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Geotechnical Investigation
Proposed Multi-Storey Development
1068 to 1090 Cummings Avenue
Ottawa, Ontario

Prepared For

Huntington Property Group
(itf Cummings Caron JV)

May 27, 2019

Report PG4875-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Huntington Property Group (itf Cummings Caron JV) to conduct a geotechnical investigation for the proposed multi-storey Development to be located at 1068 to 1090 Carling Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❑ 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.

2.0 Proposed Project

Based on available project details, it is our understanding that the proposed development will consist of three multi-storey buildings with one basement level of underground parking along with at grade parking areas and access lanes. The three proposed buildings will be connected by 2 storey podium buildings in between the multi-storey buildings. It is further understood that the existing buildings will be demolished as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Field programs for geotechncial and environmental investigations were carried out September 16 and 19, 2016. During that time, a total of 7 boreholes were advanced to a maximum depth of 7.6 m below existing ground surface. A supplemental investigation was conducted on March 26 and 27, 2019. At that time, 6 boreholes were advanced to a maximum depth of 9.6 m. The borehole locations were determined in the field by Paterson personnel and distributed in a to provide general of coverage of areas of potential environmental concerns and address of the proposed development taking into consideration the existing site features and underground services. The locations of the boreholes are presented in Drawing PG4875-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights, using a 50 mm diameter split-spoon sampler or using 47.6 mm inside diameter coring equipment. All soil samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger, split spoon and rock core samples were recovered from the test holes are shown as, AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The Standard Penetration Test (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.

Undrained shear strength testing, using a vane apparatus, was carried out in cohesive soils.

Rock coring was carried out in BH 1 to BH 3 in September 2016 as well as in BH 1-19, BH 5-19 and BH 6-19 in March 2019 to confirm depth to bedrock. The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

51 mm diameter PVC groundwater monitoring wells were installed in BH 1, BH 2, BH 3, BH 1-19, BH 5-19 and BH 6-19 and flexible polytubing was installed in all the other boreholes of March 2019 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 1.5 m long slotted 51 mm diameter PVC screen.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ Bentonite hole plug directly above PVC slotted screen to approximately 300 mm below the ground surface.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

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

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground utilities and existing site features. The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located at the southeast corner of the intersection of Snow Street and Cummings Avenue. An assumed elevation of 100 m was assigned to the TBM. The location of the TBM, boreholes and ground surface elevation at each borehole location are presented on Drawing PG4875-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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.

3.4 Analytical Testing

One soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Subsection 6.7 and shown in appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is situated on the west side of Cummings Avenue, between Caron Street and Donald Street, in the city of Ottawa. An automotive body shop, steel storage and sales office, and landscaping business occupy the property.

The site is relatively flat and approximately at grade with the adjacent roadway and properties. The majority of the site is covered with asphalt pavement or granular surfaces.

4.2 Subsurface Profile

Generally, the subsoil profile encountered at the borehole locations consists of an asphalt pavement structure or granular fill overlying a fill layer of brown silty sand with gravel. Under the fill layer, a layer of topsoil was encountered underlain by native brown silty sand in BH 1, BH 6, BH1-19, BH 2-19 and BH 3-19. A weathered black shale bedrock was encountered at all borehole locations.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

Bedrock Depth

Practical refusal to augering was encountered at depths varying between 1.0 to 4.2 m depth. Local bedrock elevations were confirmed by rock coring at depths varying between 1.8 m and 9.5 m in BH 1-17, BH 2-17, BH 3-17, BH 1-19, BH 5-19 and BH 6-19 .

Available Bedrock Mapping

Based on available geological mapping, the subject site is located in an area where bedrock consists of shale of the Billings Formation, and is expected at depths between 2 to 5 m.

4.3 Groundwater

Groundwater levels were measured in monitoring wells installed in the boreholes upon completion of the sampling program. The groundwater level readings are presented in Table 1. Long-term groundwater level can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected to be within the bedrock layer.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could be higher at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)		Recording Date
		Depth	Elevation	
BH 1	72.48	3.44	69.04	September 23, 2016
BH 2	75.33	2.83	69.50	September 23, 2016
BH 3	72.27	2.30	69.97	September 23, 2016
BH 1-19	72.44	2.08	70.36	April 12, 2019
BH 2-19	72.68	2.10	70.58	April 12, 2019
BH 3-19	75.76	1.93	70.83	April 12, 2019
BH 5-19	72.22	1.61	70.61	April 12, 2019
BH 6-19	72.30	2.40	69.90	April 12, 2019

Notes: Elevation referred to a temporary benchmark (TBM) which consists of the top spindle of a fire hydrant at the intersection of Snow Street and Cummings Avenue. A geodetic elevation of 73.86 m was provided to the TBM by Anis O'Sullivan Vollebekk Ltd.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed building could be founded by conventional spread footings placed on surface sounded bedrock bearing surface. Where the design underside of footing level will not encounter bedrock, Paterson recommends extending the footing to bedrock by means of a near vertical, zero entry trench extending at least 200 mm beyond the outside face of the footing could be excavated through the existing fill to an approved bearing surface. The trench should be in-filled to the proposed founding elevation with a minimum 15 MPa lean concrete.

Bedrock removal is expected for the proposed development. Hoe ramming should be completed where small quantities of bedrock needed to be removed. Line drilling and controlled blasting could be used where large quantities of bedrock are needed to be removed. The blasting operations should be planned and conducted under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and asphalt should be removed from within the perimeter of the proposed building and other settlement sensitive structures. Foundation walls, underground services, and other construction debris should be entirely removed from within the perimeter of the buildings. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Bedrock Removal

Based on the bedrock encountered in the area, line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock.

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s 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 an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

Vibration Considerations

Construction operations could cause 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 a cooperative environment with the residents.

