Geotechnical Desktop Review

Proposed Mixed-Use Development

1740, 1746 & 1754 Carling Avenue, 828 Boyd Avenue,
and 1755 Kerr Avenue
Ottawa, Ontario

Prepared for Kerr Broadview Properties Ltd.

Report PG7336-1 dated December 4, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Kerr Broadview Properties Ltd. to provide a geotechnical desktop review based on available subsoil information for the proposed mixed-use development to be located at 1740, 1746 & 1754 Carling Avenue, 828 Boyd Avenue, and 1755 Kerr Avenue in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2).

The objectives of the geotechnical desktop review were to:

- ☐ Determine the subsurface soil and groundwater conditions by means of existing boreholes completed by this firm.

- ☐ 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

Based on available plans and information, it is understood that the proposed development consists of three high-rise mixed-use buildings with multiple levels of underground parking.

A parkland and amenity courtyard are also proposed as part of the subject development. Associated access lanes, loading zones, at-grade parking areas, and landscaped areas are also anticipated. The buildings are expected to be serviced by municipal services.

3.0 Method of Investigations

3.1 Field Investigations

Field Programs

Previous field programs were completed by this firm between October 2013 and October 2018 in conjunction with our general knowledge of the area's geology. Previous investigations consist of 17 boreholes advanced to a maximum depth of 10.3 m below the existing ground surface (at the time of the field program). The test hole locations were distributed across the site in a manner to provide general coverage of the subject site and based on access at the time of each investigation. The locations of the test holes are shown on Drawing PG7336-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a truck and/or track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden and bedrock coring.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. Rock cores were obtained using 47.6 mm inside diameter coring equipment. Grab samples were also collected from borehole locations. All soil samples were visually inspected and initially classified on site. The auger, grab and split-spoon samples were placed in sealed plastic bags. Rock cores were placed in cardboard boxes.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger, grab, split spoon, and rock core samples were recovered from the test holes are shown as AU, G, 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.

Rock core samples were recovered from select test holes using a core barrel and diamond drilling techniques. The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented 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 test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Groundwater monitoring wells were installed in select boreholes to permit long-term groundwater measurement subsequent to the field investigation. Flexible polyethylene standpipes were installed in the remaining boreholes to permit further groundwater measurement. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site, taking into account existing site features and underground utilities. The test hole locations were measured by Paterson from existing site features and the ground surface elevation at each test hole location was surveyed by Paterson and referenced to a geodetic datum. The location of the test holes is presented on Drawing PG7336- 1 – Test Hole Location Plan in Appendix 2.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by two commercial buildings, asphalt-surfaced parking lots, loading zone areas, graveled surfaced areas, and mature trees. It is understood that the existing buildings will be demolished prior to the start of construction for the proposed development.

The site is bordered by Carling Avenue and further by residential or commercial areas to the north, by Boyd Avenue and commercial buildings to the east, and by Kerr Avenue further by residential or commercial properties to the south and by residential and commercial properties and Broadview Avenue to the west.

The ground surface at the subject site is relatively flat and at grade or slightly above grade with the surrounding properties. Reference should be made to Drawing PG7336-1 – Test Hole Location Plan in Appendix 2.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil profile encountered at the test hole locations consists of fill, compact to dense silty sand to sandy silt, silty clay to clayey silt or glacial till extending to a maximum depth of 2.3 m below the existing grade, underlain by bedrock.

The fill was generally observed to consist of crushed stone or brown silty sand with gravel and clay with the exception of a think layer of asphalt, concrete or interlock brick encountered at the ground surface. The glacial till deposit was generally observed to consist of loose brown silty sand with gravel.

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

Bedrock

Bedrock consisting of grey limestone interbedded with dolostone or shale was encountered at all boreholes. The RDQ values indicate that the bedrock consists of very poor quality within the upper 0.5 to 1.0 m of the bedrock profile, followed by excellent quality, which is consistent of observations of fractured bedrock within the upper 0.5 to 1.0 m of the bedrock profile.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location. Based on available geological mapping, the bedrock consists of interbedded limestone and dolomite of the Gull River Formation and is expected to be encountered at depths ranging from 1 to 5 m.

4.3 Groundwater

The stabilized groundwater levels measured in the piezometers and monitoring wells during the subsoil and groundwater investigations completed by this firm are presented in Table 1.

Table 1 – Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 8	99.17	5.87	93.3	October 31, 2018
BH 9	99.20	5.00	94.2	
BH 1	98.40	4.01	94.39	May 15, 2015
BH 3	99.03	1.97	97.06	July 20, 2015
BH 5	97.64	3.81	93.83	
BH 6	99.05	2.82	96.23	
BH 7	99.25	2.31	96.94	
BH 1	98.66	1.95	96.71	May 12, 2015
BH 2	99.16	1.93	97.23	
BH 3	99.25	1.86	97.39	
BH 1-14	99.27	2.38	96.89	June 5, 2014
BH 2-14	99.27	2.06	97.21	
BH 1	98.94	2.23	96.71	November 4, 2013
BH 2	99.04	2.43	96.61	

Note: The ground surface elevation at each test hole location is based on the information provided by others in the previous geotechnical investigation report.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations.

The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **2.0 to 3.0 m** below ground surface. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

It is important to note that groundwater levels may also be impacted by nearby developments and changes in natural terrains. Therefore, due to the time gap between the completion of the existing boreholes and the proposed development, it is important to complete a supplemental geotechnical investigation to confirm the groundwater levels provided herein.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed mix-use development. Based on the available information, it is understood that the proposed buildings will have multiple levels of underground parking. Therefore, it is expected that the proposed buildings will be founded on conventional footings placed on the bedrock bearing surface.

