21.6	21.5			
Mass of dry soil (g)	53.1	52.5	72.0	49.0	71.9			
Mass of water (g)	6.4	7.0	5.3	3.1	2.3			
Moisture content (%)	12.1	13.3	7.4	6.3	3.2			

Remarks:

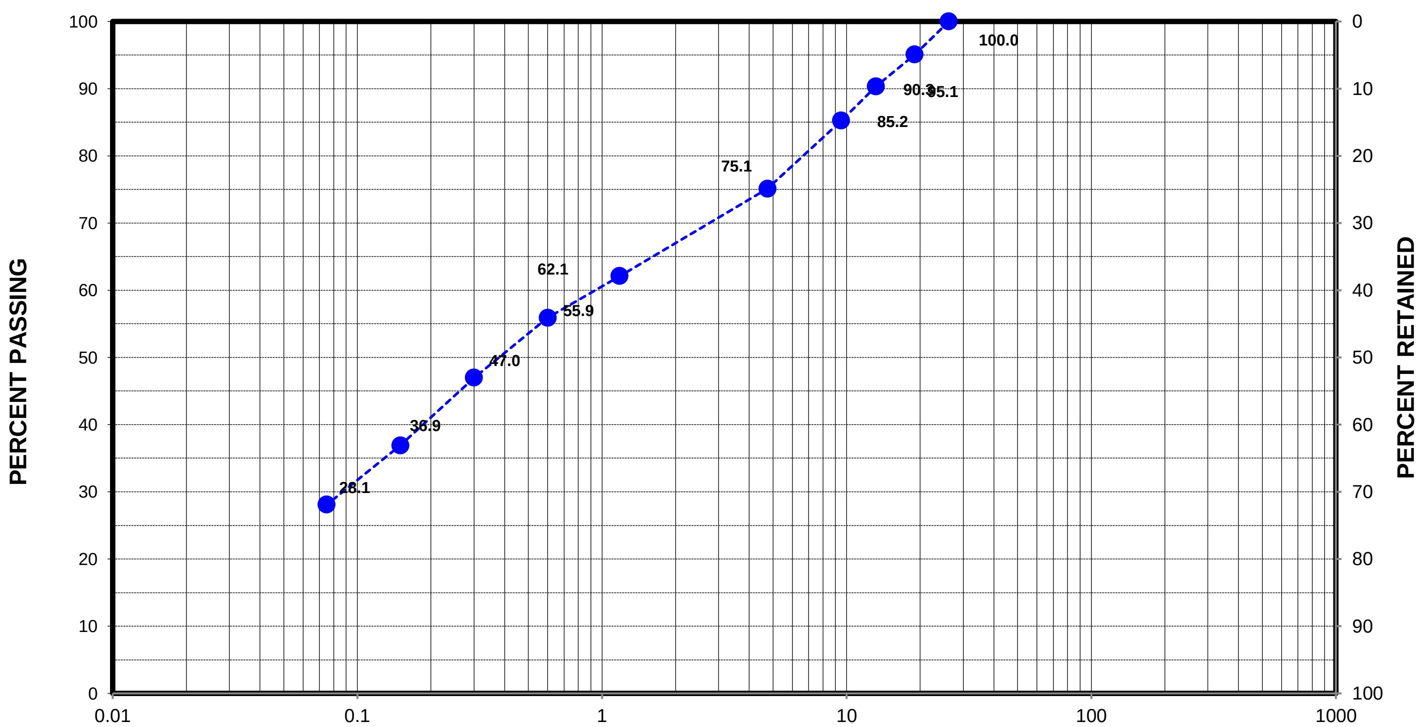
Performed by: A. Elhaddad **Date:** October 18, 2019

Verified by : [signature] **Date:** October 24, 2019

SIEVE ANALYSIS

CLIENT:	9770967 Canada Inc.	**LAB No.:**	G-19-009
PROJECT/SITE:	1098 Ogilvie Road/1178 Cummings Avenue	**PROJECT No.:**	11201061
BOREHOLE:	BH2	**SAMPLE:**	SS1
DEPTH:	0.0m - 0.6m	**SAMPLE DATE:**	September 23/24, 2019

SIEVE SIZE (mm)	SAMPLE % PASSING	
26.2	100.0	
19.0	95.1	
13.2	90.3	
9.5	85.2	
4.75	75.1	
1.18	62.1	
0.600	55.9	
0.300	47.0	
0.150	36.9	
0.075	28.1	