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

Two parameters determine the recommended vibration limit, 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). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Conventional Shallow Foundation

Footings placed on a clean, surface sounded bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

Footings bearing on surface sounded bedrock and designed using the above mentioned bearing pressures will be subjected to negligible post-construction total and differential settlements.

For areas where bedrock is encountered below design underside of footing level, lean concrete (min. 15 MPa) in-filled trenches could be used to extend footings to an approved bedrock surface. Near vertical, zero entry trenches extending at least 200 mm wider than the proposed footing face should be extended through the overlying soils to an approved bedrock bearing surface. The bearing surface should be inspected by Paterson personnel and in-filled with a lean concrete to design underside of footing level.

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 stiff silty clay or glacial till bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil. 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).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the shallow foundations at the subject site. A higher site class, such as Class A or B, may be applicable for this site and can be confirmed by a site specific shear wave velocity test. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

The native soil or approved fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone for the basement floor slab used for finished space. In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

However, 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.

Lateral Earth Pressures

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 (0.5)
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 Earth Pressures

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 Pavement Structure

Car only parking and heavy truck parking areas, and access lanes may be required at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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 recommended that a perimeter foundation drainage system be provided for all the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or building sump pit. A waterproofing system should be provided for the elevator pit (pit bottom and walls).

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Alternatively, where foundation walls are to be formed against a temporary shoring system, the following is recommended. A composite drainage system should be fastened to the shoring face. It is recommended that 150 mm diameter sleeves at 6 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. An interior perimeter drainage consisting of a minimum 150 mm diameter perforated, corrugated PVC pipe should be placed along the interior side of the exterior footing. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

Exterior unheated footings 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

At this site, temporary shoring may be required to complete the required excavations. However, it is recommended that where sufficient room is available open cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. 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.

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

Table 4 - 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	18
Submerged Unit Weight (γ), kN/m^3	11

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.

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. 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.

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.

Soldier Pile and 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 pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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.

Permit to Take Water

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped 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.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects 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.

Precaution 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.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

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:

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

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

❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❑ 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 made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A preliminary geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order 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 Huntington Property Group (itf Cummings Caron JV) or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 27, 2019
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc, EIT

David J. Gilbert, P.Eng.

Report Distribution

❑ Huntington Property Group (itf Cummings Caron JV)
❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1090 Cummings Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Cummings Avenue and Snow Street. Geodetic elevation = 73.86m, as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

HOLE NO. BH 1-19

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel	1.90
TOPSOIL	2.08
Brown **SILTY SAND**	2.74
BEDROCK: Black shale	7.06
End of Borehole (GWL @ 2.08m - April 12, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	35
SS	3	58	6
SS	4	80	50+
AU	5		
RC	1	100	78
RC	2	100	65

DEPTH (m)	ELEV. (m)
0	72.44
1	71.44
2	70.44
3	69.44
4	68.44
5	67.44
6	66.44
7	65.44

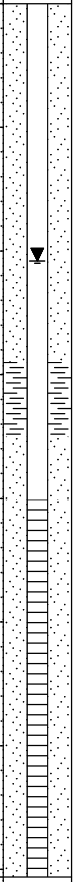
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ 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
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

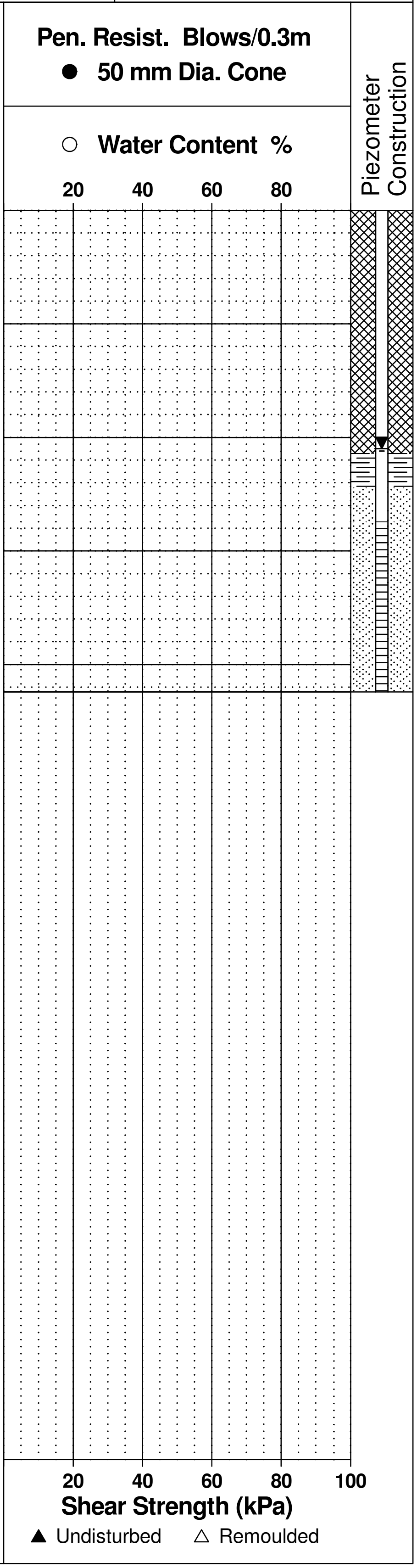
HOLE NO. BH 2-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	72.68
FILL: Brown silty sand with gravel 0.30	AU	1				
FILL: Brown silty sand with gravel, trace clay	SS	2	67	10	1	71.68
1.83	SS	3	71	10		
TOPSOIL 2.00					2	70.68
Compact to loose, brown **SILTY SAND**	SS	4		12		
- some gravel by 2.4m depth					3	69.68
	SS	5		7		
	SS	6		50+		
	AU	7				
4.24					4	68.68