Bedrock removal is expected to be required to complete the excavation for the proposed buildings. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. All contractors should be prepared for bedrock and oversized boulder removal. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. A vibration monitoring program is recommended to be implemented and monitored by the geotechnical consultant to confirm that the controlled blasting program does not negatively impact the existing adjacent structures in the vicinity of the site.

It is anticipated that the silty clay will be completely removed during the excavation operations. Therefore, a permissible grade raise restriction will not be applicable for the proposed development. However, if a change in the proposed development is introduced where silty clay will remain in place, additional recommendations must be provided by Paterson to accommodate the presence of a compressible silty clay layer below the proposed structures.

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

5.2 Site Grading and Preparation

Stripping Depth

It is expected that all overburden materials will be excavated to the bedrock surface for the majority of the subject site to accommodate the underground parking structure.

Asphalt, topsoil, and any deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed building and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the perimeter of the proposed buildings.

Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below the final grade.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming, rock grinding, and controlled blasting will be required to remove the bedrock for the underground parking structure for the proposed buildings. In areas of weathered or fractured 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 the 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 or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding 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 near vertical sidewalls. 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 of the overburden. The 1 m horizontal ledge setback can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

Construction operations are 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 into the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction.

Overbreak in Bedrock

Sedimentary bedrock formations, such as limestone, dolomite, and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Fill Placement

If fill placement is required for grading beneath the proposed building to support the floor slab, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. The granular material should be placed in lifts no greater than 300 mm thick and compacted with suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to a minimum of 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at a minimum compacted by the heavy equipment tracks to minimize voids. If these materials are to build up the subgrade level for areas to be paved, the material should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soil are not suitable as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Re-Use of Blast Rock

If site crushing operations are being considered, the use of existing blast rock may be acceptable provided that the material is well graded to a size between 50 to 100 mm minus. Grain size distribution testing must be completed by Paterson for select samples prior to the backfilling operations.

It is important to note that a 300 mm thick layer of OPSS Granular A crushed stone should be used to top up the site crushed material directly below the basement slab. All material placement should follow the same recommendations provided above for the lift thicknesses and compaction levels.

5.3 Foundation Design

It is understood that the proposed buildings will be provided with multiple levels of underground parking. Therefore, it is expected that the buildings will be founded on a bearing medium consisting of excellent quality, clean, surface sounded bedrock.

Footings placed on a clean, weaathered bedrock surface can be designed using a bearing resistance value at SLS of **500 kPa** and a factored bearing resistance value at ULS of **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Footings placed on an excellent quality, clean, surface sounded bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5, provided the bedrock is reviewed and approved by Paterson, once exposed.

Footings placed on engineered fill approved by the geotechnical consultant can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **300 kPa**.

Engineered fill should consist of an OPSS Granular A or Granular B Type II material placed in maximum 300 mm loose lifts and compacted to a minimum 98% of its SPMDD.

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. Overbreak in bedrock located directly below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passing through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Adequate lateral support is provided to an engineered fill 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.

Settlement

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

Footings placed on an engineered fill bearing medium and designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for proposed buildings. If a higher seismic site class is required (Class A or B), a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for the foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code 2024.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2024 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the native soil or engineered fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

It is expected that the basement area for the proposed buildings will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are anticipated 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. 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.

Any soft or poor performing areas within the subgrade should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab.

In consideration of the groundwater conditions encountered at the time of the construction, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet such as the building’s sump pit, should be provided in the clear stone under the basement floor.

5.6 Basement Wall

Overburden

Where the soil is to be retained, 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 of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), 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 (Po) and the seismic component (ΔP_{AE}).
The seismic earth force (ΔP) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ 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 2024. 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 2024.

Bedrock

Below the bedrock surface, it is expected that the basement walls are to be poured against a composite drainage blanket, 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 slab, which should be designed to accommodate these pressures. A hydrostatic pressure should be added for the portion below groundwater level.

5.7 Pavement Structure

Pavement Structure Over Overburden

The following pavement structures may be considered for rigid pavement, car-only parking, and access lanes, heavy traffic parking/loading areas and fire truck lanes. The proposed pavement structures are shown in Tables 2, 3, and 4.

Table 2 – Recommended Rigid Pavement Structure – Lower Level

Thickness (mm)	Material Description
125	**Rigid Concrete Pavement** – 32 MPa concrete with air entrainment
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Either fill, OPSS Granular B Type II material placed over fill or bedrock.	

Table 3 – 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 in situ soil, fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 4 – Recommended Pavement Structure – Access Lanes, Heavy Truck Parking/Loading Areas and Fire Truck Lanes

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 in situ soil, fill, or OPSS Granular B Type I or II material placed over in situ soil or 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 100% of the material's SPMDD using suitable compaction equipment. This may require the use of a geotextile, thicker subbase, or other measures that can be recommended at the time of construction as part of the field observation program.

Pavement Structure Over Podium Deck Area

If a podium deck is proposed for the proposed buildings, additional recommendations will be provided for pavement structures above the podium deck upon completing a geotechnical investigation for the proposed buildings.

6.0 Design and Construction Precautions

6.1 Foundation Drainage, Waterproofing and Backfill

Foundation Drainage and Waterproofing

It is recommended that a perimeter foundation drainage system be provided for the below-grade areas and the proposed structures. The system, where required, should consist of a 150 mm diameter, geotextile-wrapped, perforated, and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The clear crushed stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the sump pump pit or storm sewer or ditch.

A composite drainage board and waterproofing membrane should extend on the vertical foundation wall and extend to the bottom of the footings draining water towards the perimeter drainage system. The composite drainage system should consist of Delta Terraxx or equivalent. The foundation wall concrete should be properly prepared to receive a waterproofing membrane such as Colphene BSW or equivalent.