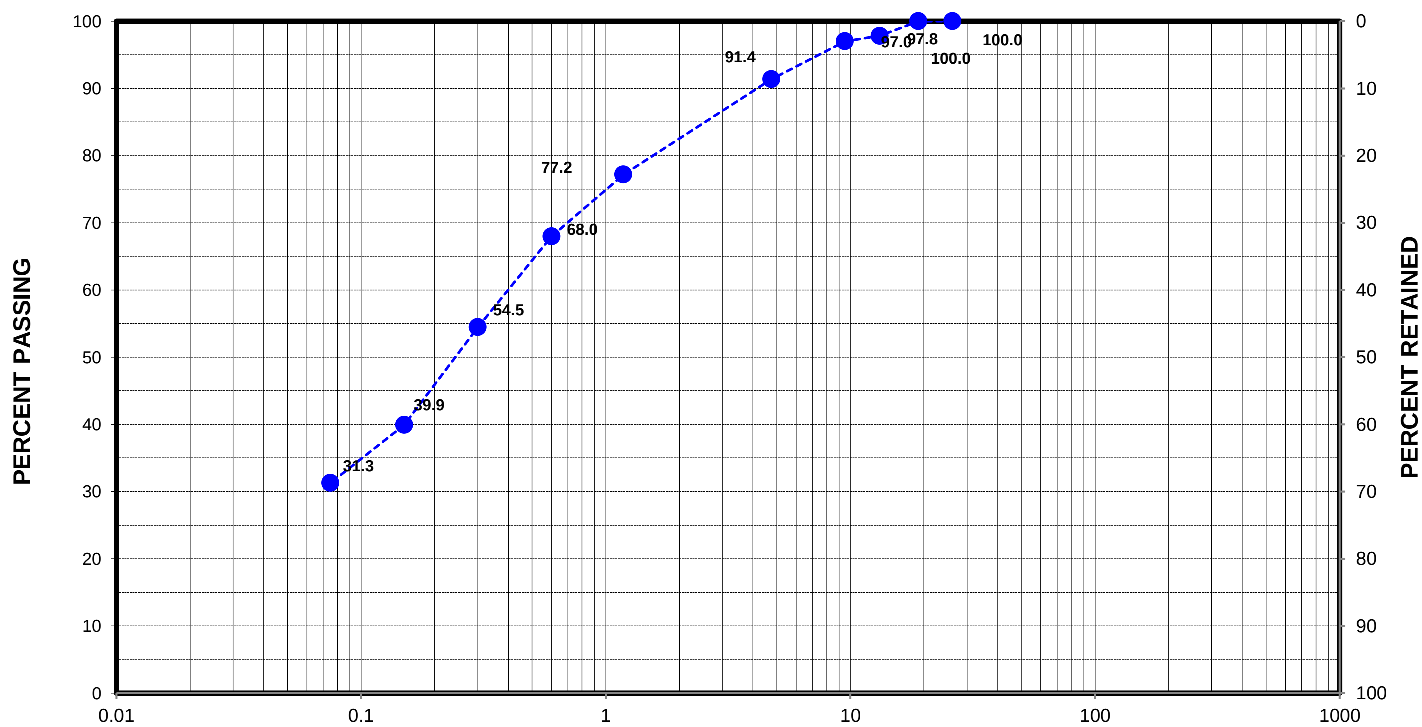
REMARKS:

PERFORMED BY:	A. Elhaddad	**DATE:**	October 23, 2019
VERIFIED BY:		**DATE:**	October 23, 2019

SIEVE ANALYSIS

CLIENT:	9770967 Canada Inc.	**LAB No.:**	G-19-009
PROJECT/SITE:	1098 Ogilvie Road/1178 Cummings Avenue	**PROJECT No.:**	11201061
BOREHOLE:	BH6	**SAMPLE:**	SS2
DEPTH:	0.8m - 1.4m	**SAMPLE DATE:**	September 23/24, 2019

SIEVE SIZE (mm)	SAMPLE % PASSING	
26.2	100.0	
19.0	100.0	
13.2	97.8	
9.5	97.0	
4.75	91.4	
1.18	77.2	
0.600	68.0	
0.300	54.5	
0.150	39.9	
0.075	31.3	



REMARKS:

PERFORMED BY:	A. Elhaddad	**DATE:**	October 23, 2019
VERIFIED BY:		**DATE:**	October 23, 2019

Appendix C
Rock Core Photo Logs

BH1-RC1/RC2/RC3 (Dry)

2.8 m

4.3 m

5.8 m

7.3 m

BH1-RC1/RC2/RC3 (Wet)

2.8 m

4.3 m

5.8 m

7.3 m

BH1-RC1, RC2, RC3 September 23, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH1 – RC1 – 2.8 to 4.3 mbgs	1.5	100	52% RQD
BH1 – RC2 – 4.3 to 5.8 mbgs	1.5	100	64% RQD
BH1 – RC3 – 5.8 to 7.3 mbgs	1.5	100	88% RQD

Core Log Photographs

BH1-RC4/RC5/RC6 (Dry)

7.3 m

8.8 m

10.2 m

11.7 m

BH1-RC4/RC5/RC6 (Wet)

