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Geotechnical and Preliminary Hydrogeological Investigation

Proposed Hi-Rise Complex
400 Albert Street and 383 Slater Street
Ottawa, Ontario

Prepared For

Main and Main Developments Inc.

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Report: PG4793-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Main and Main to conduct a geotechnical investigation for a proposed high rise complex located at 400 Albert Street and 383 Slater Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work for this geotechnical investigation. An environmental program was carried out and the information is presented under separate cover.

2.0 Proposed Development

Based on current plans, it is understood that the proposed development will consist of 3 high rise mixed-use buildings. Towers A, B and C will have 14, 32 and 34 storeys, respectively, and will share an underground parking garage with 4 to 6 levels below the existing grade. The underground parking will occupy the majority of the site. It is further understood that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on March 28, 29 and April 1, 2019. At that time, 3 boreholes were advanced to a maximum depth of 19.4 m below existing grade across the subject site to provide general coverage of the proposed development. A previous geotechnical investigation was completed in June 2015. The locations of all the test holes are shown on Drawing PG4793-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two person crew. The test hole procedure consisted of augering to the required depths at the selected locations and sampling the overburden soils. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, or drill cuttings from the auger flights. The split-spoon and auger samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was carried out at 3 borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the bedrock quality.

Subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

51 mm diameter PVC groundwater monitoring wells were installed in each of the current boreholes to permit monitoring of the groundwater levels subsequent to the completion of the drilling program. Monitoring wells will be decommissioned prior to the commencement of construction and during the excavation program. Typical monitoring well construction details are described below:

☐ Slotted 51 mm diameter PVC screen at base of borehole for 3 m length.
☐ 51 mm diameter PVC riser pipe from top of the screen to ground surface.
☐ No. 3 silica sand backfill within annular space around screen.
☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
☐ Clean backfill from top of bentonite plug to ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific monitoring well construction details.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevations at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of a manhole cover located at the west side of the subject site. A geodetic elevation of 72.77 m was provided for the TBM. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG4793-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site currently consists of a gravel surfaced at-grade parking lot. Two 3-storey apartment buildings are located in the northwest corner of the site. A 2-storey building is located near the northeast corner of the site. The ground surface across the subject site generally slopes down from north to south. The ground surface was observed to be at grade with the adjacent roadways.

The site is bordered by Albert Street to the north, Lyon Street to the east, Slater Street to the south and Bay Street to the west.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the borehole locations consists of crushed stone gravel surfacing, followed by a layer of silty sand fill. Glacial till was encountered below the above noted soils at BH 1 and BH 2, which was observed to generally consist of clayey silt with sand and gravel. The fill layer was underlain by a concrete slab at BH 3. Practical refusal to augering was encountered at depths ranging between 2.8 and 4.3 m below existing grade.

It should be noted that several test pits were excavated within the proposed building footprint of Tower C to assess the fill material used to backfill the building demolition. A fill layer consisting of crushed stone was encountered at each of the test pit locations, followed by practical refusal to excavation on concrete at 200 to 400 mm depth.

Bedrock

Grey limestone bedrock was cored at each of the borehole locations. Based on our observations, the majority of the bedrock core was noted to be of good to excellent quality.

Based on available geological mapping, the local bedrock consists of interbedded grey and black limestone of the Ottawa and Eastview formations with an overburden drift thickness of 2 to 5 m.

Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

A total of 3 groundwater monitoring wells were installed as part of our geotechnical investigation. Groundwater level measurements were recorded at the monitoring well locations and our observations are presented in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	73.24	4.37	68.87	April 9, 2019
BH 2	71.99	2.20	69.79	April 9, 2019
BH 3	72.77	3.50	69.27	April 9, 2019

The groundwater levels measured in the monitoring well locations, it may be possible that these elevated groundwater levels represent a perched groundwater condition at the soil/bedrock interface. The LRT Confederation Line is approximately one block from the subject site and is a partially tanked tunnel which is drawing the groundwater to a depth of approximately 20 m. It's expected that the drawdown of the groundwater from surrounding buildings have also lowered the general groundwater table in this area. Based on the above, the long-term groundwater table can be expected to be at a depth of 12 to 15 m below the existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is anticipated that the proposed development will be founded on spread footings placed on a clean, surface sounded bedrock bearing surface.

Bedrock removal will be required to complete the underground parking levels. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. Grinding of the bedrock will be carried out next to sensitive structures and to lessen the potential for over breaking of the bedrock vertical surface for architectural drainage details.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed development. It should be noted that concrete was encountered within the overburden fill material during the field program, and therefore building remnants such as concrete slabs and/or foundation walls should be anticipated.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking levels. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. When possible, a minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system. Consideration can also be given to providing a vertical excavation provided the temporary shoring can accommodate a vertical face with satisfactory bedrock anchoring.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. Alternatively, an engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of the material's SPMDD could be placed around the proposed footings.

Bedrock Stabilization

Rock anchors or rock bolts may be required at specific locations to stabilize the vertical bedrock excavated face to prevent pop-outs of the bedrock and slippage of the rock mass due to the dip in the bedrock planes. Furthermore, the upper levels of the bedrock is typically fractured and weathered and often requires some form of protections using geotextile with a chain link to deal with minor raveling.

The requirements for rock stabilization will be evaluated by the geotechnical engineer during the excavation program when the bedrock condition can be better evaluated.

5.3 Foundation Design

Bearing Resistance Values

A factored bearing resistance value at ultimate limit states (ULS) of **6,000 kPa** can be used for footings founded on limestone bedrock at the proposed founding elevation of the parking garage provided the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant. As an alternative to probing the bedrock, the bedrock vertical face, along the excavation sides and within depressed areas such as the elevator pit, can be assessed by the geotechnical engineer to confirm the soundness of the bedrock at depth.

Footings placed on the upper levels of the limestone bedrock surface for shallow building structures (canopy and garage ramp) should be cleaned and surface sounded. This bearing medium can be designed using a bearing resistance value at ULS of **3,000 kPa**.

A geotechnical resistance factor of 0.5 was incorporated into the above noted bearing resistance values reported at ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The shear wave velocity testing was completed by Paterson personnel. Two seismic shear wave velocity profiles from the testing are presented on Appendix 2.

Field Program

The seismic array location is presented on Drawing PG4793-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 18 horizontal geophones in a straight line in an approximately east-west orientation. The 4.5 Hz geophones were mounted to the surface with two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 5 to 10 times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e. striking both sides of the I-beam seated parallel to the geophone array). The shot locations are located 3 and 4.5 m from the first geophone and 3, 4.5 and 15 m from the last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity test results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the building foundations. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Given the bedrock depth encountered in the test holes at this site, it is anticipated that the proposed building will be founded directly on the bedrock. Based on our testing results, the bedrock shear wave velocity is **2,339 m/s**.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}339m/s}\right)}$$

$$V_{s30} = 2{,}339m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for the proposed building is 2,339 m/s provided the footings are placed directly on the bedrock surface. Therefore, a **Site Class A** is applicable for the proposed buildings, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. If storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone for finished space areas and 300 mm of OPSS Granular A for the parking garage areas. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.05
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Conditions

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 60 and 120 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 75** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **1.20 and 0.022**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	75 m=1.20 and s=0.022
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the total anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.2	0.55	1.75	250
	2	0.8	2.8	500
	3.2	1.4	4.6	1000
	5.3	2.2	7.5	2000
125	1	0.5	1.5	250
	1.7	0.7	2.4	500
	2.6	1.1	3.7	1000
	4.1	1.8	5.9	2000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Structure

For design purposes, the pavement structure presented in Table 4 could be used for the design of access lanes, if required. The bottom level of the underground parking garage could have the rigid pavement structure recommended in Table 5.

Table 4 - Recommended Flexible Pavement Structure
Access Lanes and Heavy Truck Loading Areas

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 or HL-3 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 or HL-8 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Glacial Till or OPSS Granular B Type I or II material placed over bedrock.	

Table 5 - Recommended Rigid Pavement Structure
Lower Level of Parking Garage

Thickness (mm)	Material Description
150	**Wear Course -** Concrete slab
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Bedrock or OPSS Granular B Type II material placed over bedrock.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the underground parking garage footprint will occupy the entirety of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a drainage system placed on the bedrock face.

Since the lower levels of underground parking will be located below the expected long term groundwater level, consideration may be given to installing a groundwater infiltration suppression system to manage the groundwater infiltration volumes. By waterproofing the vertical excavation side slopes at these lower levels, it will be possible to lessen the groundwater volumes entering the excavation. This can be accomplished by placing a waterproofing membrane layer (bentonite sheets) against a prepared vertical bedrock surface. The membrane should start from the bottom of the excavation when pouring the perimeter strip footings. A composite drainage system should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water that may breach the waterproofing membrane system.

For preliminary design purposes, the composite drainage system (such as Delta Drain 6000 or equivalent) should extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m spacing on centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

The requirement for the waterproofing membrane will be evaluated by the geotechnical engineer once the excavation is close to reaching the bottom.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

It’s expected that the upper 1.5 m will consist of a double sided pour once the shoring system is removed. This portion of the foundation wall should be backfill with free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material or OPSS Granular A, should otherwise be used for this purpose.

Adverse Effects of Dewatering on Adjacent Properties

The buildings located within adjacent properties are anticipated to be founded within the glacial till or directly on the bedrock. Therefore, no adverse effects to the surrounding buildings or properties are expected with the minor dewatering of the groundwater from this development.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes and Temporary Shoring

It is expected that insufficient room will be available to permit the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Rock Stabilization

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Temporary shoring will be required to support the overburden for the entire perimeter of the excavation.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

For design purposes, the temporary system will most likely consist of a soldier pile and timber lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes.

Typical Geotechnical Parameters

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. Because the depth at which the apex shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less capacity, than one where the bonded length was only the bottom part of the overall anchor.

The design of the rock anchors for temporary shoring can be based on the values provided in Subsection 5.7 of the present report.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 6 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Soldier Pile and Timber Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible. The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

The rate of flow of groundwater into the excavation through the overburden and bedrock is anticipated to be moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

A temporary MECP permit to take water (PTTW) Category 3 will be required for this project since water infiltration is expected to be greater than 50,000 L/day during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. Provided that the selected groundwater infiltration control system is properly implemented and approved by Paterson at the time of construction, it is expected that groundwater flow will be low (i.e. less than 40,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps. For sump pit and sump pump design, consideration should be given to using higher volumes of 200,000 to 300,000 L/day.

Impacts on Neighbouring Structures

It is understood that 4 to 6 underground parking levels are planned for the proposed development, with the lower portion of the foundation having a groundwater infiltration control system in place. Due to the surrounding developments that are founded at greater depths, long-term groundwater lowering is expected to be minimal to negligible for the area once steady state is achieved.

Based on our observations, the groundwater level is anticipated at a 3 to 4 m depth which is considered to be perched over the bedrock surface. Therefore, a local groundwater lowering is expected under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal groundwater lowering.