Underfloor Drainage

An underfloor drainage system will be required to redirect water from the building’s foundation drainage system to the building’s sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

Foundation Backfill

Where applicable, 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 Terraxx, connected to the perimeter foundation drainage system.

Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Finalized Drainage and Waterproofing Design

Paterson can provide a more detailed drainage and waterproofing design once the geotechnical investigation is completed. Furthermore, Paterson should be provided with the finalized architectural, civil, mechanical and structural drawings for the proposed buildings to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double-sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

Exterior unheated foundations (such as those for isolated exterior piers, parking ramps, etc.) are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes and Temporary Shoring

Unsupported Side Slopes

Excavation side slopes in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. Excavations in the soil and fill overburden above the groundwater level, extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter.

A flatter slope is required for excavation below groundwater level, if applicable. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation side slopes in sound bedrock can be carried our using almost vertical side walls. 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.

Excavation side slopes carried out for the buildings footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction.

It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements should be designed by Paterson, or another engineering firm specializing in shoring design. The shoring will depend on the proposed excavation footprint with respect to the property lines, the proximity of the adjacent structures and the bedrock depth.

The shoring designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes during construction to the approved shoring design should be reported immediately to the owner’s consultants prior to implementation.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided on the following page is to assist the contractor in completing a safe shoring system.

For preliminary design purposes, the temporary system may consist of a 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.

Generally, the shoring systems is provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. If consideration is given to utilizing a raker style support for the shoring system, the structural engineer should ensure that the design selected minimizes lateral movements to tolerable levels.

Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 5.

Table 5 – Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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 Lagging System

The 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 $0.65 \cdot K \cdot \gamma \cdot H$ 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 undrained 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 for the full height, with no hydrostatic groundwater pressure component.

A minimum factor of safety of 1.5 should be used.

Once detailed engineering drawings and excavation plans are available, Paterson should review the drawings and plans to determine the shoring requirements, as well as potential requirements for underpinning the adjacent structures.

6.4 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.

Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface.

The contractor should be prepared to direct water away from all 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 Persons 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 Structures

It is understood that two underground parking levels are being planned for the proposed building with the lower portion of the foundation having a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place, long-term groundwater lowering is anticipated to be negligible for the area. Furthermore, the proposed development will be founded on the bedrock surface and will not impact the lateral support zones of the adjacent structures. Therefore, no adverse effects to the neighboring properties are to be expected.

6.5 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.

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.

6.6 Protection of Potential Expansive Bedrock

Expansive shale may be encountered at the subject site, which introduces the potential for heaving and rapid deterioration of the shale bedrock.

If encountered, the following measure should be undertaken. To reduce long-term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. Therefore, a 50 mm thick concrete mudslab, consisting of 15 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48-hour period of being exposed.

Once the bedrock is exposed, Paterson should be notified immediately to assess the bedrock face conditions to determine the susceptibility of the bedrock to expansion. The bedrock should also be periodically inspected by Paterson as the excavation operations progress to further assess for expansive bedrock at depth, and for validation and documentation purposes.

6.7 Highly Weathered Bedrock

Due to the presence of highly weathered bedrock along the upper 0.5 to 1 m of the bedrock surface, Paterson should complete periodic inspections of the bedrock conditions once exposed. These areas are anticipated to require bedrock stabilization measures such as shotcrete, chain link fencing in conjunction with rock anchors, and/or full shoring system.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and/or detailed design of the proposed development have been prepared:

- ☐ Completion of a supplemental geotechnical field investigation specific to the proposed development, including bedrock coring.
- ☐ Review detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction.
- ☐ Review of architectural, civil, mechanical and structural plans pertaining to foundation drainage and waterproofing systems, if not designed by Paterson.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ☐ Review and inspection of the installation of the foundation drainage and waterproofing systems.
- ☐ Observation of the bedrock surface at the time of excavation to identify areas of highly weathered bedrock.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ 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 and follow-up field density tests to determine the level of compaction achieved.
- ☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils 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 that we be notified 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 Kerr Broadview Properties Ltd. or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Dec. 4, 2024
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Owen R. Canton, B.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

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- ☐ Paterson Group (1 copy)

APPENDIX 1

EXISTING SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1796 Carling and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 22, 2018