7.3 m

8.8 m

10.2 m

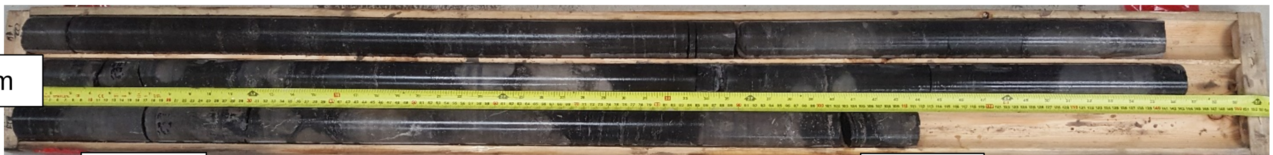
11.7 m

BH1-RC4, RC5, RC6 September 23, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH1 – RC4 – 7.3 to 8.8 mbgs	1.5	100	84% RQD
BH1 – RC5 – 8.8 to 10.2 mbgs	1.4	100	93% RQD
BH1 – RC6 – 10.2 to 11.7 mbgs	1.5	100	84% RQD

Core Log Photographs

BH1-RC7/RC8/RC9 (Dry)

BH1-RC7/RC8/RC9 (Wet)



<table>
<tr><td colspan="4">BH1-RC7, RC8, RC9
September 23, 2019</td></tr>
<tr><td rowspan="2">Core Run - Depth below ground surface (mbgs)</td><td colspan="2">Recovery</td><td rowspan="2">Remarks</td></tr>
<tr><td>m</td><td>%</td></tr>
<tr><td>BH1 – RC7 – 11.7 to 13.2 mbgs</td><td>1.5</td><td>100</td><td>92% RQD</td></tr>
<tr><td>BH1 – RC8 – 13.2 to 14.8 mbgs</td><td>1.6</td><td>100</td><td>93% RQD</td></tr>
<tr><td>BH1 – RC9 – 14.8 to 15.7 mbgs</td><td>0.9</td><td>100</td><td>91% RQD</td></tr>
</table>

Core Log Photographs

BH2-RC1/RC2/RC3 (Dry)

3.0 m

4.2 m

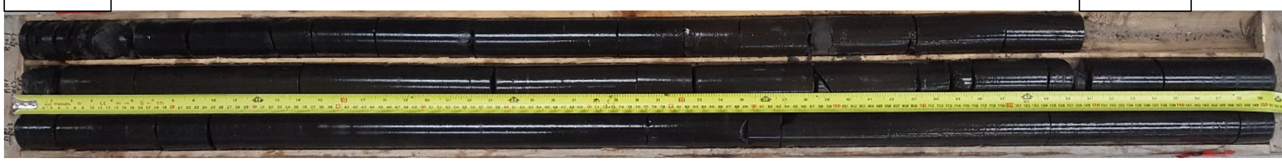
5.7 m

7.2 m

BH2-RC1/RC2/RC3 (Wet)

3.0 m

4.2 m



5.7 m

7.2 m

BH2-RC1, RC2, RC3 September 24, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH2 – RC1 – 3.0 to 4.2 mbgs	1.2	100	19% RQD
BH2 – RC2 – 4.2 to 5.7 mbgs	1.5	100	53% RQD
BH2 – RC3 – 5.7 to 7.2 mbgs	1.5	100	93% RQD

Core Log Photographs

BH2-RC4/RC5/RC6 (Dry)

7.2 m

8.7 m

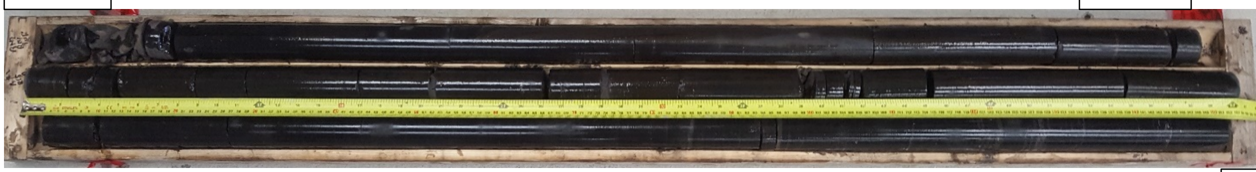
10.2 m

11.6 m

BH2-RC4/RC5/RC6 (Wet)

7.2 m

8.7 m



10.2 m

11.6 m

BH2-RC4, RC5, RC6 September 24, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH2 – RC4 – 7.2 to 8.7 mbgs	1.5	100	88% RQD
BH2 – RC5 – 8.7 to 10.2 mbgs	1.5	100	65% RQD
BH2 – RC6 – 10.2 to 11.7 mbgs	1.5	100	93% RQD; RC6 continued on next page

BH2-RC6/RC7/RC8/RC9 (Dry)