The neighbouring structures are founded within native glacial till or directly over a bedrock bearing surface based on available soils information. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precautions must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed foundation drainage design and waterproofing requirements.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials used.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Main and Main Developments Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Nathan F. S. Christie, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Aug. 23-2019
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

Report Distribution

- ☐ Main and Main Developments Inc. (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
383 Slater St., 400 Albert St. & 156-160 Lyon Street
Ottawa, Ontario

DATUM TBM - Top of manhole cover located along east side of Bay Street, north of Slater Street. Geodetic elevation = 72.77m

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 28, 2019

FILE NO. PG4793

HOLE NO. BH 1

SOIL DESCRIPTION

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
25mm Asphaltic concrete over **FILL:** Brown clayey sand with sand and gravel	0.60
Compact, brown **SILTY SAND**	0.86
Very stiff, brown **SILTY CLAY,** some gravel	2.44
GLACIAL TILL: Brown clayey silt with sand and gravel	4.29
BEDROCK: Fair to excellent quality, grey limestone	18.01

End of Borehole

(GWL @ 4.37m depth - Apr 9/19)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	79	12
SS	3	4	62
SS	4	88	10
SS	5	62	10
SS	6	47	50+
RC	1	100	0
RC	2	98	96
RC	3	100	84
RC	4	100	89
RC	5	100	100
RC	6	100	97
RC	7	100	100
RC	8	100	97
RC	9	100	100
RC	10	100	100

DEPTH (m)	ELEV. (m)
0	73.24
1	72.24
2	71.24
3	70.24
4	69.24
5	68.24
6	67.24
7	66.24
8	65.24
9	64.24
10	63.24
11	62.24
12	61.24
13	60.24
14	59.24
15	58.24
16	57.24
17	56.24
18	55.24

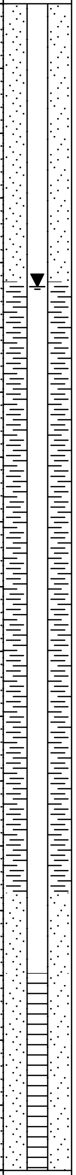
Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

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

Monitoring Well Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
383 Slater St., 400 Albert St. & 156-160 Lyon Street
Ottawa, Ontario

DATUM TBM - Top of manhole cover located along east side of Bay Street, north of Slater Street. Geodetic elevation = 72.77m

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 29, 2019

FILE NO. PG4793

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06	AU	1		
FILL: Brown silty sand with gravel	2.08	SS	2	50	52
		SS	3	46	17
Compact, brown **SILTY SAND**	2.39				
GLACIAL TILL: Brown clayey silt with sand and gravel	2.84	SS	4	33	2
BEDROCK: Good to excellent quality, grey limestone	17.88	RC	1	100	87
		RC	2	100	93
		RC	3	100	95
		RC	4	100	95
		RC	5	100	100
		RC	6	100	100
		RC	7	100	100
		RC	8	100	100
		RC	9	100	97
		RC	10	100	100

End of Borehole

(GWL @ 2.20m depth - Apr 9/19)

DEPTH (m)	ELEV. (m)
0	71.99
1	70.99
2	69.99
3	68.99
4	67.99
5	66.99
6	65.99
7	64.99
8	63.99
9	62.99
10	61.99
11	60.99
12	59.99
13	58.99
14	57.99
15	56.99
16	55.99
17	54.99

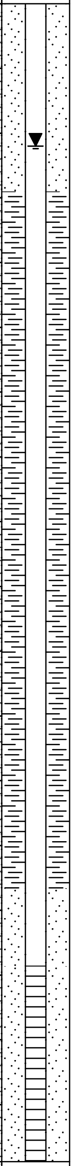
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

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

Monitoring Well Construction



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
383 Slater St., 400 Albert St. & 156-160 Lyon Street
Ottawa, Ontario

DATUM TBM - Top of manhole cover located along east side of Bay Street, north of Slater Street. Geodetic elevation = 72.77m

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 1, 2019

FILE NO. PG4793

HOLE NO. BH 3

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Grey silty sand with gravel, some clay		AU	1		
		SS	2	38	66
		SS	3	46	21
		SS	4	8	11
	3.28	SS	5	44	50+
Concrete with rebar and ties	3.73				
BEDROCK: Good to excellent quality, grey limestone		RC	1	17	0
		RC	2	97	83
		RC	3	100	61
		RC	4	100	82
		RC	5	100	93
		RC	6	100	100
		RC	7	100	100
		RC	8	100	100
		RC	9	100	100
		RC	10	100	95
		RC	11	100	100
	19.38	RC	12	100	10
End of Borehole					
(GWL @ 3.50m depth - Apr 9/19)					

DEPTH (m)	ELEV. (m)
0	72.28
1	71.28
2	70.28
3	69.28
4	68.28
5	67.28
6	66.28
7	65.28
8	64.28
9	63.28
10	62.28
11	61.28
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13	59.28
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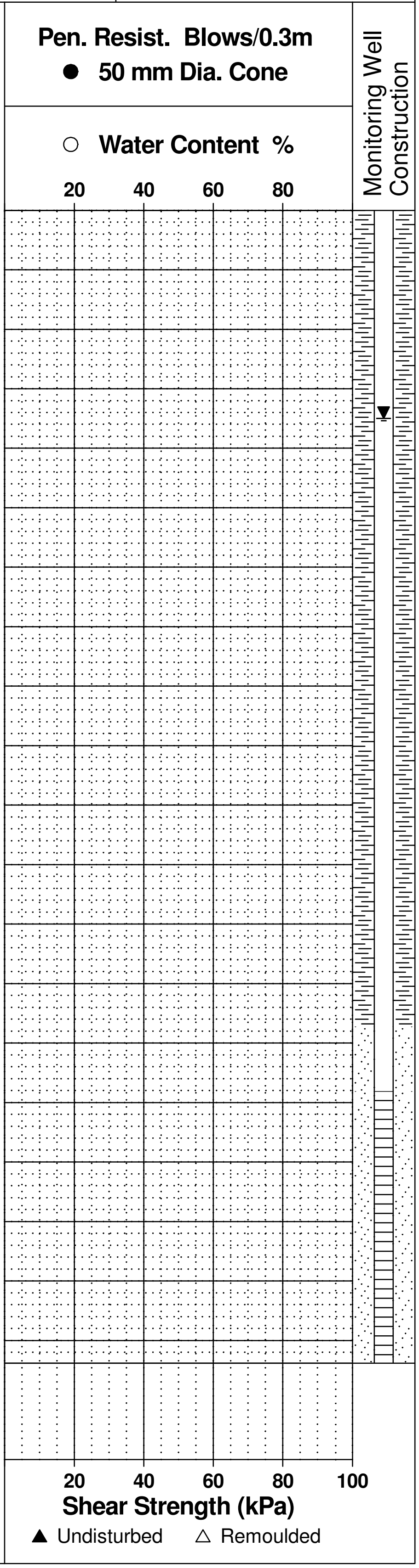
15	57.28
16	56.28
17	55.28
18	54.28
19	53.28



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Redevelopment
383 Slater St. & 400 Albert St., Ottawa, Ontario

DATUM TBM - Top cover of manhole located along east side of Bay Street, just north of Slate Street. A geodetic elevation of 72.77m was provided to the TBM by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

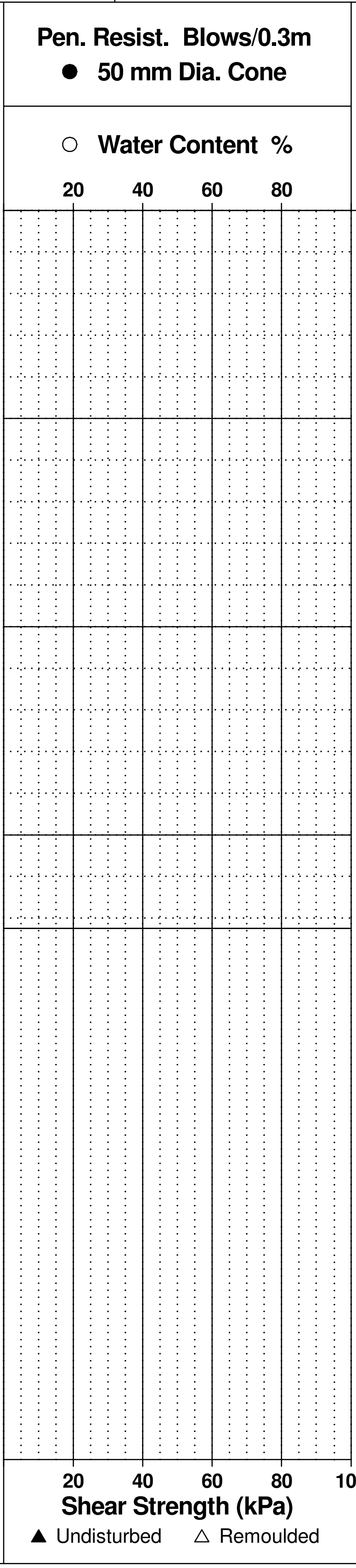
BORINGS BY CME 55 Power Auger

DATE December 9, 2016

FILE NO. PG3914

HOLE NO. BH 4-16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.36
FILL: Brown silty fine sand with crushed stone, some blast rock — 0.81	▨	AU	1				
FILL: Brown sandy silt, some asphalt and concrete, trace brick — 1.52	▨	SS	2	21	4	1	71.36
FILL: Brown sandy silt, some gravel, cobbles and topsoil — 1.98	▨	SS	3	17	4		
FILL: Crushed concrete — 2.59	▨	SS	4	33	50+	2	70.36
Concrete slab — 2.74							
FILL: Crushed stone, some sand — 2.90	▨						
Inferred **GLACIAL TILL:** Grey silty sand, some gravel, cobbles and boulders — 3.45		SS	5	0	50+	3	69.36
End of Borehole Practical refusal to augering at 3.45m depth (BH dry upon completion)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Redevelopment
383 Slater St. & 400 Albert St., Ottawa, Ontario

DATUM TBM - Top cover of manhole located along east side of Bay Street, just north of Slate Street. A geodetic elevation of 72.77m was provided to the TBM by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 9, 2016

FILE NO. PG3914

HOLE NO. BH 5-16

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 / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	71.91		
FILL: Crushed stone 0.30		AU	1						
FILL: Brown silty sand, some concrete, brick and mortar, trace crushed stone 1.42		SS	2	46	6	1	70.91		
FILL: Brown silty fine sand, trace boulders 2.64		SS	3	83	7	2	69.91		
GLACIAL TILL: Grey silty sand, some gravel, cobbles and boulders 3.00		SS	4	75	27				▽
End of Borehole Practical refusal to augering at 3.00m depth (GWL @ 2.7m depth based on field observations)						3	68.91		

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Redevelopment
383 Slater St. & 400 Albert St., Ottawa, Ontario

DATUM TBM - Top cover of manhole located along east side of Bay Street, just north of Slate Street. A geodetic elevation of 72.77m was provided to the TBM by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 9, 2016