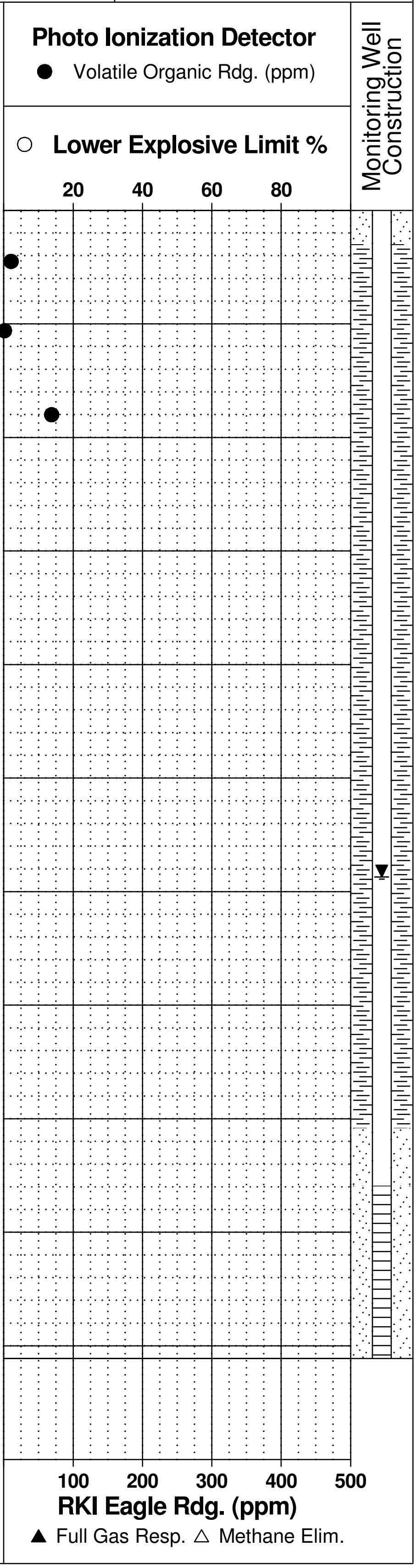
FILE NO. PE4425

HOLE NO. BH 8

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Concrete	0.13
FILL: Brown silty sand	0.60
FILL: Brown silty sand, some gravel, trace clay	1.52
Dense, brown **SANDY SILT,** trace clay	2.08
BEDROCK: Grey limestone interbedded with shale	10.11
End of Borehole	
(GWL @ 5.87m - Oct. 31, 2018)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	25	9
SS	3	73	46
RC	1	92	48
RC	2	100	98
RC	3	100	100
RC	4	100	100
RC	5	100	93
RC	6	100	100

DEPTH (m)	ELEV. (m)
0	99.17
1	98.17
2	97.17
3	96.17
4	95.17
5	94.17
6	93.17
7	92.17
8	91.17
9	90.17
10	89.17



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1796 Carling and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY CME 55 Power Auger

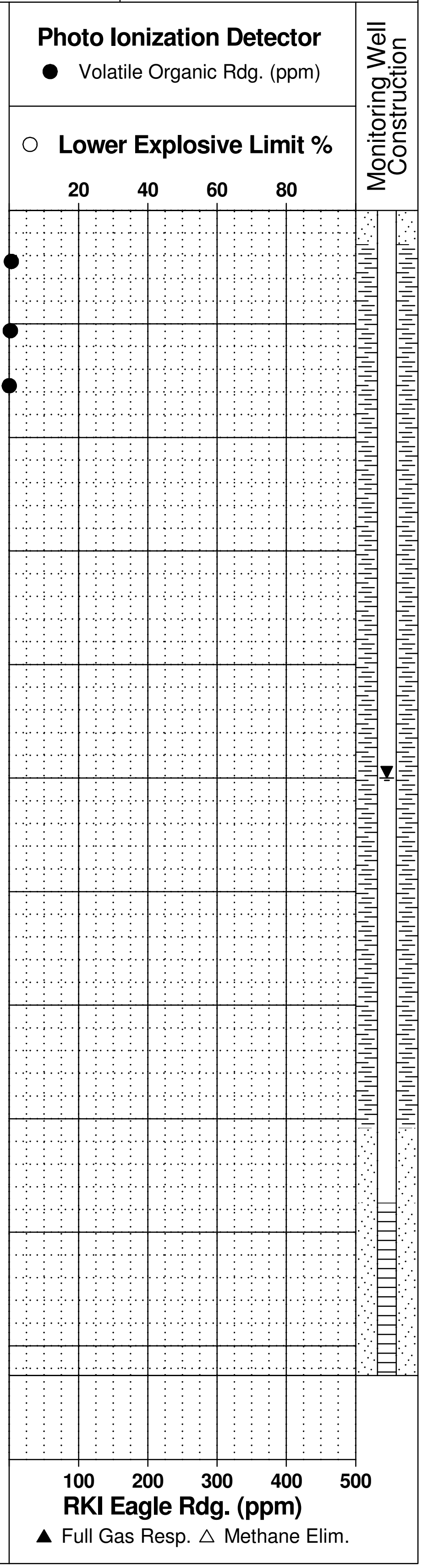
DATE October 22, 2018

FILE NO. PE4425

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel	AU	1		
	AU	2		
1.37				
Dense, brown **SILTY SAND,** some gravel	SS	3	100	50+
1.58				
BEDROCK: Grey limestone interbedded with shale	RC	1	100	81
	RC	2	100	90
	RC	3	100	100
	RC	4	100	100
	RC	5	100	100
	RC	6	100	96
10.26				
End of Borehole				
(GWL @ 5.00m - Oct. 31, 2018)				