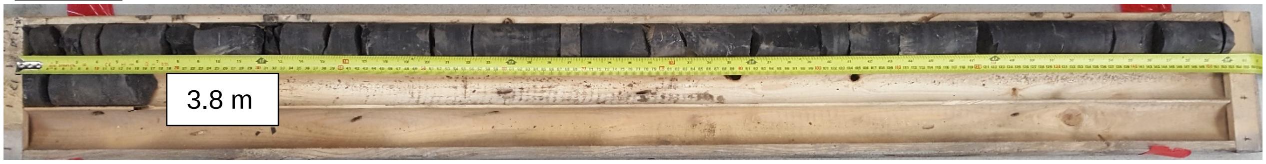
BH2-RC6/RC7/RC8/RC9 (Wet)

BH2-RC6, RC7, RC8, RC9 September 24, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH2 – RC6 – 10.2 to 11.7 mbgs	1.5	100	93% RQD
BH2 – RC7 – 11.7 to 13.2 mbgs	1.5	100	100% RQD
BH2 – RC8 – 13.2 to 14.7 mbgs	1.5	100	92% RQD
BH2 – RC8 – 14.7 to 15.6 mbgs	0.9	100	96% RQD

Core Log Photographs

BH3-RC1 (Dry)

2.3 m



BH3-RC1 (Wet)

2.3 m

BH3-RC1 September 23, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH3 – RC1 – 2.3 to 3.8 mbgs	1.6	100	30% RQD

Core Log Photographs

BH4-RC1/RC2 (Dry)

BH4-RC1/RC2 (Wet)

BH4-RC1, RC2 September 23, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH4 – RC1 – 1.4 to 2.7 mbgs	1.3	100	44% RQD
BH4 – RC2 – 2.7 to 3.8 mbgs	1.1	100	47% RQD

Core Log Photographs

BH5-RC1/RC2/RC3 (Dry)

1.4 m

2.7 m

4.3 m

5.6 m

BH5-RC1/RC2/RC3 (Wet)

1.4 m

2.7 m

4.3 m

5.6 m

BH5-RC1, RC2, RC3 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH5 – RC1 – 1.4 to 2.7 mbgs	1.3	100	30% RQD
BH5 – RC2 – 2.7 to 4.3 mbgs	1.6	100	88% RQD
BH5 – RC3 – 4.3 to 5.8 mbgs	1.5	100	90% RQD; RC3 continued on next page

Core Log Photographs

BH5-RC4/RC5/RC6 (Dry)

5.8 m

7.0 m

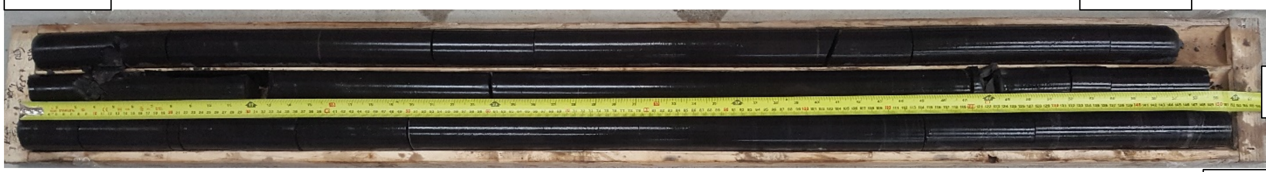
8.6 m

10.0 m

BH5-RC4/RC5/RC6 (Wet)

5.8 m

7.0 m



8.6 m

10.0 m

BH5-RC4, RC5, RC6 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH5 – RC4 – 5.8 to 7.2 mbgs	1.4	100	90% RQD
BH5 – RC5 – 7.2 to 8.7 mbgs	1.5	100	97% RQD
BH5 – RC6 – 8.7 to 10.2 mbgs	1.5	100	100% RQD; RC6 continued on next page

BH5-RC6/RC7/RC8/RC9 (Dry)

10.2 m

11.4 m

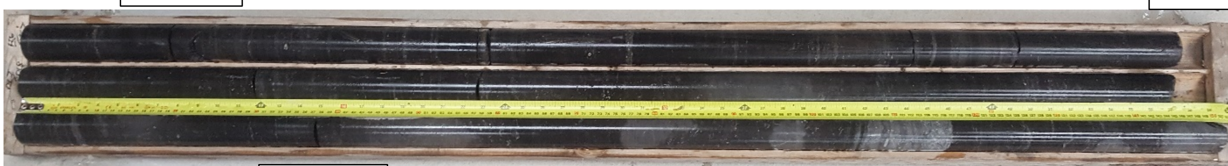
14.3 m

13.3 m

BH5-RC6/RC7/RC8/RC9 (Wet)