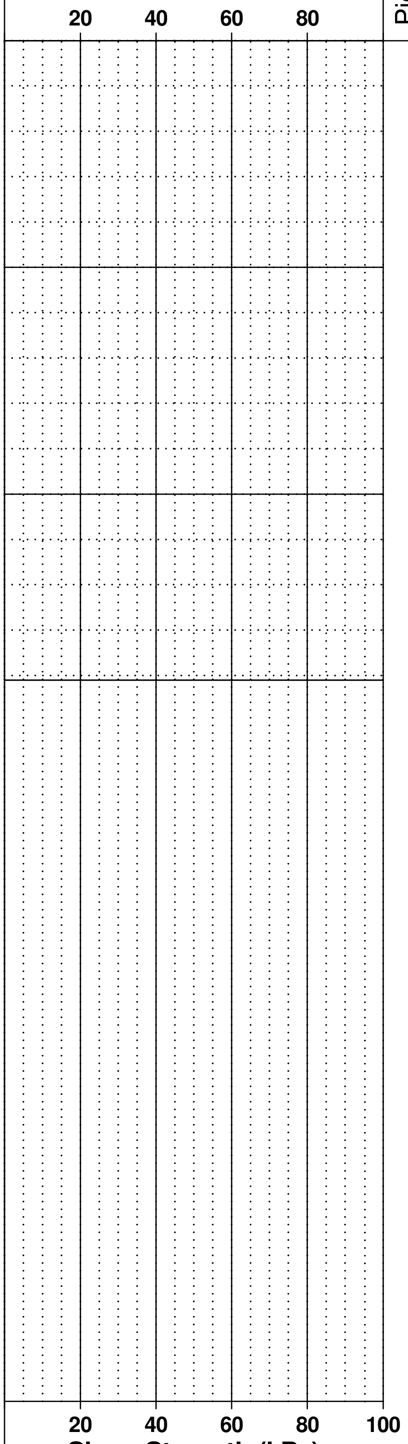
FILE NO. PG3914

HOLE NO. BH 6-16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE					0	72.19	
FILL: Crushed stone 0.30	AU	1					
FILL: Brown silty sand with crushed stone, some concrete and mortar 0.60							
	SS	2	21	5	1	71.19	
FILL: Brown fine to coarse sand, some gravel - trace coal by 1.5m depth	SS	3	46	2			
					2	70.19	∇
	SS	4	33	2			
- trace mortar by 2.7m depth 2.82							
End of Borehole Practical refusal to augering at 2.82m depth (GWL @ 2.3m depth based on field observations)							

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Redevelopment
383 Slater St. & 400 Albert St., Ottawa, Ontario

DATUM TBM - Top cover of manhole located along east side of Bay Street, just north of Slate Street. A geodetic elevation of 72.77m was provided to the TBM by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 9, 2016

FILE NO. PG3914

HOLE NO. BH 7-16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.12
FILL: Crushed stone 0.30		AU	1				
FILL: Construction debris (concrete, mortar, tile) 1.32		SS	2	36	27	1	71.12
Concrete slab 1.47							
FILL: Brown silty fine to medium sand with crushed stone 1.98		SS	3	71	11		
Brown **CLAYEY SILT** with sand 2.21						2	70.12
GLACIAL TILL: Grey sandy silt, some gravel, cobbles and boulders 2.87		SS	4	100	50+		

End of Borehole

Practical refusal to augering at 2.87m depth

(BH dry upon completion)

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Redevelopment
383 Slater St. & 400 Albert St., Ottawa, Ontario

DATUM TBM - Top cover of manhole located along east side of Bay Street, just north of Slate Street. A geodetic elevation of 72.77m was provided to the TBM by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

FILE NO. PG3914

HOLE NO. BH 8-16

BORINGS BY CME 55 Power Auger **DATE** December 9, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	72.08		
FILL: Crushed stone, trace brick									
0.25		AU	1						
FILL: Brown sandy silt with topsoil, some brick, mortar, trace wood and gravel		SS	2	79	11	1	71.08		
1.62		SS	3	88	13				
						2	70.08		
FILL: Brown clayey silt with sand, trace gravel and organics		SS	4	79	8				
2.97									
Brown **CLAYEY SILT,** trace sand						3	69.08		
3.35		SS	5	50	10				∇
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	6	71	15	4	68.08		
		SS	7	85	50+				
4.90									
End of Borehole Practical refusal to augering at 4.90m depth (GWL @ 4.8m depth based on field observations)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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R.F.

LVM

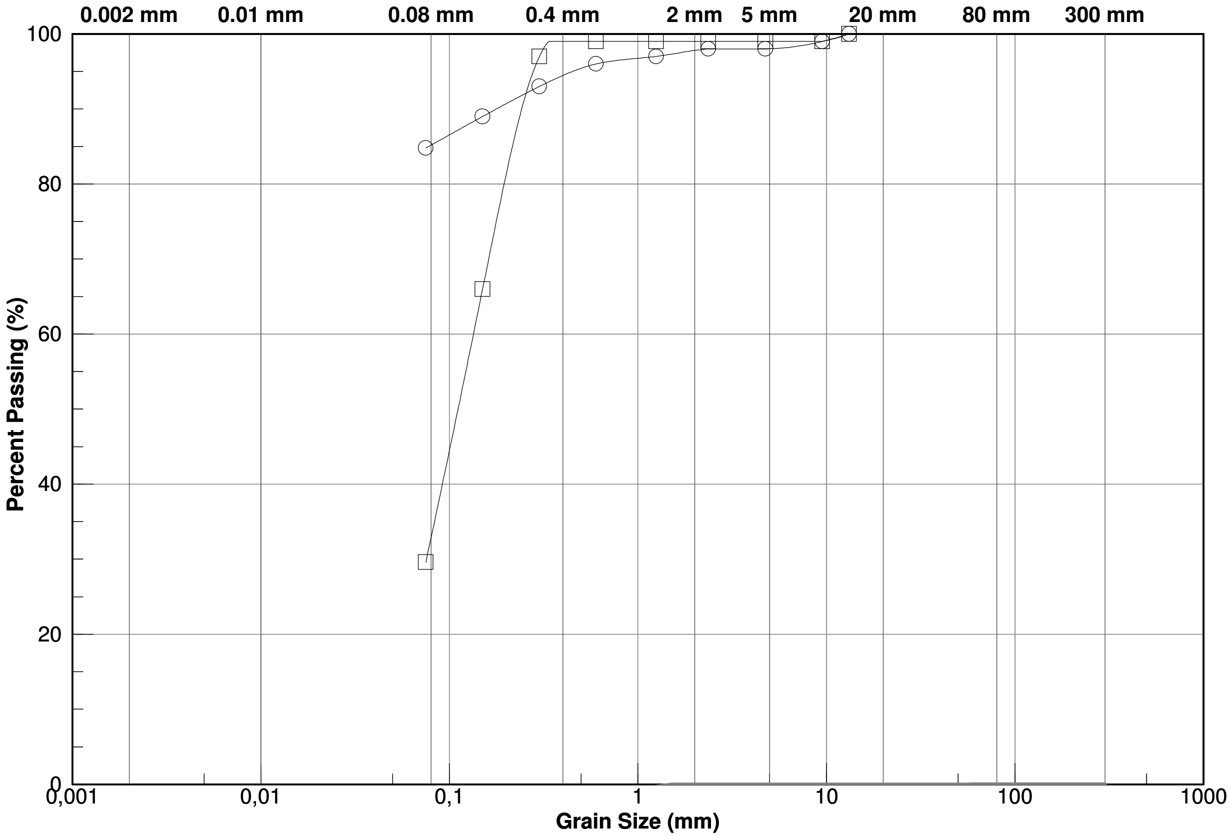
GRAIN-SIZE ANALYSIS

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Figure n° : **1**

Location:**400 Albert street, Ottawa, ON**

File n° : **B-0001436-1**



CLAY	SILT	SAND			GRAVEL		COBBLE	BOULDER
		FINE	MEDIUM	COARSE	FINE	COARSE		

Symbol	Borehole n°	Sample n°	Depth (m)	Description	USCS class. (ASTM D-2487)
○	BH-01-12	SS-4	2.29 - 2.90	Silty sand, traces of gravel.	SM
□	BH-03-12	SS-2	0.76 - 1.37	Silt with some sand, traces of gravel.	ML

EQ-09-Ge-68A R.1 04.03.2009

LVM

EXPLANATION NOTE ON SOUNDING LOGS

The following sounding logs summarize soils and rock geotechnical properties as well as ground water conditions, as collected during field work and/or obtained from laboratory tests. This note explains the different symbols and abbreviations used in these logs.

STRATIGRAPHIC UNITS

Elevation/Depth: Reference to the geodesic elevation of the soil or to a bench mark of arbitrary elevation, at the location of the sounding. Depth of the different geological boundaries as measured from ground surface. On the left, the scale is in meters while on the right, it is in feet.

Description of the stratigraphic units: Every geological formation is detailed.

The proportion of the different elements of the soil, defined according to the size of the particles, is given following the classification hereafter. The relative compactness of cohesionless soils is defined by the "N" index of the Standard Penetration Test. The consistency of cohesive soils is defined by their shear resistance.

Classification	**Particle size (mm)**
Clay	< 0.002
Clay and silt (undifferentiated)	< 0.08
Sand	0.08 to 5
Gravel	5 to 80
Cobble	80 to 300
Boulder	> 300

Descriptive terminology	**Proportion (%)**
"Traces" (tr.)	1 to 10
"Some" (s.)	10 to 20
Adjective (ex.: sandy, silty)	20 to 35
"And" (ex.: sand and gravel)	35 to 50

Compactness of cohesionless soils	**Standard Penetration Test index ("N" value), ASTM D-1586 (blows for a 300mm penetration)**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

Consistency of cohesive soils	**Undrained shear strength (kPa)**
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

Plasticity of cohesive soils	**Liquid limit (%)**
Low	< 30
Medium	30 to 50
High	> 50

Sensitivity of cohesive soils	**$S_t = (C_u/C_{ur})$**
Low	$S_t < 2$
Medium	$2 < S_t < 4$
High	$4 < S_t < 8$
Extra-sensitive	$8 < S_t < 16$
Quick (sensitive) clay	$S_t > 16$

Classification of rock	**RQD (%)**
Very poor quality	< 25
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

SYMBOLS

TOP SOIL	▨	SAND	░	COBBLE	▒
BACKFILL	▦	SILT	▨	BOULDER	▒
GRAVEL	▒	CLAY	▨	ROCK	░

WATER LEVEL

This column shows the ground water level, as measured at a given time during the geotechnical investigation. The details of the installation (type and depth) are also illustrated in this column.

SAMPLES

Type and number: Each sample is labelled in accordance with the number of this column and the given notation refers to samples types.

Sub-sample: When a sample contains two or more different stratigraphic units, it is sometimes necessary to separate it and create sub-samples. This column allows for the identification of the latter and the association to *in situ* or laboratory measurements to these sub-samples.

Condition: The position, length and condition of each sample are shown in this column. The symbol shows the condition of the sample, following the legend given on the sounding log.

Size: This column indicates the split spoon sampler size.

"N" index The standard penetration index shown in this column is expressed with the letter "N". This index is obtained with the Standard Penetration Test. It corresponds to the number of blows required to drive the last 300mm of the split spoon, using a 622 Newton hammer falling freely from a height of 762mm (ASTM D-1586). For a 610mm long split spoon, the "N" index is obtained by adding the number of blows required for the driving of the 2nd and 3rd 150mm of the split spoon. Refusal (R) indicates a number of blows greater than 100. A set of numbers such as 28-30-50/60mm indicates that the number of blows required to drive the 1st and 2nd 150mm of the split spoon are respectively 28 and 30. Moreover, it indicates that 50 blows were necessary to get a penetration of 60mm, whereupon the test was suspended.