DEPTH (m)	ELEV. (m)
0	99.20
1	98.20
2	97.20
3	96.20
4	95.20
5	94.20
6	93.20
7	92.20
8	91.20
9	90.20
10	89.20



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 8, 2015

FILE NO. PE3533

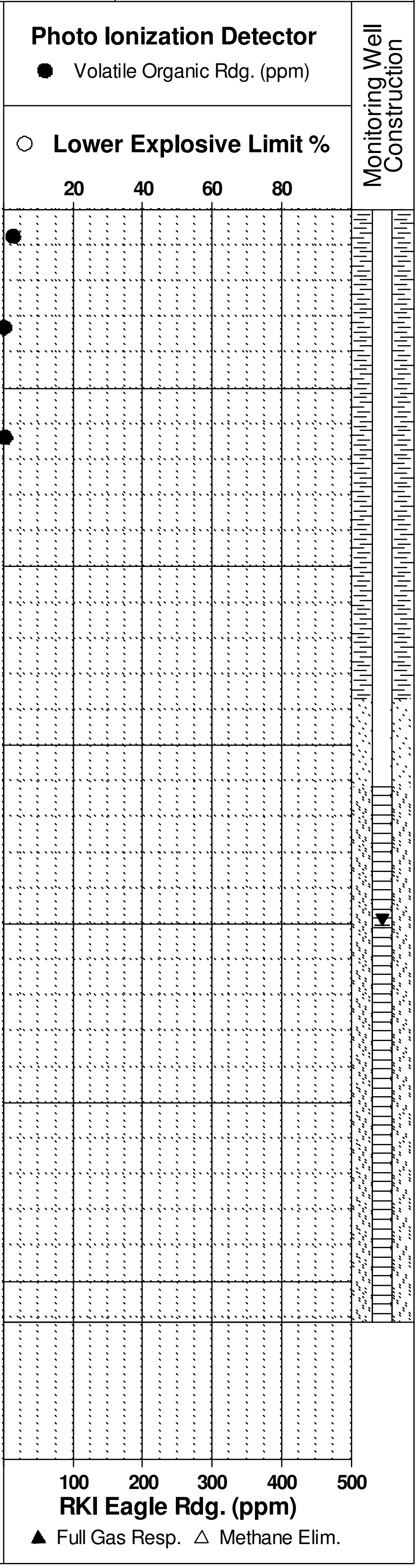
HOLE NO. BH 1

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08	AU	1		
FILL: Crushed stone	0.23				
Brown **SANDY SILT**	0.61				
Fractured **BEDROCK**	1.22	SS	2	0	+ 50
BEDROCK: Interbedded limestone and dolostone	6.23	SS	3	0	+ 50
		RC	1	78	87
		RC	2	95	84
		RC	3	100	100
		RC	4	100	97

End of Borehole

(GWL @ 4.01m-May 15, 2015)

DEPTH (m)	ELEV. (m)
0	98.40
1	97.40
2	96.40
3	95.40
4	94.40
5	93.40
6	92.40



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

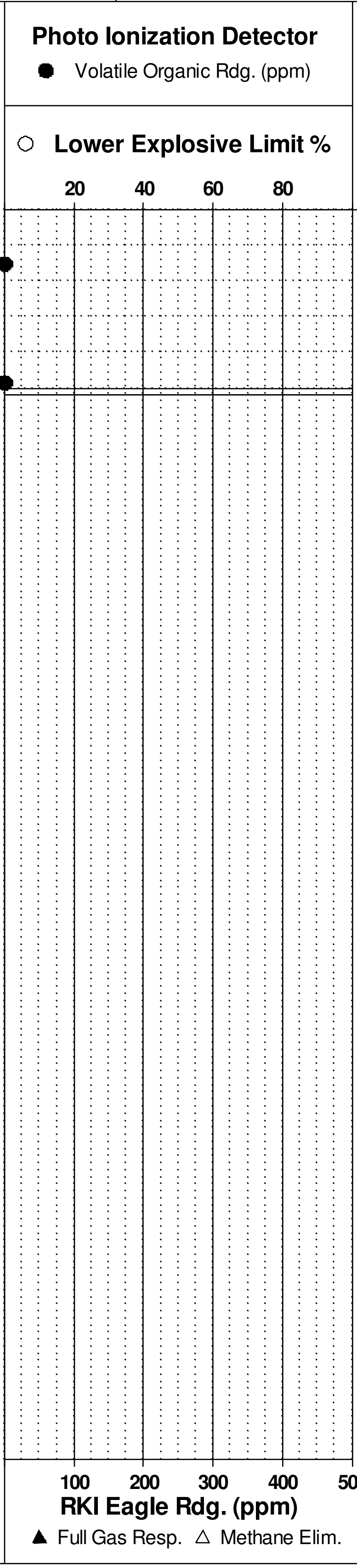
BORINGS BY CME 55 Power Auger

DATE May 8, 2015

FILE NO. PE3533

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.41
Asphaltic concrete 0.05							
FILL: Crushed stone 0.15							
Brown **SANDY SILT** 0.84		AU	1				
Fractured **BEDROCK** 1.04		SS	2	0	+ 50	1	97.41
End of Borehole Practical refusal to augering at 1.04m depth							



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 8, 2015

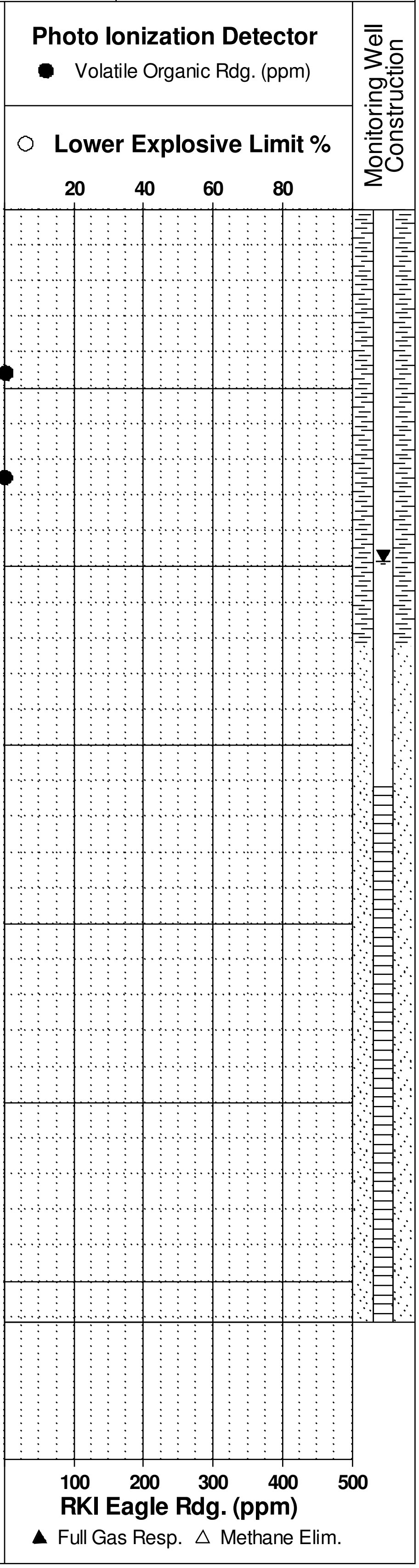
FILE NO. PE3533

HOLE NO. BH 3

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Crushed stone	0.20
Brown **SANDY SILT**	1.62
Fractured **BEDROCK**	1.72
BEDROCK: Interbedded limestone and dolostone	6.23
End of Borehole	
(GWL @ 1.97m-July 20, 2015)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	38	7
SS	2	38	28
RC	1	98	85
RC	2	100	100
RC	3	100	98