10.2 m

11.4 m



14.3 m

13.3 m

BH5-RC6, RC7, RC8, RC9 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH5 – RC6 – 8.7 to 10.2 mbgs	1.5	100	100% RQD
BH5 – RC7 – 10.2 to 11.7 mbgs	1.5	100	98% RQD
BH5 – RC8 – 11.7 to 13.3 mbgs	1.6	100	100% RQD
BH5 – RC9 – 13.3 to 14.6 mbgs	1.3	100	97% RQD; RC9 continued on next page

Core Log Photographs

BH5-RC9/RC10 (Dry)

14.6 m

15.8 m

BH5-RC9/RC10 (Wet)

14.6 m

15.8 m

BH5-RC6, RC7, RC8, RC9 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH5 – RC9 – 13.3 to 14.6 mbgs	1.3	100	97% RQD
BH5 – RC10 – 14.6 to 15.8 mbgs	1.2	100	96% RQD

Core Log Photographs

BH6-RC1/RC2/RC3 (Dry)

2.0 m

3.4 m

4.9 m

5.7 m

BH6-RC1/RC2/RC3 (Wet)

2.0 m

3.4 m

4.9 m

5.7 m

BH6-RC1, RC2, RC3 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH6 – RC1 – 2.0 to 2.6 mbgs	0.6	100	16% RQD
BH6 – RC2 – 2.6 to 4.2 mbgs	1.6	100	72% RQD
BH6 – RC3 – 4.2 to 5.7 mbgs	0.8	100	92% RQD

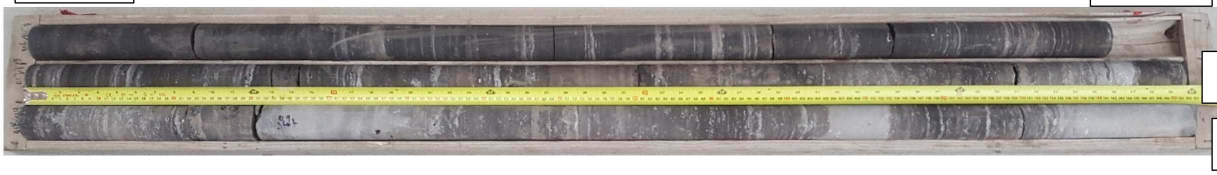
Core Log Photographs

BH6-RC4/RC5/RC6 (Dry)

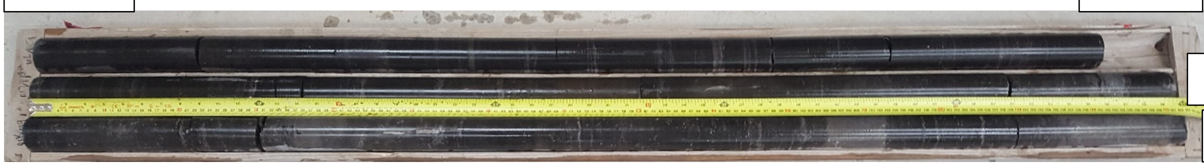
BH6-RC4/RC5/RC6 (Wet)

BH6-RC4, RC5, RC6 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH6 – RC4 – 5.7 to 7.3 mbgs	1.6	100	84% RQD
BH6 – RC5 – 7.3 to 8.7 mbgs	1.4	100	91% RQD
BH6 – RC6 – 8.7 to 10.2 mbgs	1.5	100	89% RQD: RC6 continued on next page

BH6-RC6/RC7/RC8/RC9 (Dry)



BH6-RC6/RC7/RC8/RC9 (Wet)



BH6-RC6, RC7, RC8, RC9 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH6 – RC6 – 8.7 to 10.2 mbgs	1.5	100	89% RQD
BH6 – RC7 – 10.2 to 11.7 mbgs	1.5	100	100% RQD
BH6 – RC8 – 11.7 to 13.2 mbgs	1.5	100	97% RQD
BH6 – RC9 – 13.2 to 14.8 mbgs	1.6	100	100% RQD; RC9 continued on next page

BH6-RC9/RC10 (Dry)

14.4 m

15.5 m

BH6- RC9/RC10 (Wet)

14.4 m

15.5 m

BH6- RC9, RC10 September 25, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH6 – RC9 – 13.2 to 14.8 mbgs	1.6	100	100% RQD
BH6 – RC10 – 14.8 to 15.5 mbgs	0.7	100	90% RQD

Appendix D
National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.425N 75.632W

2019-10-15 19:48 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.461	0.255	0.153	0.045
Sa (0.1)	0.538	0.308	0.191	0.062
Sa (0.2)	0.451	0.261	0.165	0.056
Sa (0.3)	0.342	0.199	0.127	0.044
Sa (0.5)	0.242	0.141	0.090	0.031
Sa (1.0)	0.120	0.070	0.045	0.015
Sa (2.0)	0.057	0.033	0.021	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.288	0.167	0.104	0.033
PGV (m/s)	0.201	0.113	0.069	0.021