End of Borehole

Practical refusal to augering at 4.24m depth

(GWL @ 2.10m - April 12, 2019)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

HOLE NO. BH 3-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	72.76
FILL: Brown silty sand with gravel	AU	1				
	SS	2	71	52	1	71.76
1.62						
TOPSOIL 1.75	SS	3	83	8	2	70.76
Compact to dense, brown **SILTY SAND**	SS	4	71	16	3	69.76
	SS	5		15		
3.89	SS	6		50+		
End of Borehole Practical refusal to augering at 3.89m depth (GWL @ 1.93m - April 12, 2019)						

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

HOLE NO. BH 4-19

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	72.88		
FILL: Brown silty sand with gravel		AU	1						
		SS	2	83	36	1	71.88		
1.52									
Brown **SILTY SAND,** trace gravel and shale		SS	3	83	5	2	70.88		
		SS	4		20				
2.80									
BEDROCK: Weathered black shale									
3.05						3	69.88		
End of Borehole									

Pen. Resist. / Water Content scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

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REMARKS

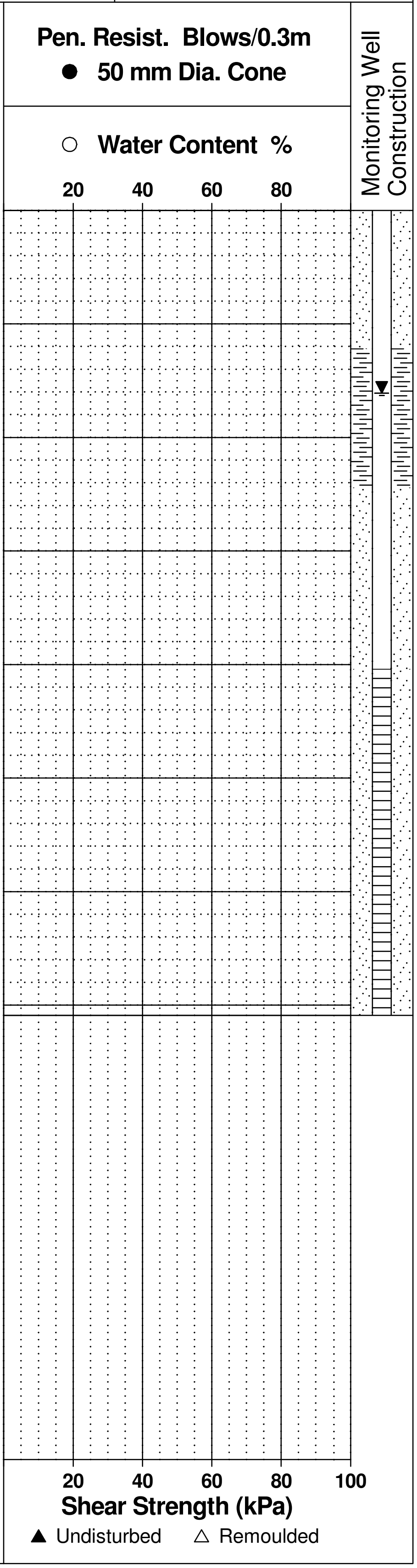
BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

HOLE NO. BH 5-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	72.22
Asphaltic concrete 0.06						
FILL: Brown silty sand with gravel, trace clay	AU	1				
	SS	2	91	50+		
					1	71.22
1.78	SS	3	80	50+		
					2	70.22
	AU	4				
					3	69.22
	RC	1	100	20		
BEDROCK: Black shale					4	68.22
	RC	2	100	44		
					5	67.22
					6	66.22
	RC	3	100	86		
7.09					7	65.22
End of Borehole						
(GWL @ 1.61m - April 12, 2019)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