DEPTH (m)	ELEV. (m)
0	99.03
1	98.03
2	97.03
3	96.03
4	95.03
5	94.03
6	93.03



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

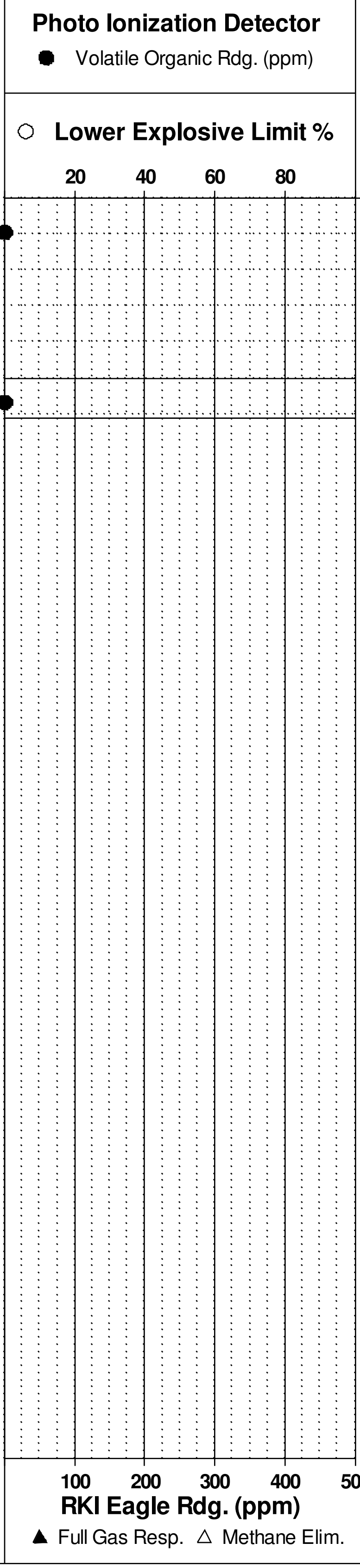
BORINGS BY CME 55 Power Auger

DATE May 8, 2015

FILE NO. PE3533

HOLE NO. BH 4

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	99.44	
Asphaltic concrete 0.08								
FILL: Crushed stone 0.23		AU	1					
Brown **SANDY SILT** 1.07						1	98.44	
Fractured **BEDROCK** 1.22		SS	2	29	10			
End of Borehole Practical refusal to augering at 1.22m depth								



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY Portable Drill

DATE July 13, 2015

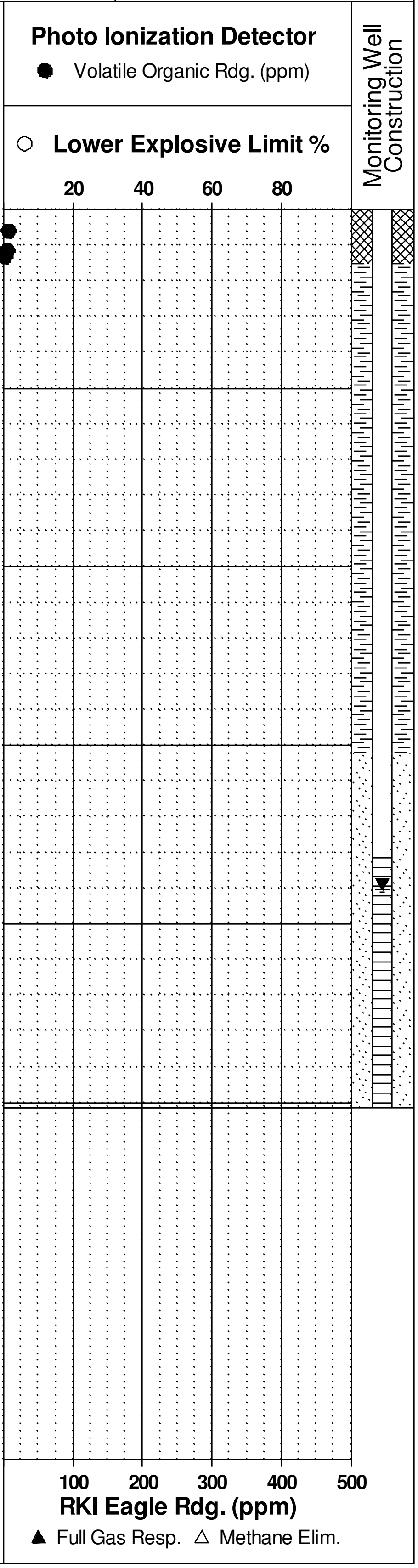
FILE NO. PE3533

HOLE NO. BH 5

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Rock dust and gravel	0.15
FILL: Brown silty sand with gravel	0.23
FILL: Cobbles	0.30
FILL: Mixture of limestone, shale, gravel and cobbles	2.23
BEDROCK: Grey limestone interbedded with shale	5.03
End of Borehole (GWL @ 3.81m-July 20, 2015)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GR	1		
GR	2	100	
RC	1		
RC	2	21	
RC	3	40	
RC	4	67	
RC	5	100	
RC	6	100	
RC	7	100	