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

References

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

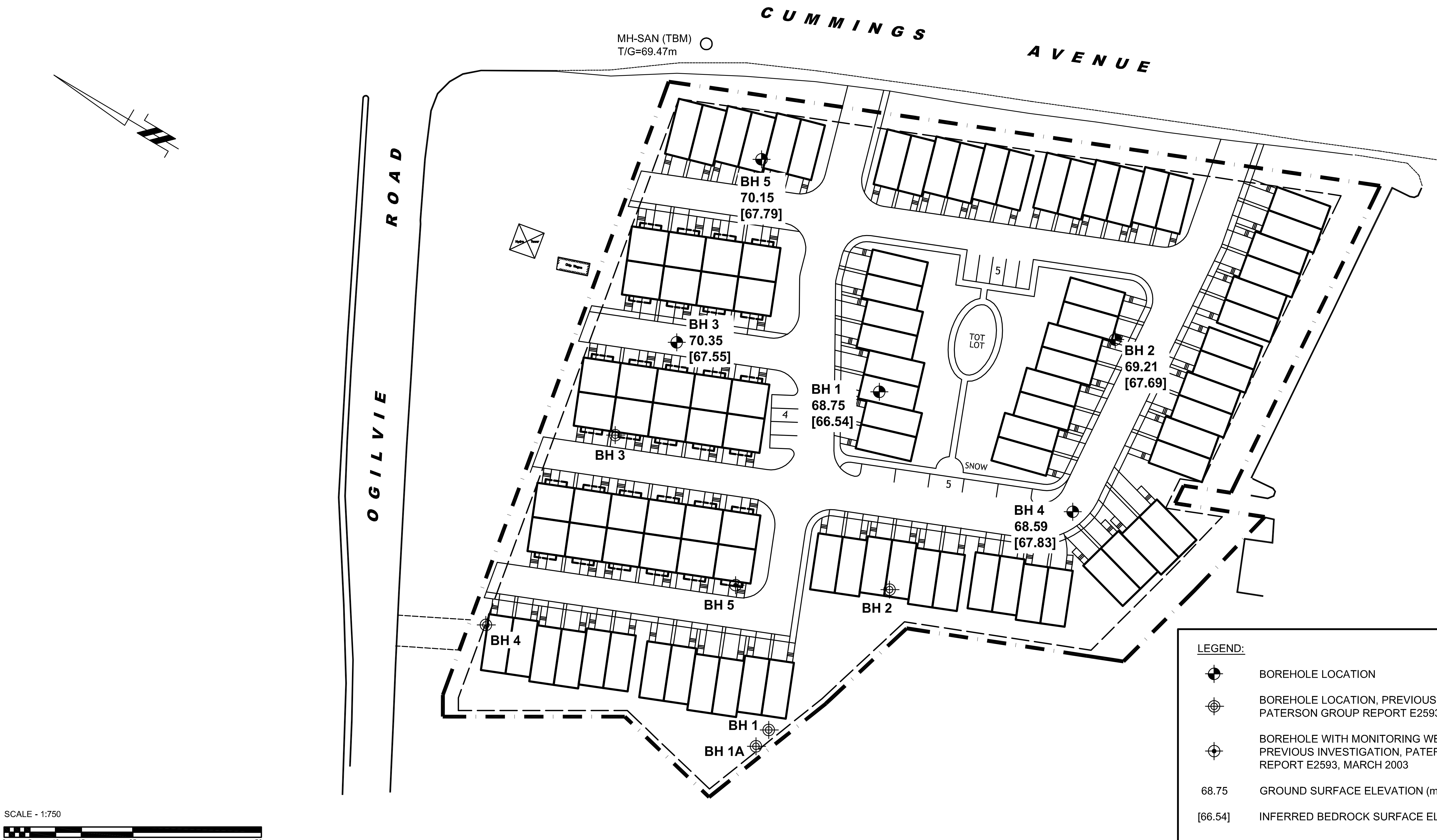
Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

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

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

Appendix E

Paterson Group Borehole Plan and Report



LEGEND:

	BOREHOLE LOCATION
	BOREHOLE LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT E2593, MARCH 2003
	BOREHOLE WITH MONITORING WELL INSTALLED, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT E2593, MARCH 2003
68.75	GROUND SURFACE ELEVATION (m)
[66.54]	INFERRED BEDROCK SURFACE ELEVATION (m)

BASE PLAN PROVIDED BY ROSALIE J. HILL ARCHITECT & DEV. CONSULTANT.

patersongroup
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

Scale:	1:750
Des.:	MK
Dwn:	MPG
Chkd:	DG

6770967 CANADA INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1098 OGILVIE ROAD AND 1178 CUMMINGS AVE.
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG2463-1
Report No.:	PG2463-1
Date:	09/2011

11x17

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Ogilvie Road
Ottawa, Ontario

DATUM TBM - Top of manhole located on centreline of Cummings Avenue, south of Ogilvie Road. Geodetic elevation = 69.47m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 August 2011