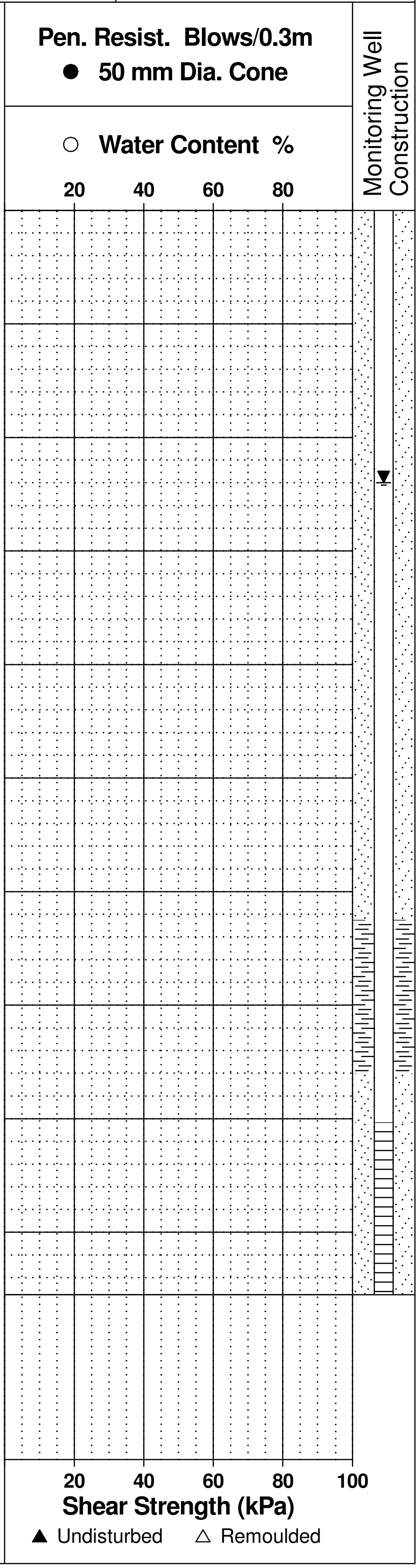
BORINGS BY CME 55 Power Auger

DATE 2019 March 26

FILE NO. PG4875

HOLE NO. BH 6-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	72.30
Asphaltic concrete 0.08	AU	1				
FILL: Brown silty sand with gravel	SS	2	38	69	1	71.30
1.83	SS	3	33	33		
					2	70.30
	AU	4				
					3	69.30
	RC	1	100	20		
					4	68.30
	RC	2	100	17		
BEDROCK: Black shale					5	67.30
					6	66.30
	RC	3	100	46		
					7	65.30
	RC	4	100	97		
					8	64.30
					9	63.30
	RC	5	100	100		
9.55						
End of Borehole						
(GWL @ 2.40m - April 12, 2019)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1090 Cummings Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of Cummings Avenue and Snow Street. Geodetic elevation = 73.86m, as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

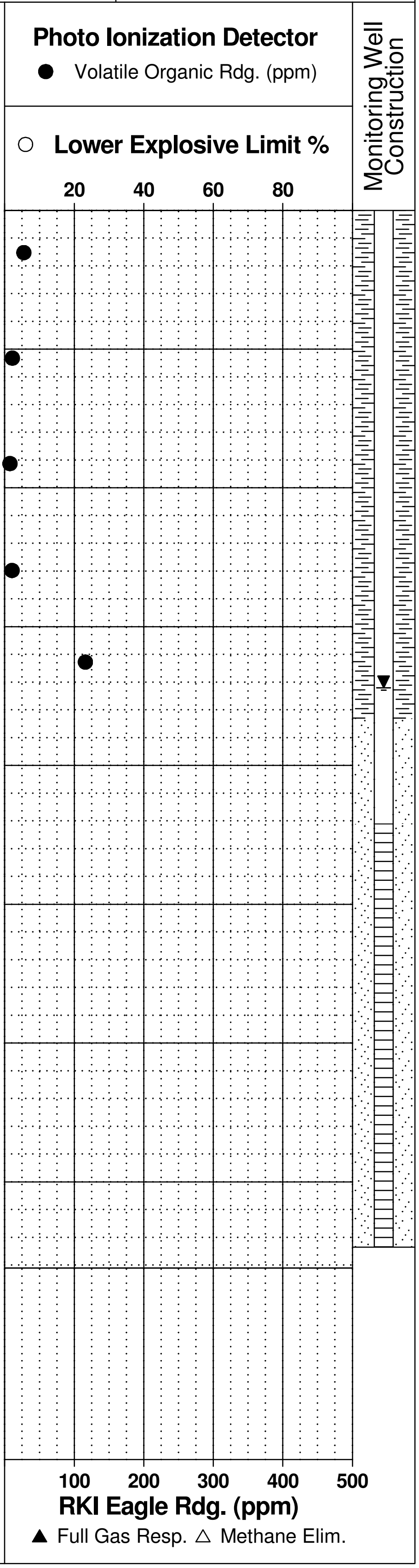
DATE 2016 September 16

FILE NO. PE3839

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Crushed stone with sand. 0.61	AU	1		
FILL: Dark brown, loose silty sand, some gravel and clay	SS	2	83	7
	SS	3	50	5
2.21 **TOPSOIL** 2.39				
Very loose, brown **SANDY SILT**	SS	4	83	3
3.33	SS	5	75	50+
BEDROCK: Poor to good quality, black shale - mud seams @ 4.65m depth	RC	1	100	30
	RC	2	100	70
	RC	3	90	85
7.62 End of Borehole (GWL @ 3.44m-Sept. 23, 2016)				

DEPTH (m)	ELEV. (m)
0	72.48
1	71.48
2	70.48
3	69.48
4	68.48
5	67.48
6	66.48
7	65.48



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1090 Cummings Avenue
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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2016 September 19

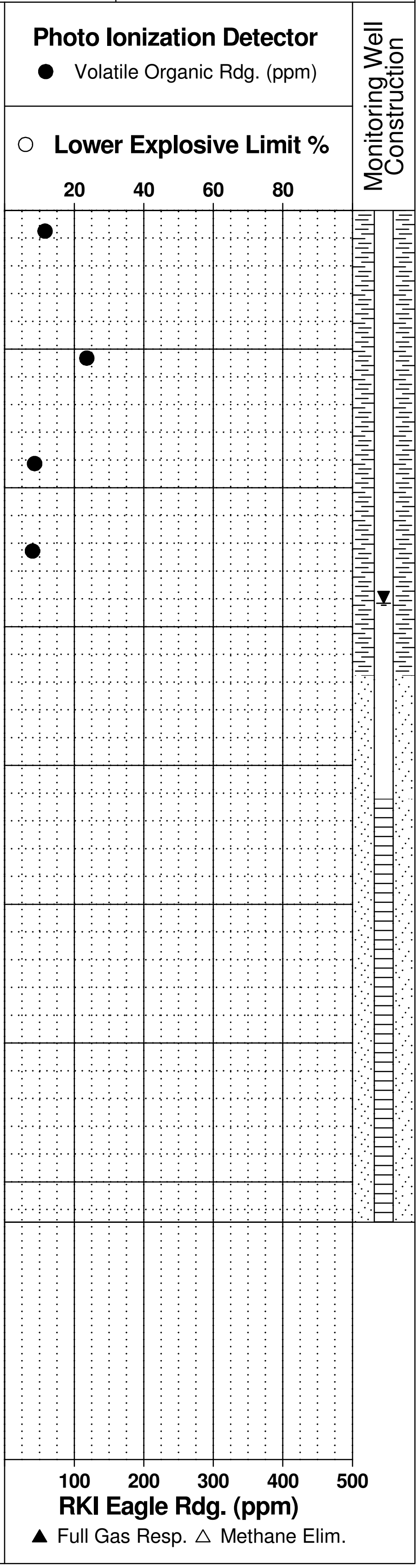
FILE NO. PE3839

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Concrete Slab	0.15	AU	1		
FILL: Brown silty sand with crushed stone, trace cobbles	0.46				
FILL: Dark brown sandy silt / silty sand, some crushed stone, gravel, asphalt, wood and metal	2.18	SS SS	2 3	92 33	39 9
BEDROCK: Very poor to excellent quality, black shale	7.29	SS RC RC RC	4 1 2 3	100 90 100 100	<50 40 33 98
End of Borehole					