DEPTH (m)	ELEV. (m)
0	97.64
1	96.64
2	95.64
3	94.64
4	93.64
5	92.64



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

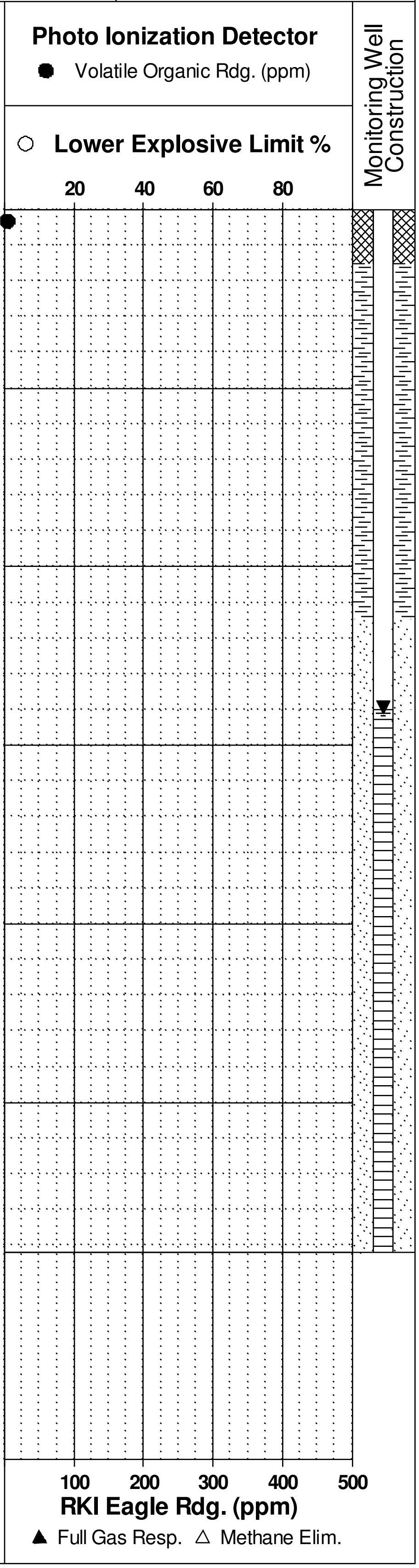
BORINGS BY Portable Drill

DATE July 14, 2015

FILE NO. PE3533

HOLE NO. BH 6

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.05
Asphaltic concrete	0.04	GR	1				
FILL: Crushed stone	0.09	GR	2				
FILL: Brown silty sand with gravel	0.25	RC	1	60			
FILL: Cobbles	0.51						
Fractured **BEDROCK:**	0.94	RC	2	80			
						1	98.05
		RC	3	85			
						2	97.05
		RC	4	100			
						3	96.05
		RC	5	95			
BEDROCK: Grey limestone interbedded with shale							
						4	95.05
		RC	6	100			
						5	94.05
		RC	7	100			
	5.84						
End of Borehole							
(GWL @ 2.82m-July 20, 2015)							



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1755 Kerr Avenue
Ottawa, Ontario

DATUM TBM - Painted pin located near the northeast corner of 1755 Kerr Avenue. Temporary elevation = 99.228m.

REMARKS

BORINGS BY Portable Drill

DATE July 14, 2015

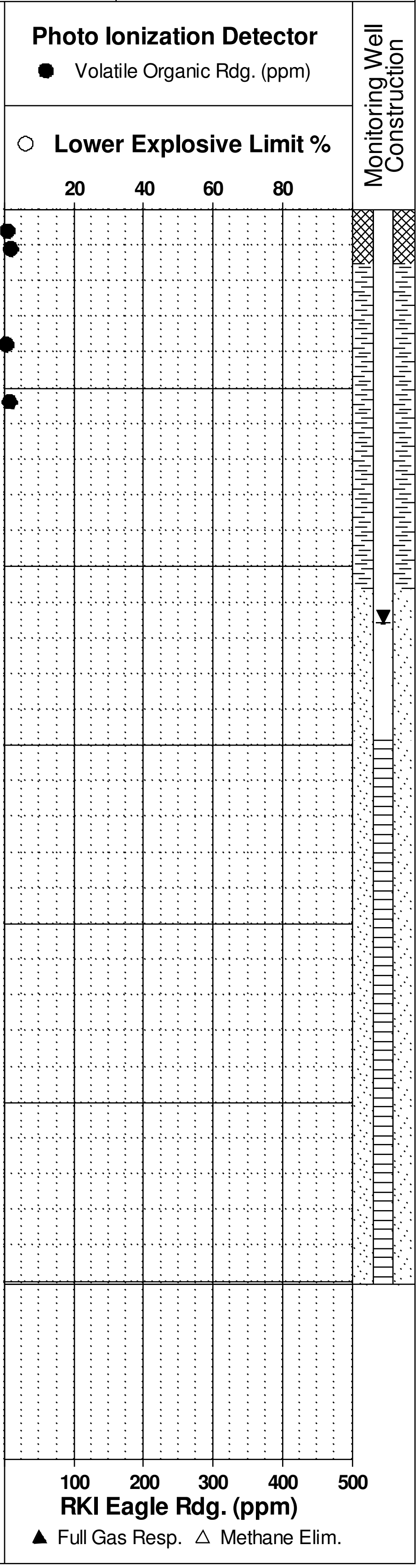
FILE NO. PE3533

HOLE NO. BH 7

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Gravel	0.18
FILL: Brown silty sand, occasional gravel	0.30
Compact, dark brown to brown **SANDY SILT**	0.91
BEDROCK: Grey limestone interbedded with shale	6.02
End of Borehole (GWL @ 2.31m-July 20, 2015)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GR	1		
GR	2		
SS	3	100	
SS	4	77	
RC	1	93	
RC	2		
RC	3	100	
RC	4	100	