FILE NO. PG2463

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.75
FILL: Brown silty sand with gravel, cobbles and brick		AU	1				
		AU	2				
		SS	3	4	9	1	67.75
		SS	4	8	1	2	66.75
2.21							
BEDROCK: Black shale		SS	5		50+		
						3	65.75
						4	64.75
						5	63.75
						6	62.75
6.22							
End of Borehole							
(GWL @ 5.33m-Sept. 2/11)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Ogilvie Road
Ottawa, Ontario

DATUM TBM - Top of manhole located on centreline of Cummings Avenue, south of Ogilvie Road. Geodetic elevation = 69.47m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 August 2011

FILE NO. PG2463

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	69.21		
Compact, brown **SILTY SAND,** trace gravel and clay		AU	1						
		SS	2	25	23	1	68.21		
1.52									
BEDROCK: Black shale		SS	3		50+				
1.90									
End of Borehole									

Water Content % scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Ogilvie Road
Ottawa, Ontario

DATUM TBM - Top of manhole located on centreline of Cummings Avenue, south of Ogilvie Road. Geodetic elevation = 69.47m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 August 2011

FILE NO. PG2463

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	70.35		
TOPSOIL 0.10									
FILL: Brown silty sand with gravel		SS	1		19				
		SS	2		18	1	69.35		
2.21		SS	3		6	2	68.35		
Grey **CLAYEY SILT,** trace gravel 2.80		SS	4	52	6				
BEDROCK: Black shale 3.35						3	67.35		
End of Borehole									
(BH dry - Sept. 2/11)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Ogilvie Road
Ottawa, Ontario

DATUM TBM - Top of manhole located on centreline of Cummings Avenue, south of Ogilvie Road. Geodetic elevation = 69.47m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 August 2011

FILE NO. PG2463

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	68.59		
TOPSOIL 0.30		AU	1						
Brown **SILTY SAND** with gravel 0.76		AU	2						
BEDROCK: Black shale 1.52		SS	3	100	50+	1	67.59		
End of Borehole									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Ogilvie Road
Ottawa, Ontario

DATUM TBM - Top of manhole located on centreline of Cummings Avenue, south of Ogilvie Road. Geodetic elevation = 69.47m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 August 2011

FILE NO. PG2463

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.15
FILL: Brown silty sand with gravel, some clay		AU	1				
		AU	2				
0.69							
FILL: Grey sandy silt with wood chips		SS	3	67	17	1	69.15
1.45							
FILL: Grey silty clay wtih sand and gravel							
1.60							
TOPSOIL		SS	4	75	6		
1.80						2	68.15
Grey **SILTY CLAY** with shale							
2.36		SS	5	100	50+		
BEDROCK: Black shale						3	67.15
						4	66.15
						5	65.15
6.10						6	64.15
End of Borehole							
(GWL @ 5.82m-Sept. 1/11)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Engineers
28 Concourse Gate, Unit 1, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
1098 Ogilvie Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 75 Power Auger

DATE 27 FEB 03

FILE NO. **E2593**

HOLE NO. **BH 1A**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Lower Explosive Limit % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE							0			
Asphaltic concrete	0.04									
FILL: Black clayey silt, some sand, gravel and organic matter			AU	1					△	
	0.60		AU	2						
End of Borehole Practical auger refusal to augering @ 0.60m depth										

100 200 300 400 500
Gastech 1314 Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
1098 Ogilvie Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 75 Power Auger

DATE 27 FEB 03

FILE NO. **E2593**

HOLE NO. **BH 1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Lower Explosive Limit % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0			
FILL: Black clayey silt with organic matter		AU	3					△	
		SS	4	50	29	1		△	
1.22 BEDROCK: Weathered shale 1.42									
End of Borehole									
Practical refusal to augering @ 1.42m depth									

Gastech 1314 Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
1098 Ogilvie Road
Ottawa, Ontario