(GWL @ 2.83m-Sept. 23, 2016)

DEPTH (m)	ELEV. (m)
0	72.33
1	71.33
2	70.33
3	69.33
4	68.33
5	67.33
6	66.33
7	65.33



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2016 September 16

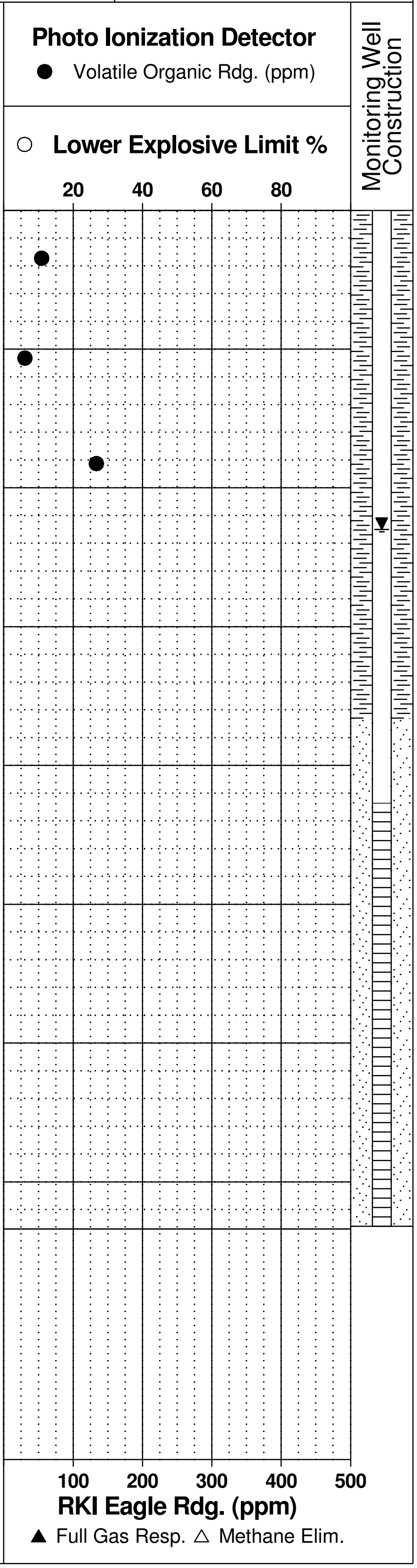
FILE NO. PE3839

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic Concrete	0.08
FILL: Crushed stone with sandy silt	0.46
FILL: Dark brown sandy silt with some gravel and organic material, trace coal	1.52
Compact, brown **SAND** with silt	1.83
BEDROCK: Very poor to excellent quality black shale	7.34
End of Borehole	
(GWL @ 2.30m-Sept. 23, 2016)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	38	19
SS	3	0	12
RC	1	85	0
RC	2	100	73
RC	3	98	87
RC	4	100	100

DEPTH (m)	ELEV. (m)
0	72.27
1	71.27
2	70.27
3	69.27
4	68.27
5	67.27
6	66.27
7	65.27



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

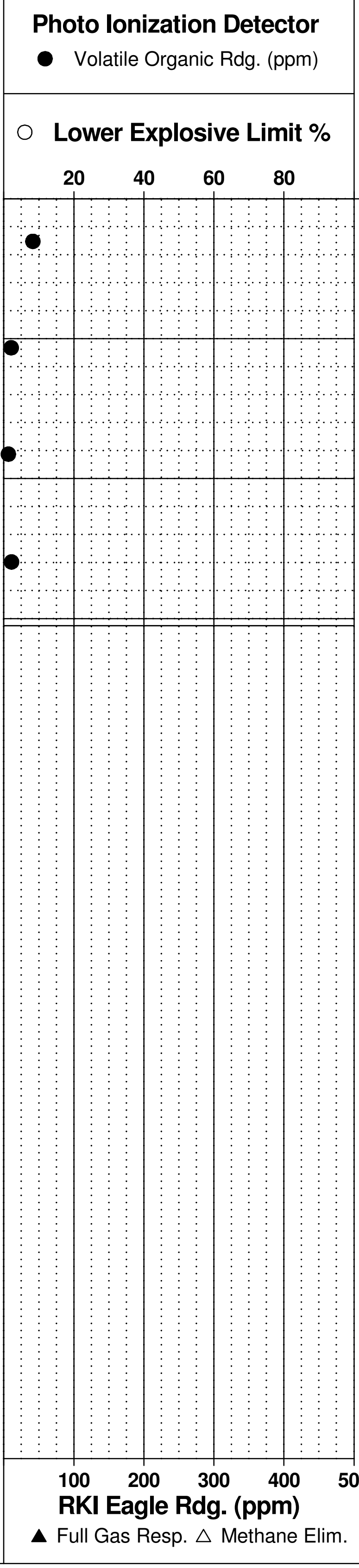
DATE 2016 September 16

FILE NO. PE3839

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown sand with some gravel, trace asphalt		SS	1	67	12
0.61					
Very loose, brown **SILTY SAND,** trace gravel		SS	2	58	4
		SS	3	38	2
2.69		SS	4	92	12
BEDROCK: Very weathered black shale					
3.05					
End of Borehole					