RQD index: Rock Quality Designation index: This index is defined as the ratio between the total length of all rock cores of 100mm and more in length over the total length of the core run. The RQD index is an indirect measurement of the number of "natural" fractures and of the amount of the alteration in a rock mass.

TESTS

Results: This column shows, for the corresponding depth, the results of tests carried out in the field or in the laboratory (shear strength, dynamic penetration, Atterberg limits with the cone, etc.). For more information, please refer to the legend in the upper part of the sounding log. However, an abbreviation indicating the type of analysis performed is shown next to the sample tested.

Graph: This graph shows the undrained shear strength resistance of cohesive soils, as measured *in situ* or in the laboratory (NQ 2501-200). It is also used to present the Dynamic Cone Penetration Test (NQ 2501-145) results.

Moreover, this graph is used for the representation of the water content and Atterberg limits test results.

LVM

Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-01-12**
Date:	**2012-06-12**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):		
	North	4983184,0 (Y)
	East	444442,0 (X)
	Elevation	**98,90 (Z)**
Bedrock: 4,27 m	End depth:	7,24 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◫ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

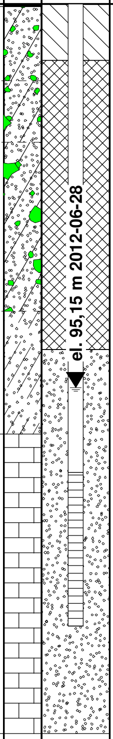
L	Consistency Limits
W_L	Liquid Limit (%)
W_P	Plastic Limit (%)
I_P	Plasticity Index (%)
I_L	Liquidity Index
W	Natural Water Content (%)
GS	Grain Size Analysis
S	Hydrometer analysis
R	Refusal
VBS	Methylene Blue Value
WR	Weight of Rods
O.M.	Organic Matter (%)
K	Permeability (cm/s)
UW	Unit Weight (kN/m³)
A	Absorption (l/min. m)
U	Uniaxial Compressive strength (MPa)
RQD	Rock Quality Designation (%)
CA	Chemical Analysis
P_L	Limit Pressure (kPa)
E_M	Pressuremeter Modulus (MPa)
E_r	Modulus of subgrade reaction (MPa)
SP_O	Segregation Potential (mm²/H °C)
▼	Water Level
N	Std Penetration test (blows/300mm)
N_C	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index

Undrained shear strength

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

STRATIGRAPHY

ELEVATION - m DEPTH - m	SOIL OR BEDROCK DESCRIPTION
98,90 0,00	Asphalt
98,88 0,03	***Fill:*** Gravelly sand, grey, with lump of clay
98,14 0,76	Clayey and gravelly sand with clay lump and concrete fragments
97,53 1,37	Clayey and gravelly sand with clay lump and concrete fragments, moist
95,86 3,05	Silt and fine sand, brown
94,64 4,27	***Rock :*** Limestone, black, fine-grained, with clay horizon
91,66 7,24	End of borehole



SAMPLES

TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual	RESULTS
SS-1		☒		43	1-5 5-3	10			
SS-2		☒		67	2-2 3-4	5			
SS-3		☒		48	1-3 5-50 /8 cm	8			
SS-4		☒		58	2-1 2-3	3			GS
SS-5		☒		42	1-3 7-5	10			
RC-6		◫		96		75			U= 100 MPa
RC-7		◫		95		88			

FIELD AND LABORATORY TESTS

NATURAL WATER CONTENT AND LIMITS (%)
Wp W WL
20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION
20 40 60 80 100 120

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Diamond** Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.** Approved by: **T. Lampron** 2012-07-11

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Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-03-12**
Date:	**2012-06-12**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):		
	North	4983146,0 (Y)
	East	444465,0 (X)
	Elevation	**99,13 (Z)**

Bedrock: 3,05 m End depth: 6,33 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◫ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	**O.M.**	Organic Matter (%)	▼	Water Level
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)	**N**	Std Penetration test (blows/300mm)
W_P	Plastic Limit (%)	**UW**	Unit Weight (kN/m³)	**N_C**	Dyn. Penetration test (blows/300mm) ●
I_P	Plasticity Index (%)	**A**	Absorption (l/min. m)	**σ'_P**	Preconsolidation Pressure (kPa)
I_L	Liquidity Index	**U**	Uniaxial Compressive strength (MPa)	**SCI**	Soil Corrosivity Index
W	Natural Water Content (%)	**RQD**	Rock Quality Designation (%)		
GS	Grain Size Analysis	**CA**	Chemical Analysis		**Undrained shear strength** Field Laboratory
S	Hydrometer analysis	**P_L**	Limit Pressure (kPa)	**C_U**	Undisturbed (kPa) ▲ ■
R	Refusal	**E_M**	Pressuremeter Modulus (MPa)	**C_{UR}**	Remoulded (kPa) △ □
VBS	Methylene Blue Value	**E_r**	Modulus of subgrade reaction (MPa)		
WR	Weight of Rods	**SP_O**	Segregation Potential (mm²/H °C)		

STRATIGRAPHY **SAMPLES** **FIELD AND LABORATORY TESTS**

ELEVATION - m DEPTH - m	SOIL OR BEDROCK DESCRIPTION	WATER LEVEL (m) / DATE	TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual	RESULTS
99,13												
0,00	Asphalt											
99,08 **0,05**	***Fill:*** Sandy gravel with some silt, grey-black, moist		SS-1		☒		33	2-3 7-2	10			
98,37 **0,76**	Fine sand with some silt, beige, dry		SS-2		☒		62	3-2 3-4	5			GS
			SS-3		☒		67	2-2 4-3	6			
96,84 **2,29**	Sandy silt with traces of gravel, grey, saturated, with trace of oxydation		SS-4		☒		58	4-6 4-50 /3 cm	10			
			SS-5				0	50	R			
96,08 **3,05**	***Rock :*** Limestone, black, fine-grained, with clay horizon		CR-6		◫		100		89			
			CR-7		◫		98		83			
			CR-8		◫		100		100			
92,80 **6,33**	End of borehole											

DEPTH - ft: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

DEPTH - m: 1 2 3 4 5 6 7 8

Organo. Exam

NATURAL WATER CONTENT AND LIMITS (%)
Wp W WL
20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION
20 40 60 80 100 120

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Diamond** Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.**	Approved by: **T. Lampron**	2012-07-11	Page: 1 of 1

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Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-04-12**
Date:	**2012-06-12**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):		
	North	4983177,0 (Y)
	East	444497,0 (X)
	Elevation	**99,61 (Z)**
Bedrock: 4,85 m	End depth:	8,86 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◨ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	**O.M.**	Organic Matter (%)
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)
W_P	Plastic Limit (%)	**UW**	Unit Weight (kN/m³)
I_P	Plasticity Index (%)	**A**	Absorption (l/min. m)
I_L	Liquidity Index	**U**	Uniaxial Compressive strength (MPa)
W	Natural Water Content (%)	**RQD**	Rock Quality Designation (%)
GS	Grain Size Analysis	**CA**	Chemical Analysis
S	Hydrometer analysis	P_L	Limit Pressure (kPa)
R	Refusal	E_M	Pressuremeter Modulus (MPa)
VBS	Methylene Blue Value	E_r	Modulus of subgrade reaction (MPa)
WR	Weight of Rods	SP_O	Segregation Potential (mm²/H °C)

▼	Water Level
N	Std Penetration test (blows/300mm)
N_C	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index

Undrained shear strength

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

STRATIGRAPHY			SAMPLES						FIELD AND LABORATORY TESTS
ELEVATION - m DEPTH - m	SOIL OR BEDROCK DESCRIPTION	WATER LEVEL (m) / DATE	TYPE AND NUMBER	CONDITION	RECOVERY %	Blows/150mm	"N" or RQD	RESULTS
99,61 0,00	Topsoil							
99,56 0,05	***Fill:*** Sandy silt with some gravel, grey-black		SS-1	☒	87	4-7 5-4	12	
98,85 0,76	Silty fine sand, brown, with clay lump and roots		SS-2	☒	79	4-4 4-4	8	
98,09 1,52	Silty fine sand, beige, with clay lump		SS-3	☒	83	4-3 3-3	6	
97,33 2,29	Clay, grey, with roots, moist		SS-4	☒	46	2-2 2-3	4	
96,56 3,05	Clay with trace of coarse gravel, grey, with roots, moist	el. 94,95 m 2012-06-28	SS-5	☒	42	2-3 8-12	11	
95,04 4,57	Sandy and silty gravel, grey-black	▼	SS-6	☒	60	25-50	R	
94,76 4,85	***Rock:*** Limestone, black, fine-grained, with clay horizon		RC-7	■	87		66	
			RC-8	■	100		83	
			RC-9	■	98		90	U = 107 MPa
90,75								

NATURAL WATER CONTENT AND LIMITS (%) Wp W WL: 20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION: 20 40 60 80 100 120

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Diamond** Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.** Approved by: **T. Lampron** 2012-07-11

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Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-05-12**
Date:	**2012-06-13**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):	North	4983153,0 (Y)
	East	444477,0 (X)
	Elevation	**99,15 (Z)**
Bedrock: m	End depth:	3,43 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◫ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	**O.M.**	Organic Matter (%)
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)
W_P	Plastic Limit (%)	**UW**	Unit Weight (kN/m³)
I_P	Plasticity Index (%)	**A**	Absorption (l/min. m)
I_L	Liquidity Index	**U**	Uniaxial Compressive strength (MPa)
W	Natural Water Content (%)	**RQD**	Rock Quality Designation (%)
GS	Grain Size Analysis	**CA**	Chemical Analysis
S	Hydrometer analysis	**P_L**	Limit Pressure (kPa)
R	Refusal	**E_M**	Pressuremeter Modulus (MPa)
VBS	Methylene Blue Value	**E_r**	Modulus of subgrade reaction (MPa)
WR	Weight of Rods	**SP_O**	Segregation Potential (mm²/H °C)

▼	Water Level
N	Std Penetration test (blows/300mm)
N_c	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index

Undrained shear strength

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

STRATIGRAPHY

ELEVATION - m DEPTH - m	SOIL OR BEDROCK DESCRIPTION
99,15 0,00	Asphalt
99,06 0,09	***Fill:*** Sandy silt with some gravel, grey-black
98,54 0,61	Gravelly sand with some silt, grey, with trace of oxydation
97,93 1,22	Sand with some silt and trace of gravel, grey-beige, with zone of black sand, moist
97,32 1,83	Sand with some silt and trace of gravel, grey, moist
96,71 2,44	Silty sand with trace of gravel, grey, very moist
95,72 3,43	End of borehole

SAMPLES

TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual	RESULTS
SS-1		☒		49	1-7 9-11	16			
SS-2		☒		58	6-13 6-7	19			
SS-3		☒		75	6-12 9-9	21			VOC: 70ppm
SS-4		☒		71	3-4 5-5	9			VOC: 0ppm
SS-5		☒		58	2-3 4-4	7			VOC: 0ppm
SS-6		☒		40	2-8 50 /8 cm	R			

FIELD AND LABORATORY TESTS

NATURAL WATER CONTENT AND LIMITS (%)
Wp W WL
20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION
20 40 60 80 100 120

DEPTH - ft: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29
DEPTH - m: 1 2 3 4 5 6 7 8

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Auger**

Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.**	Approved by: **T. Lampron**	2012-07-11	Page: 1 of 1