FILE NO.	E2593
HOLE NO.	BH 2

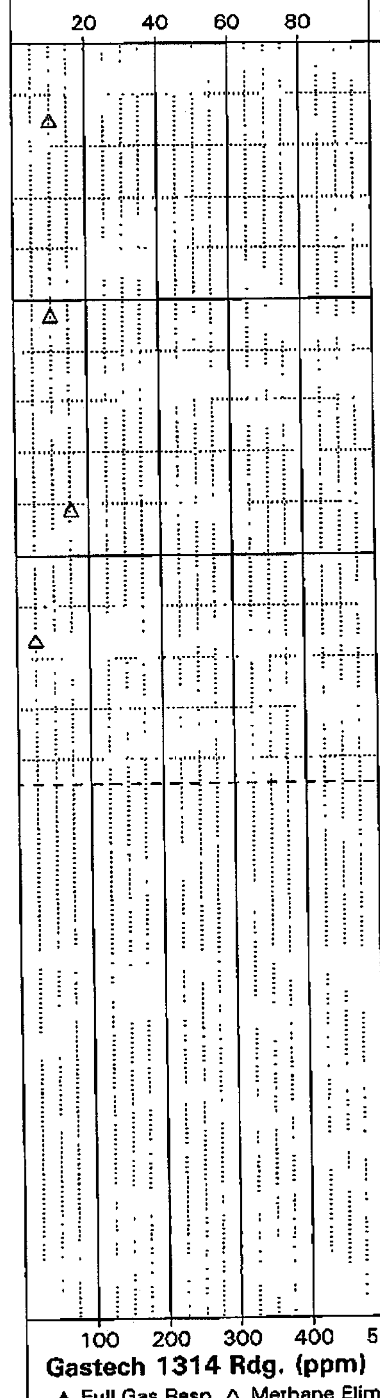
DATUM

REMARKS

BORINGS BY CME 75 Power Auger DATE 28 FEB 03

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Black clayey silt with shale layer and rock fragments		SS	5	50	29		
		SS	6	50	23	1	
1.68							
BEDROCK: Weathered shale		SS	7	25	50+	2	
		SS	8	17	50+		
2.90							
End of Borehole							

Pen. Resist. Blows/0.3m	Monitoring Well Construction
● 50 mm Dia. Cone	
○ Lower Explosive Limit %	



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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
1098 Ogilvie Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 75 Power Auger

DATE 28 FEB 03

FILE NO. E2593

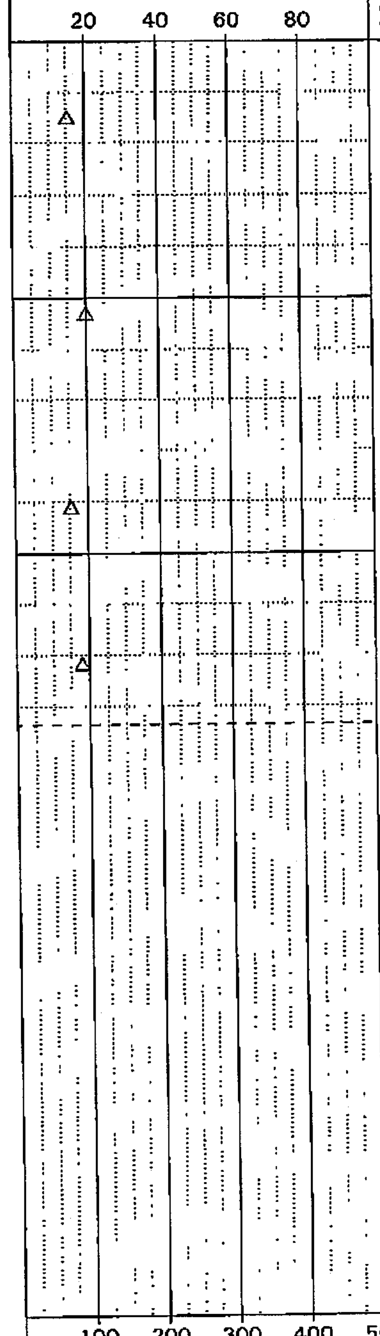
HOLE NO. **BH 3**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
FILL: Black clayey silt with organic matter	SS	9	62	32		
	SS	10	33	28	1	
	SS	11	25	20		
1.98					2	
BEDROCK: Weathered shale with silty clay layers	SS	12	43	50+		
2.67						
End of Borehole Practical refusal to augering @ 2.67m depth						

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Lower Explosive Limit %

Monitoring Well Construction



Gastech 1314 Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
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Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 75 Power Auger

DATE 28 FEB 03

FILE NO. **E2593**

HOLE NO. **BH 4**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0			
FILL: Black to brown clayey silt with organic matter, some sand		SS	13	50	29				
		SS	14	62	22	1			
		SS	15	54	19				
2.26						2			
End of Borehole Practical refusal to augering @ 2.26m depth									

Lower Explosive Limit % scale: 20 40 60 80

Gastech 1314 Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE & TEST DATA

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DATUM

REMARKS

BORINGS BY CME 75 Power Auger

DATE 28 FEB 03

FILE NO. **E2593**

HOLE NO. **BH 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Black to brown clayey silt with some sand		SS	16	54	29		
		SS	17	62	29	1	
1.52							
BEDROCK: Weathered shale		SS	18	60	50+		
						2	
						3	
						4	
4.57 End of Borehole							
(GWL @ 3.36m-March 6/03)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Lower Explosive Limit %

20 40 60 80

Monitoring Well Construction

100 200 300 400 500

Gastech 1314 Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

about GHD

GHD is one of the world's leading professional services companies operating in the global markets of water, energy and resources, environment, property and buildings, and transportation. We provide engineering, environmental, and construction services to private and public sector clients.

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