DEPTH (m)	ELEV. (m)
0	71.56
1	70.56
2	69.56
3	68.56



Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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REMARKS

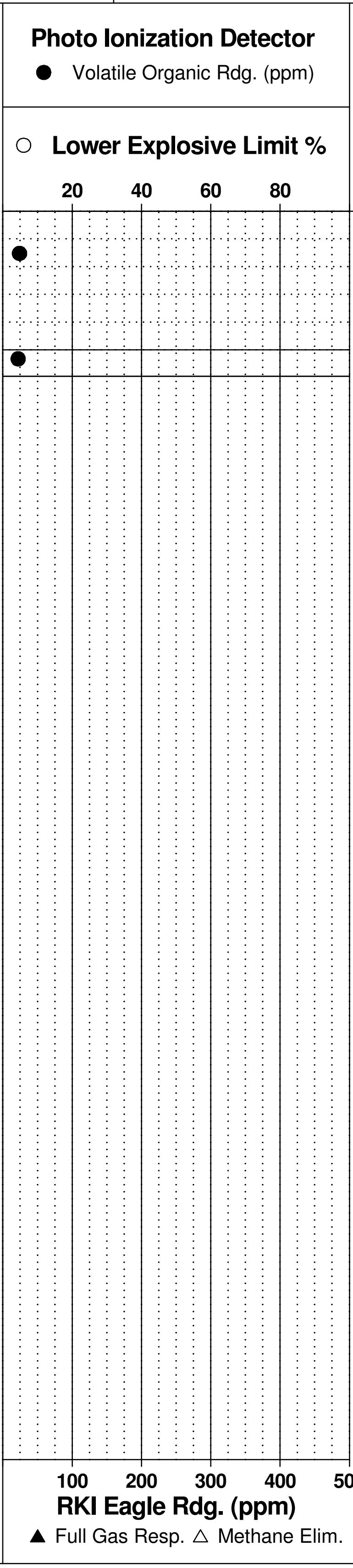
BORINGS BY CME-55 Low Clearance Drill

DATE 2016 September 16

FILE NO. PE3839

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0	71.95	
FILL: Brown silty sand, trace gravel and brick 0.56		SS	1	79	16			
Brown **SILTY SAND** trace shale 0.97								
BEDROCK: Very weathered shale 1.19		SS	2	92	29	1	70.95	
End of Borehole								



SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1090 Cummings Avenue
Ottawa, Ontario

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REMARKS

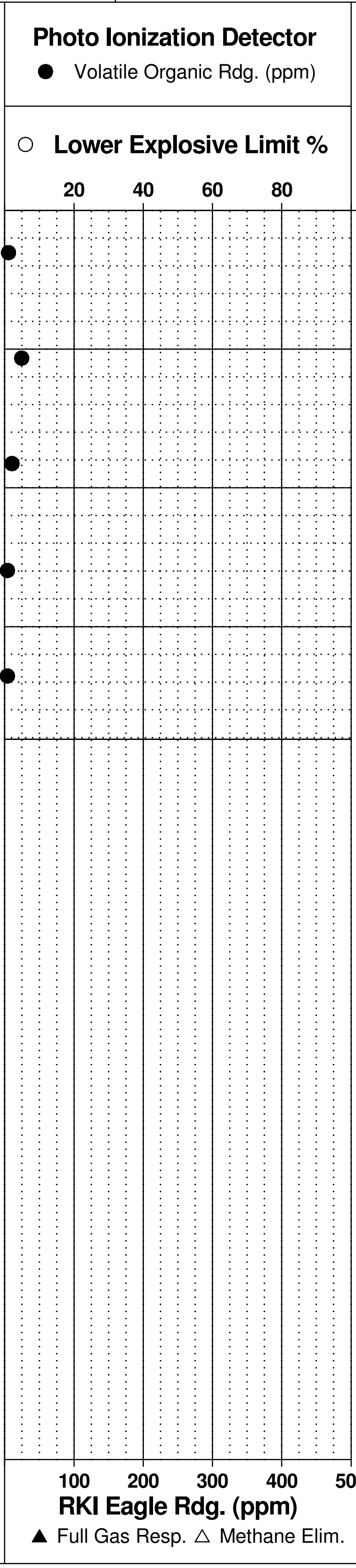
BORINGS BY CME-55 Low Clearance Drill

DATE 2016 September 16

FILE NO. PE3839

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	72.72
FILL: Compact crushed stone with sand 0.61		AU	1				
FILL: Brown silty sand with some crushed stone, trace asphaltic concrete		SS	2	63	17	1	71.72
2.03		SS	3	75	6		
TOPSOIL 2.13						2	70.72
Compact, brown **SILTY SAND** trace gravel 3.05		SS	4	83	12		
						3	69.72
Compact, brown **SILT** with sand 3.81		SS	5	71	13		
End of Borehole Practical refusal to augering at 3.81m depth							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