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Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-06-12**
Date:	**2012-06-13**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):		
	North	4983159,0 (Y)
	East	444468,0 (X)
	Elevation	**99,26 (Z)**
Bedrock: m	End depth:	3,12 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◧ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits
W_L	Liquid Limit (%)
W_P	Plastic Limit (%)
I_P	Plasticity Index (%)
I_L	Liquidity Index
W	Natural Water Content (%)
GS	Grain Size Analysis
S	Hydrometer analysis
R	Refusal
VBS	Methylene Blue Value
WR	Weight of Rods
O.M.	Organic Matter (%)
K	Permeability (cm/s)
UW	Unit Weight (kN/m³)
A	Absorption (l/min. m)
U	Uniaxial Compressive strength (MPa)
RQD	Rock Quality Designation (%)
CA	Chemical Analysis
P_L	Limit Pressure (kPa)
E_M	Pressuremeter Modulus (MPa)
E_r	Modulus of subgrade reaction (MPa)
SP_O	Segregation Potential (mm²/H °C)
▼	Water Level
N	Std Penetration test (blows/300mm)
N_C	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index

Undrained shear strength

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

STRATIGRAPHY

ELEVATION - m DEPTH - m	SOIL OR BEDROCK DESCRIPTION
99,26 0,00	Asphalt
99,20 0,05	***Fill:*** Gravelly sand with some silt, grey
98,65 0,61	Clayey silt, grey
97,43 1,83	Silty sand with trace of gravel and clay, grey, with trace of oxydation, very moist
96,82 2,44	Silty sand with some gravel, grey, very moist
96,21 3,05	Sand with some silt and rock fragment, grey
96,13 3,12	End of borehole

SAMPLES

TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Organo. Exam Odor	Organo. Exam Visual	RESULTS
SS-1		☒		46	1-7 3-2	10			VOC: 0ppm
SS-2		☒		83	2-3 4-7	7			VOC: 0ppm
SS-3		☒		87	3-3 4-4	7			VOC: 0ppm
SS-4		☒		46	4-2 3-3	5			
SS-5		☒		71	2-1 4-8	5			VOC: 30ppm
SS-6		☒		100	50 /8 cm	R			

FIELD AND LABORATORY TESTS

NATURAL WATER CONTENT AND LIMITS (%)
Wp W WL
20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION
20 40 60 80 100 120

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Auger**

Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.**

Approved by: **T. Lampron**

2012-07-11

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Client :

BROCCOLINI CONSTRUCTION inc.

BOREHOLE REPORT

File n°:	**B-0001436-1**
Borehole n°:	**BH-07-12**
Date:	**2012-06-13**

Project: **Phase II Environmental Site Investigation and Geotechnical Investigation**

Location: **400 Albert street, Ottawa, ON**

Coordinates (m):		
	North	4983169,0 (Y)
	East	444460,0 (X)
	Elevation	**99,59 (Z)**
Bedrock: m	End depth:	2,97 m

Sample condition

▨ Intact ☒ Remoulded ■ Lost ◨ Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	LVM Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	**O.M.**	Organic Matter (%)
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)
W_P	Plastic Limit (%)	**UW**	Unit Weight (kN/m³)
I_P	Plasticity Index (%)	**A**	Absorption (l/min. m)
I_L	Liquidity Index	**U**	Uniaxial Compressive strength (MPa)
W	Natural Water Content (%)	**RQD**	Rock Quality Designation (%)
GS	Grain Size Analysis	**CA**	Chemical Analysis
S	Hydrometer analysis	**P_L**	Limit Pressure (kPa)
R	Refusal	**E_M**	Pressuremeter Modulus (MPa)
VBS	Methylene Blue Value	**E_r**	Modulus of subgrade reaction (MPa)
WR	Weight of Rods	**SP_O**	Segregation Potential (mm²/H °C)

▼	Water Level
N	Std Penetration test (blows/300mm)
N_C	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index

Undrained shear strength

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

STRATIGRAPHY

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION
99,59	
0,00	Asphalt
99,51 0,08	***Fill:*** Clayey silt with some sand and trace of gravel, grey
98,98 0,61	Clayey silt, grey
97,76 1,83	Clayey silt with some sand and trace of gravel, grey
97,15 2,44	Silty sand with some gravel et trace of rock fragment, grey
96,62 2,97	End of borehole

SAMPLES

TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual	RESULTS
SS-1		☒		48	2-5 4-3	9			
SS-2		☒		83	3-4 5-4	9			VOC: 10ppm
SS-3		☒		87	2-3 5-5	8			VOC: 0ppm
SS-4		☒		67	4-5 6-6	11			VOC: 0ppm
SS-5		☒		76	1-50 /30 cm	R			VOC: 0ppm

WATER LEVEL (m) / DATE

FIELD AND LABORATORY TESTS

NATURAL WATER CONTENT AND LIMITS (%) Wp W WL
20 40 60 80 100 120

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION
20 40 60 80 100 120

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Auger**

Boring equipment: **CME-75**

Prepared by: **S. Séguin, tech.**

Approved by: **T. Lampron**

2012-07-11

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

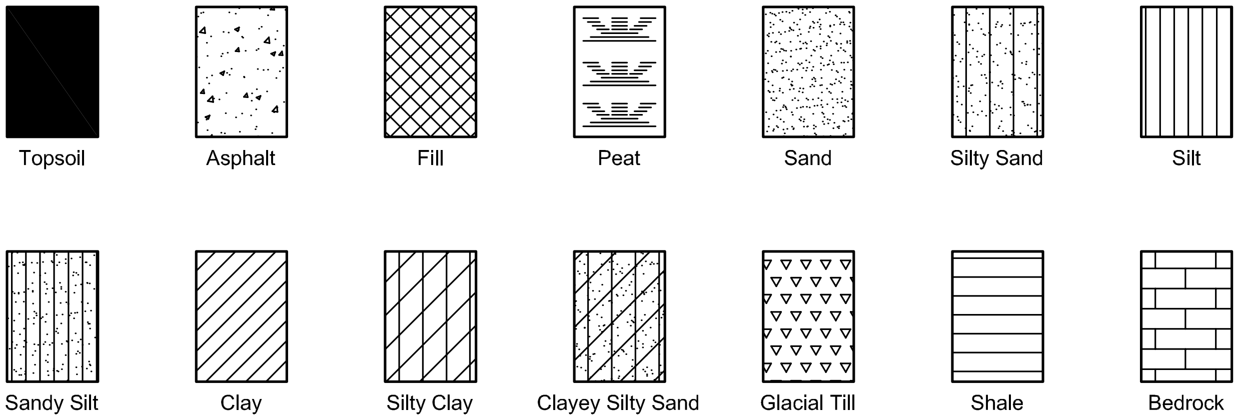
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

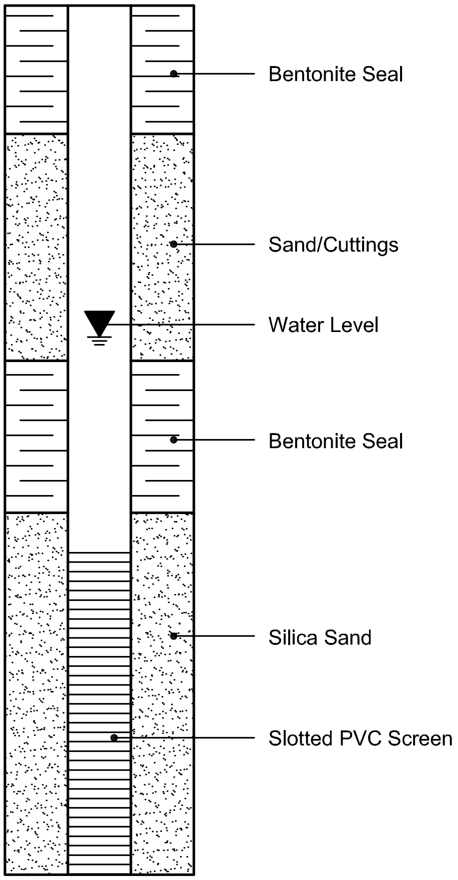
SYMBOLS AND TERMS (continued)

STRATA PLOT

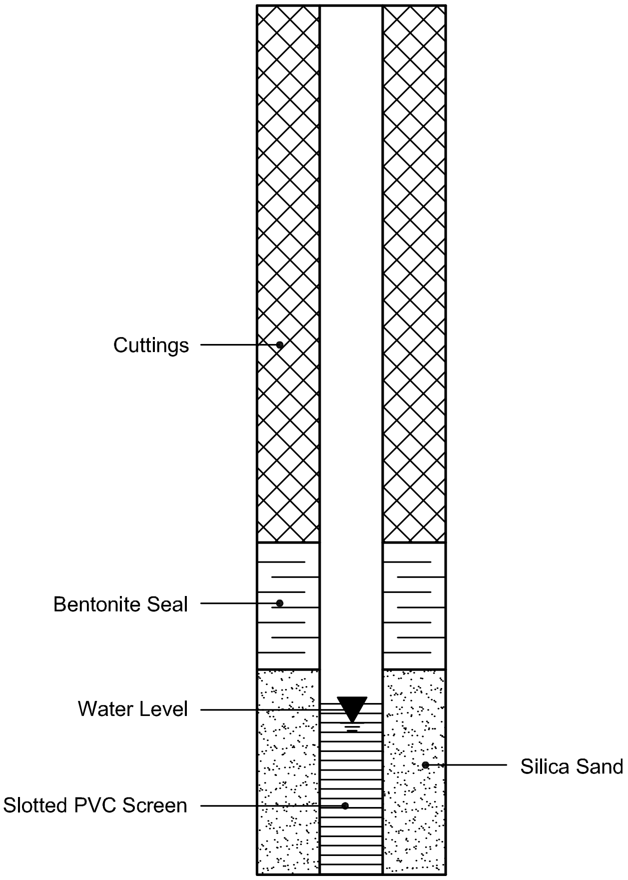


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
383 Slater Street
Ottawa, Ontario

DATUM Benchmark (BM) - Top cover of manhole located along east side of Bay Street, just north of Slater Street. Geodetic elevation = 72.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 9, 2016

FILE NO. PE3774

HOLE NO. BH 8-16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE					0	72.08	
FILL: Crushed stone, trace brick 0.25	AU	1					
FILL: Brown sandy silt with topsoil, some brick, mortar, trace wood and gravel 1.62	SS	2	79	11	1	71.08	
FILL: Brown clayey silt with sand, trace gravel and organics	SS	3	88	13	2	70.08	
2.97	SS	4	79	8			
Brown **CLAYEY SILT,** trace sand 3.35					3	69.08	
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders	SS	5	50	10			∇
	SS	6	71	15	4	68.08	
4.90	SS	7	85	50+			

End of Borehole

Practical refusal to augering at 4.90m depth

(GWL @ 4.8m depth based on field observations)

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**
20 40 60 80

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
383 Slater Street
Ottawa, Ontario

DATUM Benchmark (BM) - Top cover of manhole located along east side of Bay Street, just north of Slater Street. Geodetic elevation = 72.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 18, 2017