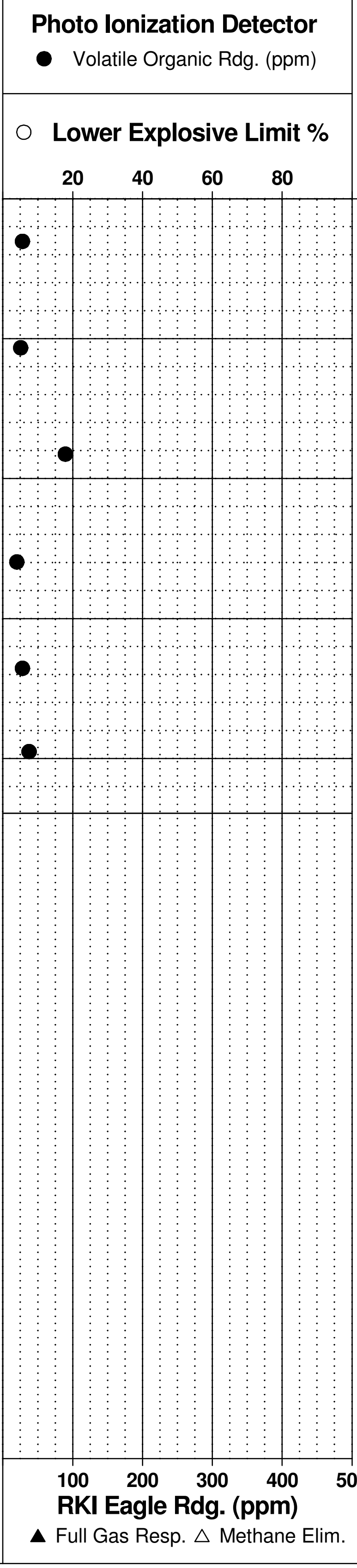
DATE 2016 September 16

FILE NO. PE3839

HOLE NO. BH 7

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	73.06
	AU	1				
FILL: Brown silty sand with crushed stone and boulders, some concrete	SS	2	33	9	1	72.06
- wood @ 1.8m depth	SS	3	50	9	2	71.06
2.49	SS	4	8	8		
					3	70.06
Loose, brown **SAND,** some silt	SS	5	71	9		
4.14	SS	6	100	<50	4	69.06
BEDROCK: Black shale 4.39						
End of Borehole						

Monitoring Well Construction



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)

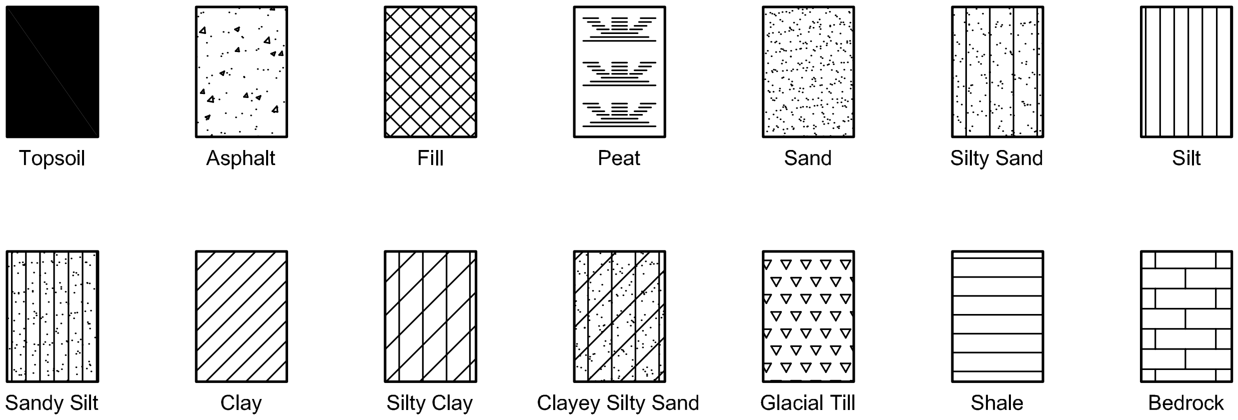
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

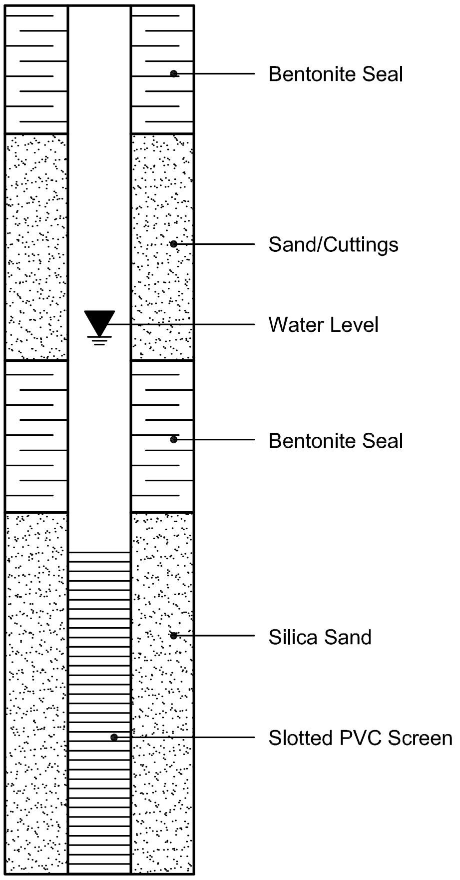
SYMBOLS AND TERMS (continued)

STRATA PLOT

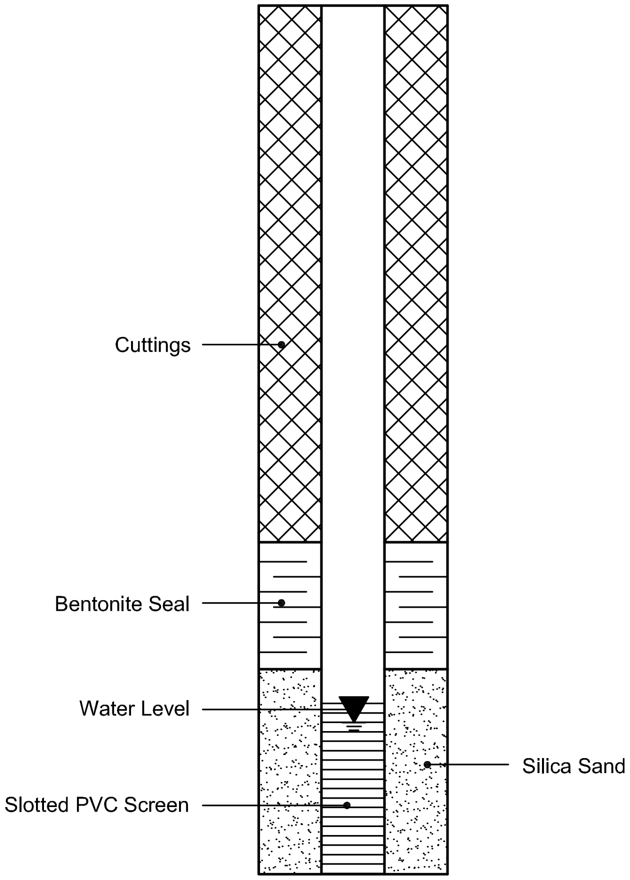


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25592

Report Date: 02-Apr-2019

Order Date: 27-Mar-2019

Project Description: PG4875

	Client ID:	BH2-19 SS4	-	-	-
	Sample Date:	03/26/2019 09:00	-	-	-
	Sample ID:	1913436-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.63	-	-	-
Resistivity	0.10 Ohm.m	7.57	-	-	-
Anions					
Chloride	5 ug/g dry	651	-	-	-
Sulphate	5 ug/g dry	180	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4875-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

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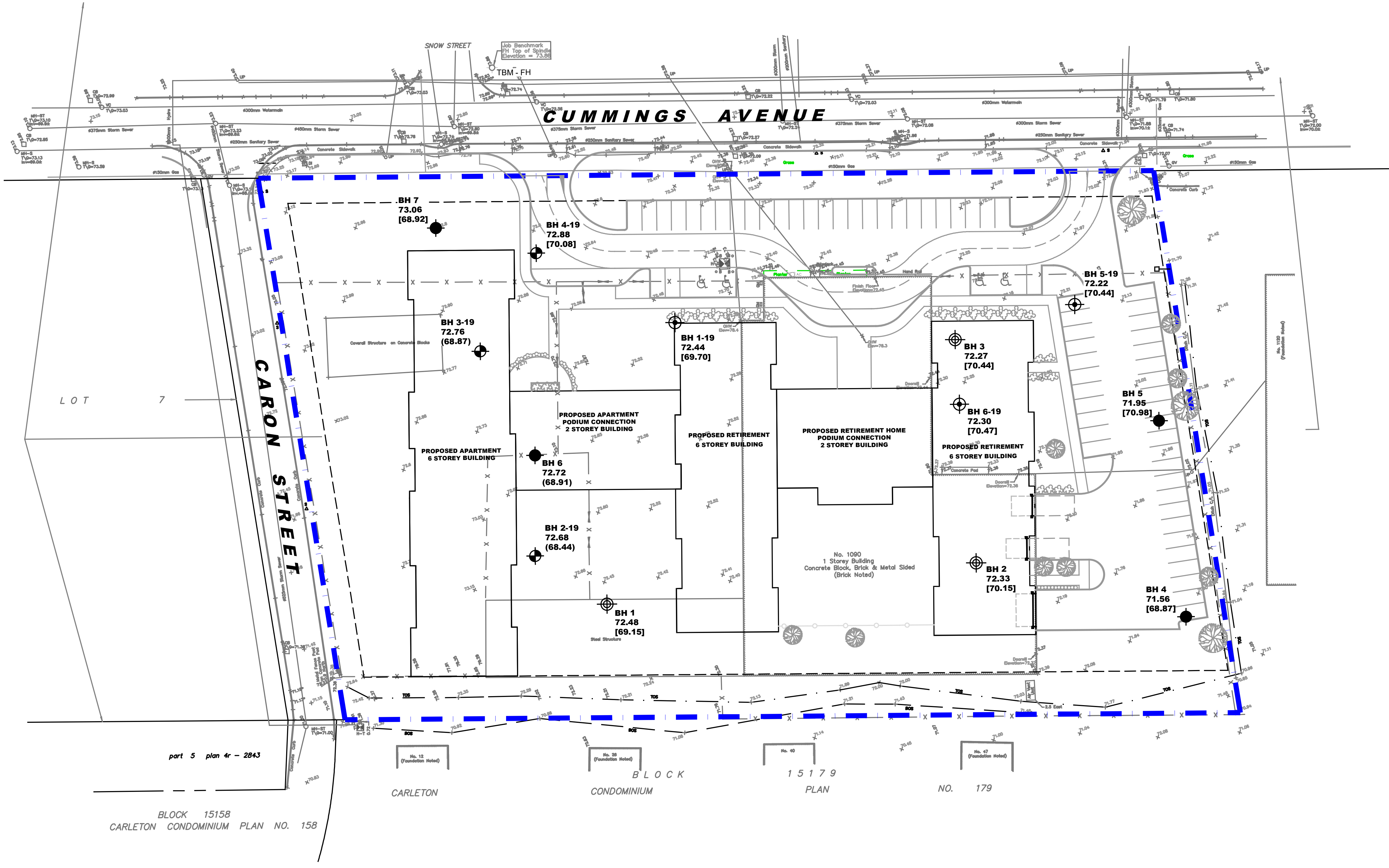

LEGEND:

◐	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION, PATERSON GROUP REPORT PE3839, 2016
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3839, 2016
98.36	GROUND SURFACE ELEVATION (m)
[96.58]	BEDROCK SURFACE ELEVATION (m)
(95.05)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT. AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

HUNTINGTON PROPERTY GROUP (ITF CUMMINGS CARON JOINT VENTURE)

GEOTECHNICAL INVESTIGATION
1068-1090 CUMMINGS AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Drawn by: YA
Checked by: JV
Approved by: DJG

Date: 05/2019
Report No.: PG4875-1

PG4875-1

Revision No.:

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