FILE NO. PE3774

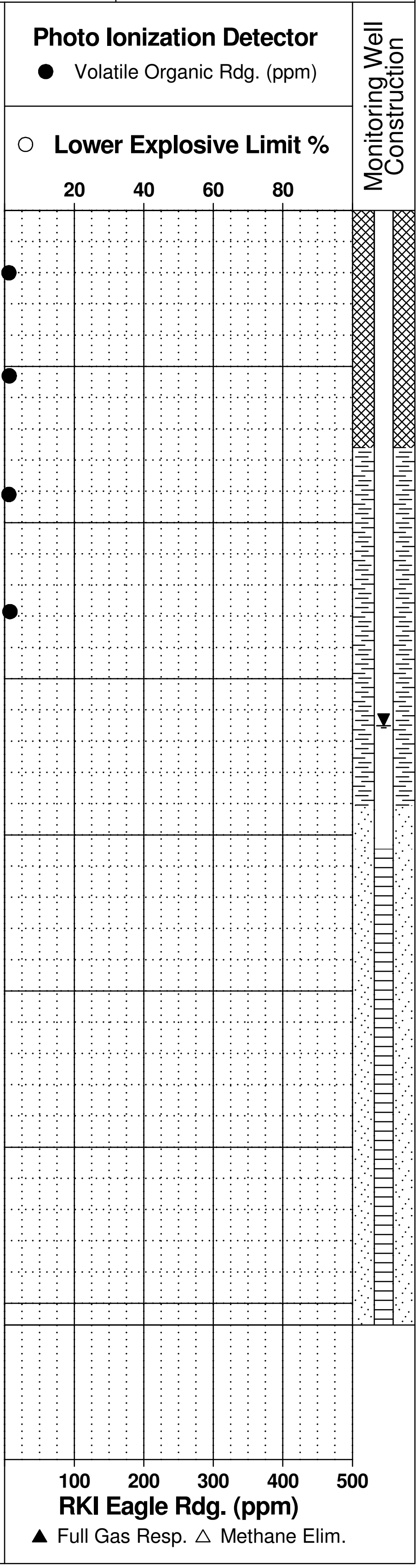
HOLE NO. BH 9

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Crushed stone	0.25				
FILL: Brown silty clay, some sand	0.71	SS	1	75	12
FILL: Brown silty sand, some clay	1.37	SS	2	67	5
Very stiff, brown **SILTY CLAY,** some sand		SS	3	92	5
	2.90	SS	4	100	8
BEDROCK: Good to excellent quality, grey limestone		RC	1	100	77
		RC	2	100	97
	7.14	RC	3	100	97

End of Borehole

(GWL @ 3.30m - April 27, 2017)

DEPTH (m)	ELEV. (m)
0	72.06
1	71.06
2	70.06
3	69.06
4	68.06
5	67.06
6	66.06
7	65.06



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
383 Slater Street
Ottawa, Ontario

DATUM Benchmark (BM) - Top cover of manhole located along east side of Bay Street, just north of Slater Street. Geodetic elevation = 72.77m.

REMARKS

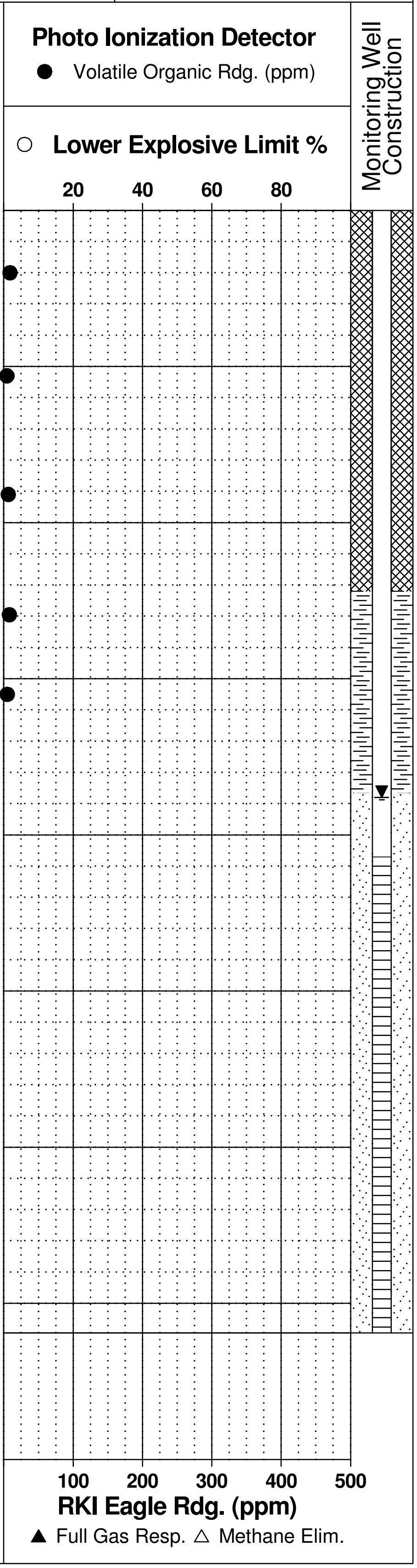
BORINGS BY CME 55 Power Auger

DATE April 18, 2017

FILE NO. PE3774

HOLE NO. BH10

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.90
Asphaltic concrete 0.10						
FILL: Crushed stone 0.30						
FILL: Brown silty sand, trace clay	SS	1	58	7		
	SS	2	83	5	1	70.90
1.37						
FILL: Brown sand, trace silt	SS	3	75	6		
2.13					2	69.90
Compact, brown **SILTY SAND,** some clay	SS	4	83	10		
2.90						
Very stiff, grey-brown **CLAYEY SILT** 3.20	SS	5	100	50+	3	68.90
	RC	1	89	66		
					4	67.90
BEDROCK: Fair to excellent quality, grey limestone	RC	2	100	98		
					5	66.90
					6	65.90
	RC	3	100	100		
					7	64.90
7.19						
End of Borehole						
(GWL @ 3.76m - April 27, 2017)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
383 Slater Street
Ottawa, Ontario

DATUM Benchmark (BM) - Top cover of manhole located along east side of Bay Street, just north of Slater Street. Geodetic elevation = 72.77m.

REMARKS

BORINGS BY CME 55 Power Auger

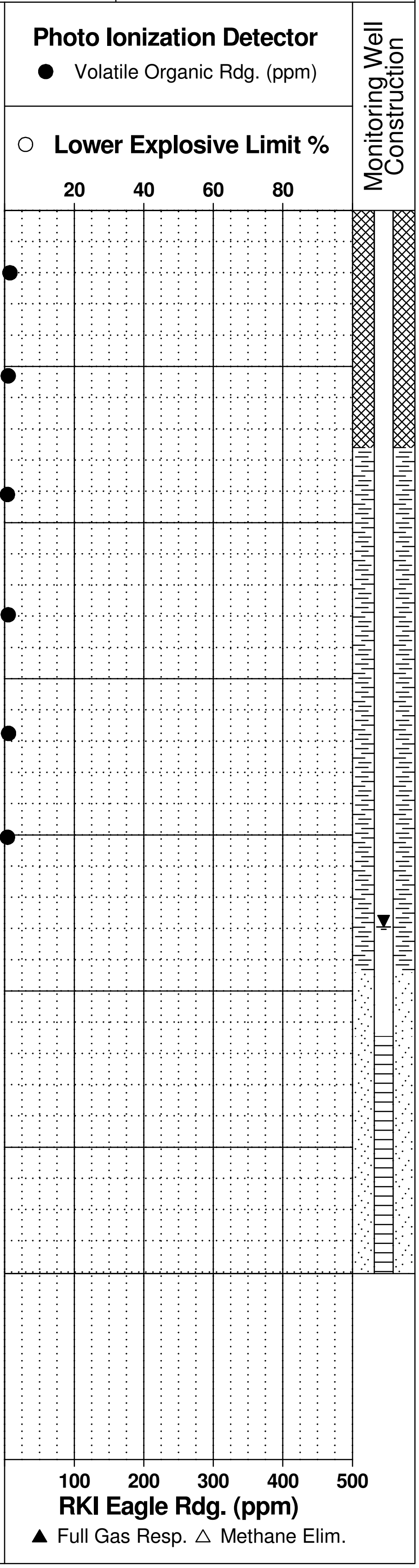
DATE April 18, 2017

FILE NO. PE3774

HOLE NO. BH11

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Crushed stone	0.25				
FILL: Brown clayey sand, some silt	0.90	SS	1	58	4
Very stiff, brown **SILTY CLAY** - some sand by 2.7m depth	3.66	SS	2	83	4
		SS	3	100	12
		SS	4	100	6
		SS	5	83	7
Very dense, brown **SILTY SAND,** some clay	4.27	SS	6	62	50+
BEDROCK: Excellent quality, grey limestone	6.81	RC	1	98	93
		RC	2	100	98
End of Borehole (GWL @ 4.59m - April 27, 2017)					

DEPTH (m)	ELEV. (m)
0	72.60
1	71.60
2	70.60
3	69.60
4	68.60
5	67.60
6	66.60



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
383 Slater Street
Ottawa, Ontario

DATUM Benchmark (BM) - Top cover of manhole located along east side of Bay Street, just north of Slater Street. Geodetic elevation = 72.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 18, 2017

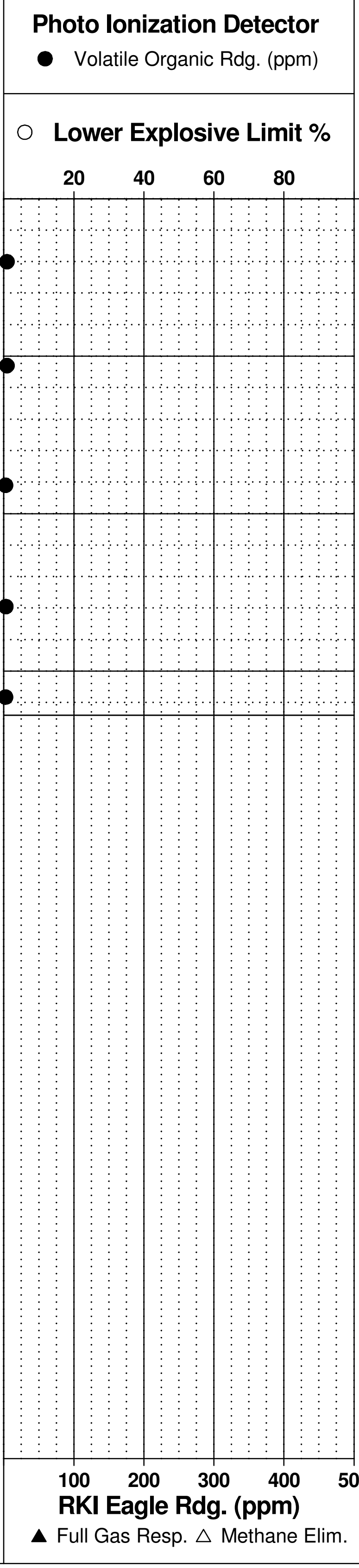
FILE NO. PE3774

HOLE NO. BH12

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Crushed stone	0.30
FILL: Brown silty sand	0.76
Very stiff, brown **SILTY CLAY** - some sand by 2.1m depth	2.90
Dense, brown **SILTY SAND,** some clay	3.20
End of Borehole Practical refusal to augering at 3.20m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	58	11
SS	2	100	11
SS	3	100	7
SS	4	100	3
SS	5	67	50+

DEPTH (m)	ELEV. (m)
0	72.30
1	71.30
2	70.30
3	69.30



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Groundwater Quality Assessment
Proposed Development - 383 Slater St. & 400 Albert St.
Ottawa, Ontario

DATUM TBM - Top of manhole located on the east side of Bay Street, just north of Slater Street. A geodetic elevation of 72.77m was provided by Annis, O'Sullivan, Vollebekk Ltd. for the TBM.

REMARKS

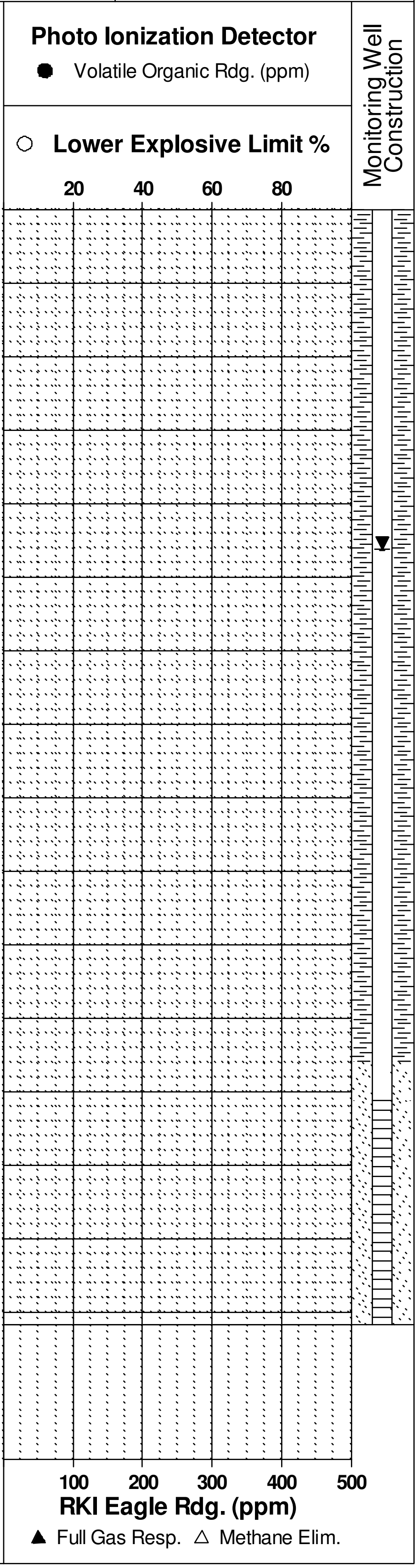
BORINGS BY CME 55 Power Auger

DATE June 5, 2015

FILE NO. PG3543

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.10	AU	1			0	72.74
FILL: Brown silty sand, some gravel and cobbles	SS	2	62	3	1	71.74
- some brick and concrete by 0.4m depth 2.29	SS	3	25	16	2	70.74
	RC	1	94	61	3	69.74
	RC	2	100	86	4	68.74
	RC	3	97	92	5	67.74
					6	66.74
BEDROCK: Grey limestone with shale lenses	RC	4	98	95	7	65.74
	RC	5	98	77	8	64.74
					9	63.74
	RC	6	97	97	10	62.74
- calcite noted from 10.5m to 11.9m depth	RC	7	100	97	11	61.74
					12	60.74
	RC	8	98	98	13	59.74
	RC	9	100	100	14	58.74
15.16					15	57.74
End of Borehole (GWL @ 4.61m-August 17, 2015)						



SOIL PROFILE AND TEST DATA

Groundwater Quality Assessment
Proposed Development - 383 Slater St. & 400 Albert St.
Ottawa, Ontario

DATUM TBM - Top of manhole located on the east side of Bay Street, just north of Slater Street. A geodetic elevation of 72.77m was provided by Annis, O'Sullivan, Vollebekk Ltd. for the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 5, 2015

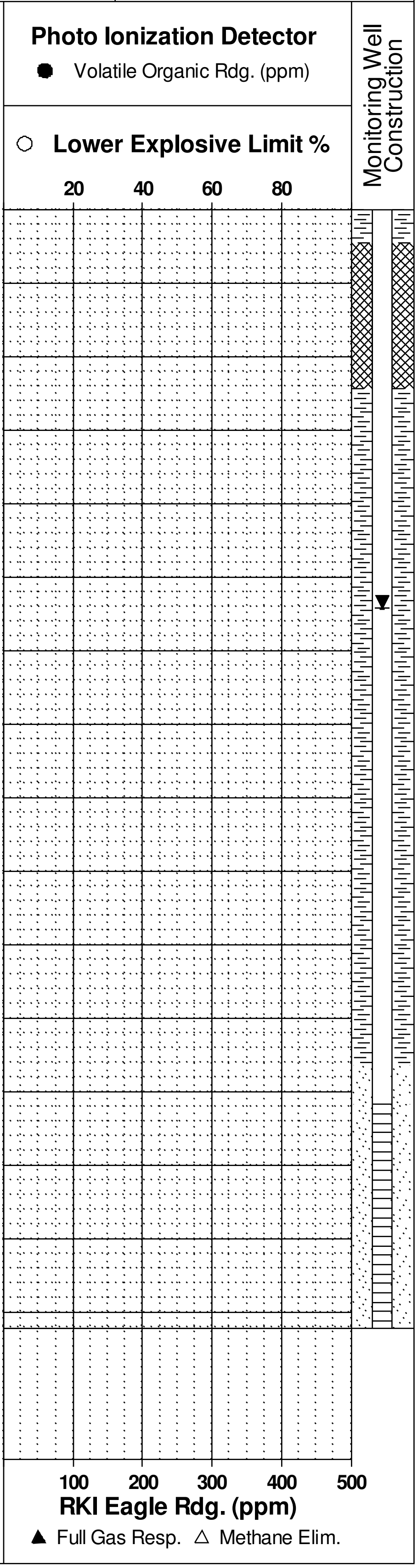
FILE NO. PG3543

HOLE NO. BH 2

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone with sand	0.25
FILL: Brown silty sand with crushed stone, trace brick	1.52
FILL: Gravel and boulders, some silt and sand	2.44
Grey **SILTY CLAY,** trace sand	2.87
GLACIAL TILL: Grey-brown silty sand with gravel and cobbles	4.27
BEDROCK: Grey limestone with shale lenses	15.22
End of Borehole	
(GWL @ 5.41m-August 17, 2015)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	19
SS	3	67	45
SS	4	70	10
SS	5	42	12
SS	6	72	14
RC	1	100	97
RC	2	98	98
RC	3	98	88
RC	4	95	95
RC	5	100	100
RC	6	98	98
RC	7	97	97
RC	8	100	100

DEPTH (m)	ELEV. (m)
0	73.72
1	72.72
2	71.72
3	70.72
4	69.72
5	68.72
6	67.72
7	66.72
8	65.72
9	64.72
10	63.72
11	62.72
12	61.72
13	60.72
14	59.72
15	58.72



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Groundwater Quality Assessment
Proposed Development - 383 Slater St. & 400 Albert St.
Ottawa, Ontario

DATUM TBM - Top of manhole located on the east side of Bay Street, just north of Slater Street. A geodetic elevation of 72.77m was provided by Annis, O'Sullivan, Vollebekk Ltd. for the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

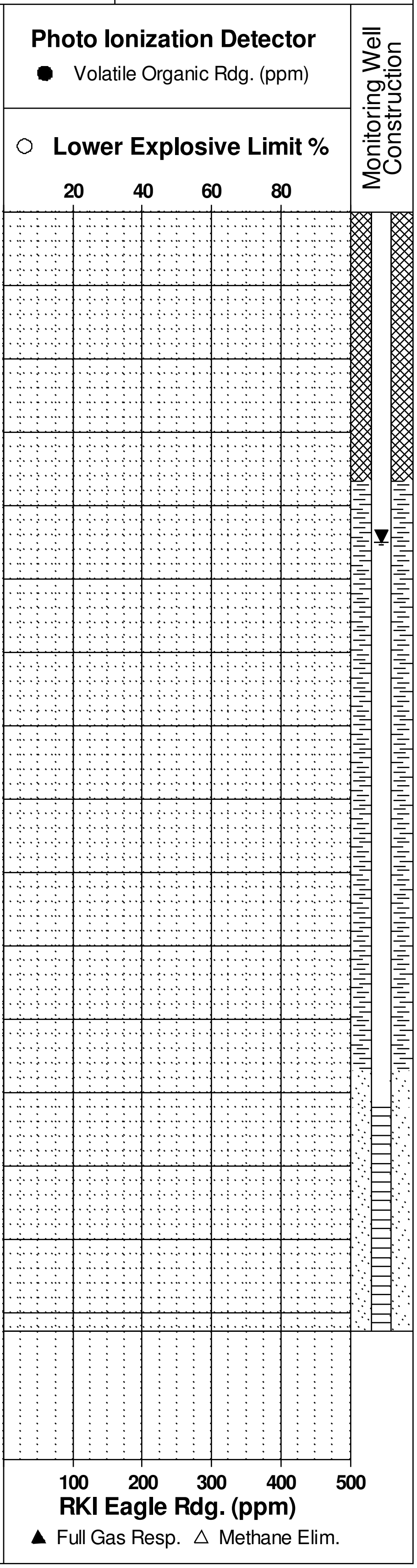
DATE June 8, 2015

FILE NO. PG3543

HOLE NO. BH3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.10	AU	1		
FILL: Crushed stone with silty sand 0.76				
FILL: Brown silty sand 1.55	SS	2	67	4
GLACIAL TILL: Grey silty sand, some gravel, cobbles and boulders 3.68	SS	3	96	9
	SS	4	71	50+
	RC	1		
BEDROCK: Grey limestone with shale lenses 15.24	RC	2		
	RC	3	97	90
	RC	4	100	100
	RC	5	97	85
	RC	6	100	100
	RC	7	100	90
	RC	8	100	100
	RC	9	100	100
End of Borehole				
(GWL @ 4.50m-August 17, 2015)				

DEPTH (m)	ELEV. (m)
0	72.12
1	71.12
2	70.12
3	69.12
4	68.12
5	67.12
6	66.12
7	65.12
8	64.12
9	63.12
10	62.12
11	61.12
12	60.12
13	59.12
14	58.12
15	57.12



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4793-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

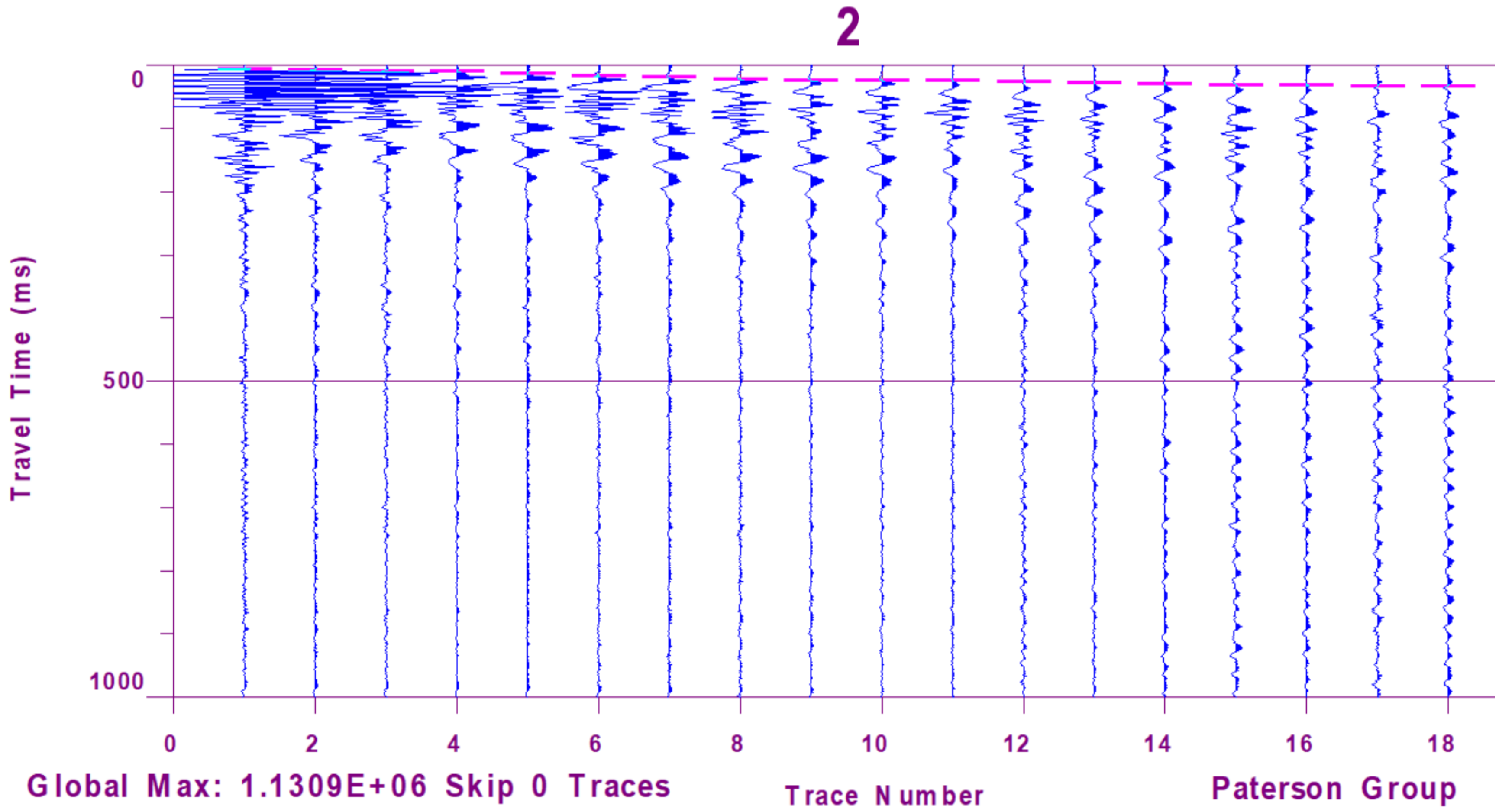


Figure 2 – Shear Wave Velocity Profile at Shot Location -4.5 m

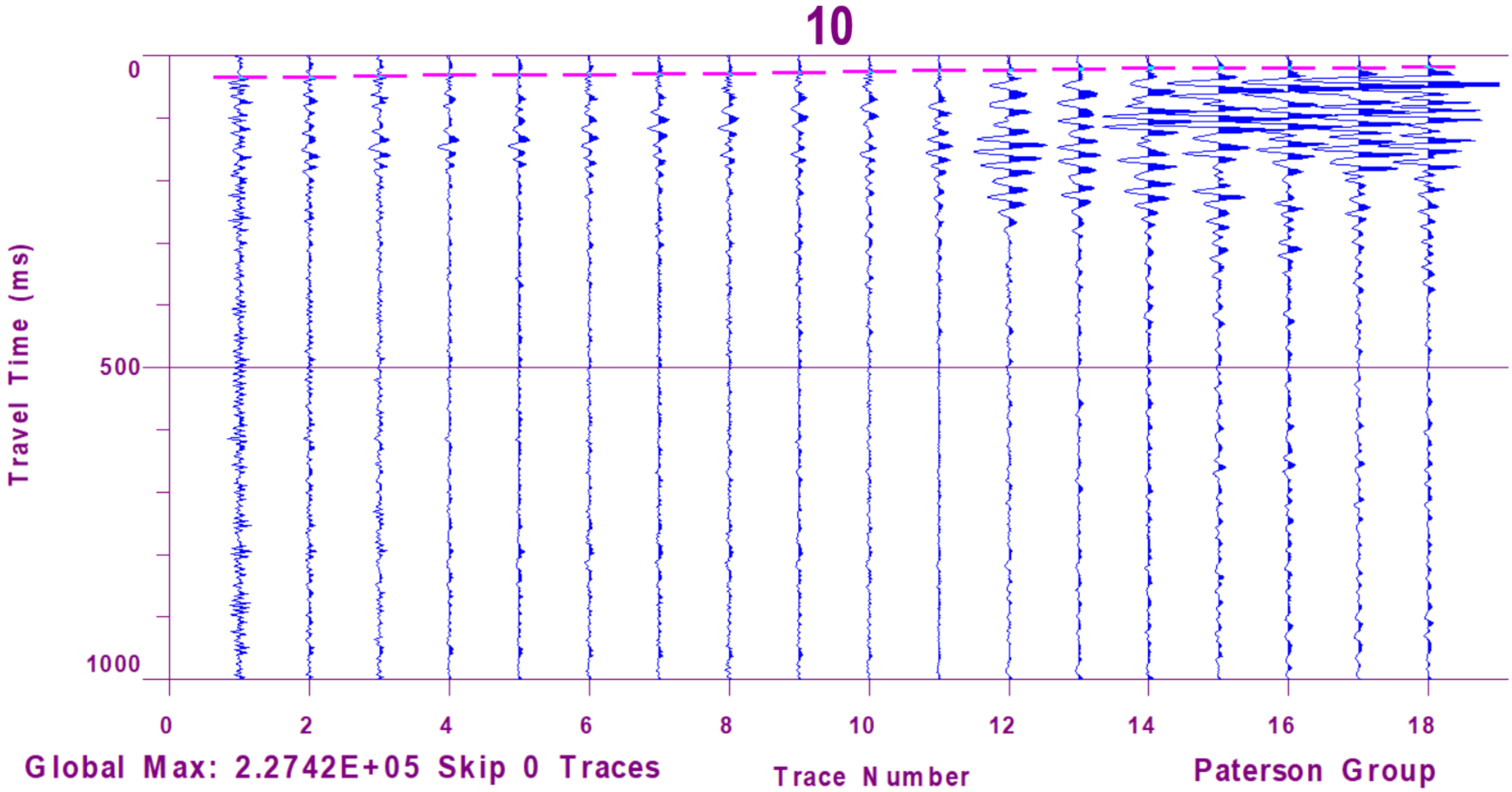
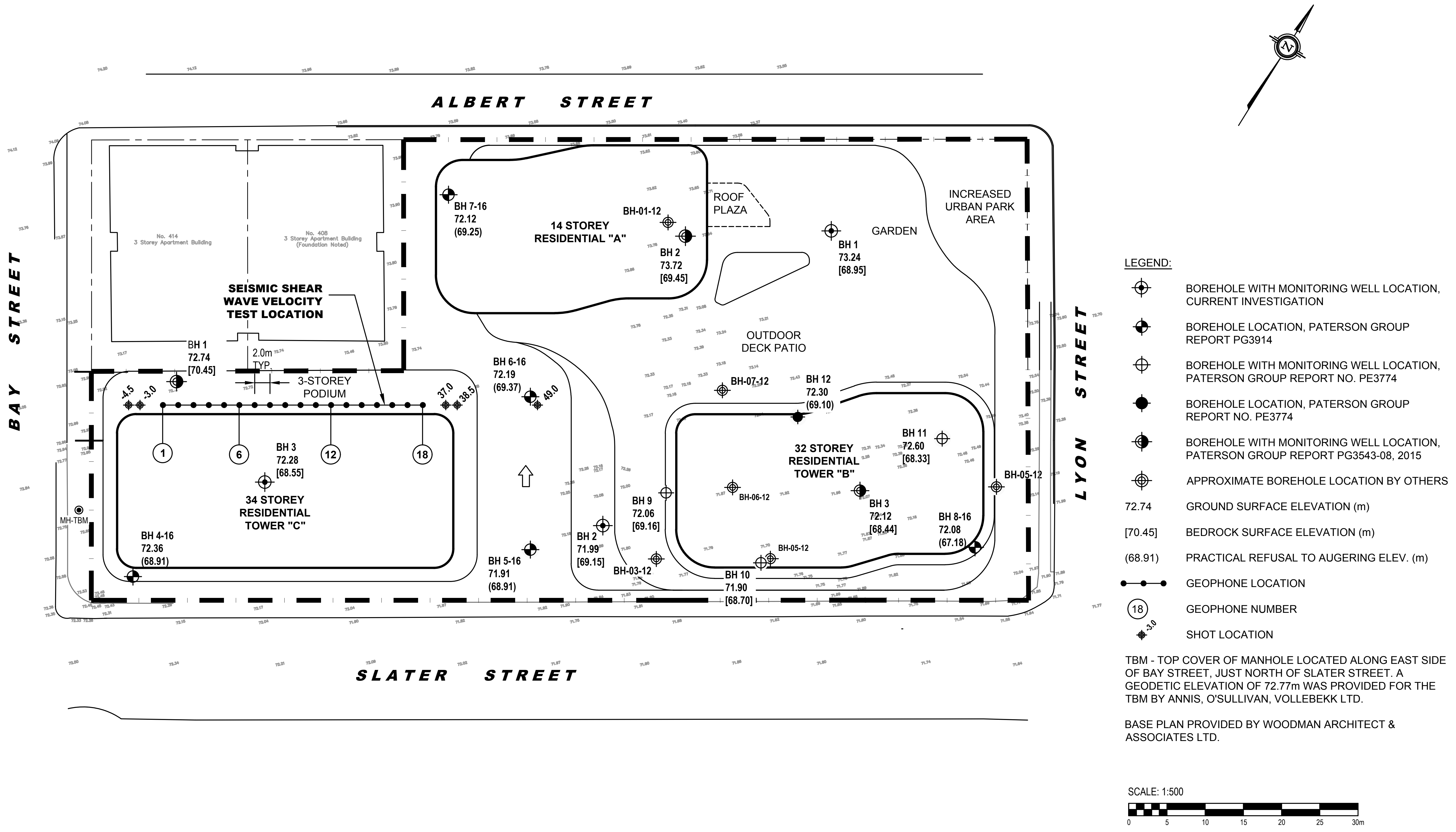


Figure 3 – Shear Wave Velocity Profile at Shot Location 49 m



TBM - TOP COVER OF MANHOLE LOCATED ALONG EAST SIDE OF BAY STREET, JUST NORTH OF SLATER STREET. A GEODETIC ELEVATION OF 72.77m WAS PROVIDED FOR THE TBM BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

BASE PLAN PROVIDED BY WOODMAN ARCHITECT & ASSOCIATES LTD.

patersongroup
consulting engineers

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Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

MAIN AND MAIN DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
MULTI-STOREY REDEVELOPMENT - 383 SLATER ST. & 400 ALBERT ST.
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 04/2019
Drawn by: RCG	Report No.: PG4793-1
Checked by: NC	Dwg. No.: **PG4793-1**
Approved by: DJG	Revision No.: 0

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