DEPTH (m)	ELEV. (m)
0	99.25
1	98.25
2	97.25
3	96.25
4	95.25
5	94.25
6	93.25



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1746 Carling Avenue and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south-west corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

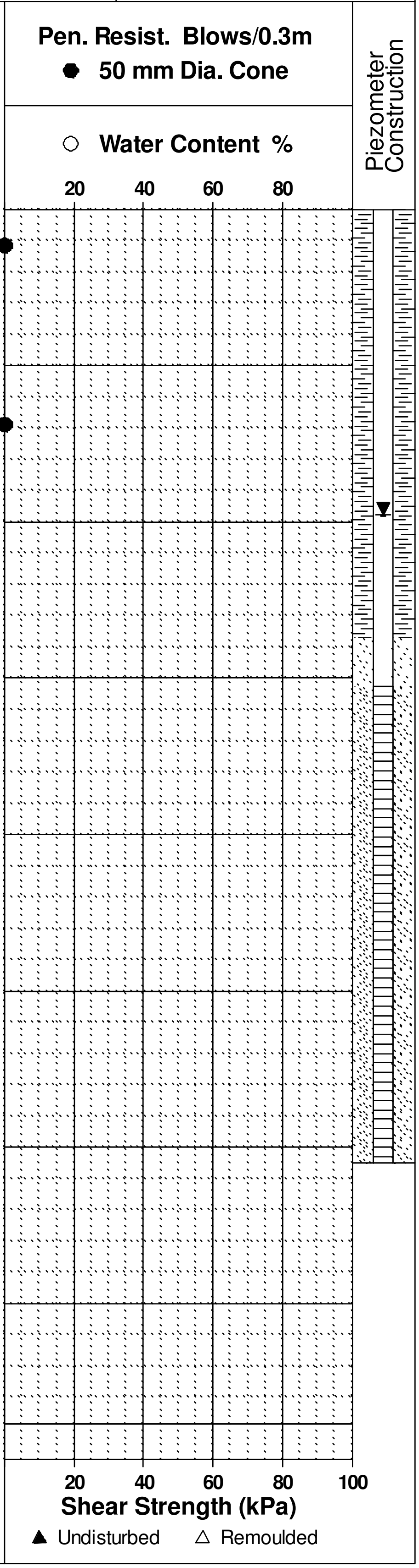
BORINGS BY CME 55 Power Auger

DATE May 6, 2015

FILE NO. PE3540

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	98.66
FILL: Granulars with brown silty sand	AU	1				
0.46						
Brown **CLAYEY SILT** with sand					1	97.66
	SS	2	42			
1.65						
Grey **LIMESTONE** with shale					2	96.66
	RC	1	81	100		
					3	95.66
	RC	2	100	100		
					4	94.66
	RC	3	100	100		
					5	93.66
	RC	4	100	100		
					6	92.66
6.25						
End of Borehole						
(GWL @ 1.95m May 12, 2015)						
					7	91.66



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1746 Carling Avenue and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south-west corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

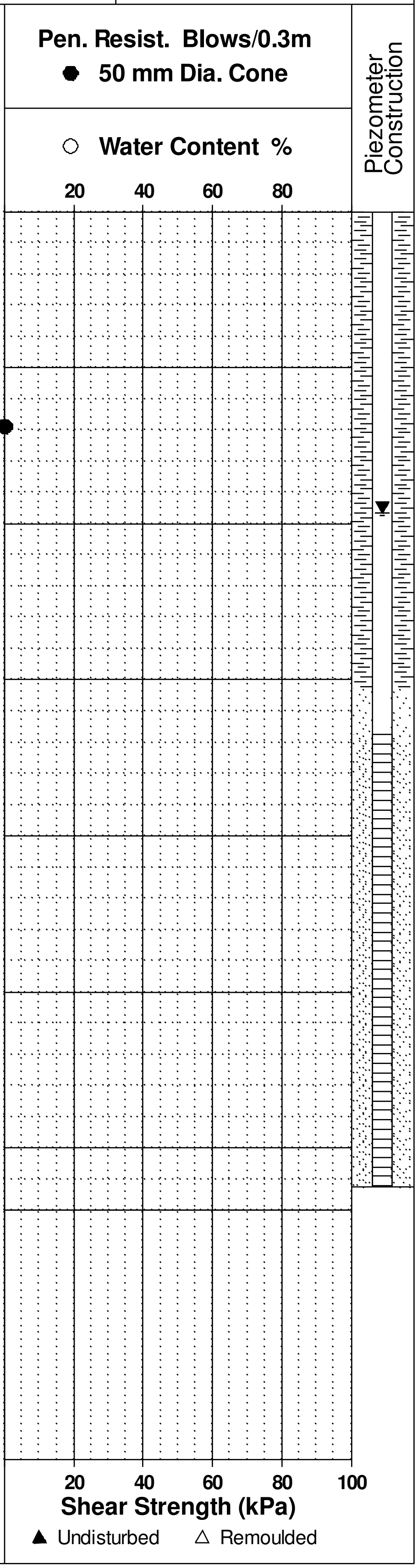
BORINGS BY CME 55 Power Auger

DATE May 6, 2015

FILE NO. PE3540

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.16
FILL: Granulars with trace concrete						
0.36						
Brown **CLAYEY SILT** and gravel					1	98.16
	SS	1	42	11		
1.55						
Grey **LIMESTONE** with quartz seams	RC	1	83	23	2	97.16
					3	96.16
	RC	2	93	70		
					4	95.16
	RC	3	67	67		
					5	94.16
	RC	4	120	120	6	93.16
6.40						
End of Borehole						
GWL @ 1.93m May 12, 2015)						



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1746 Carling Avenue and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south-west corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 6, 2015

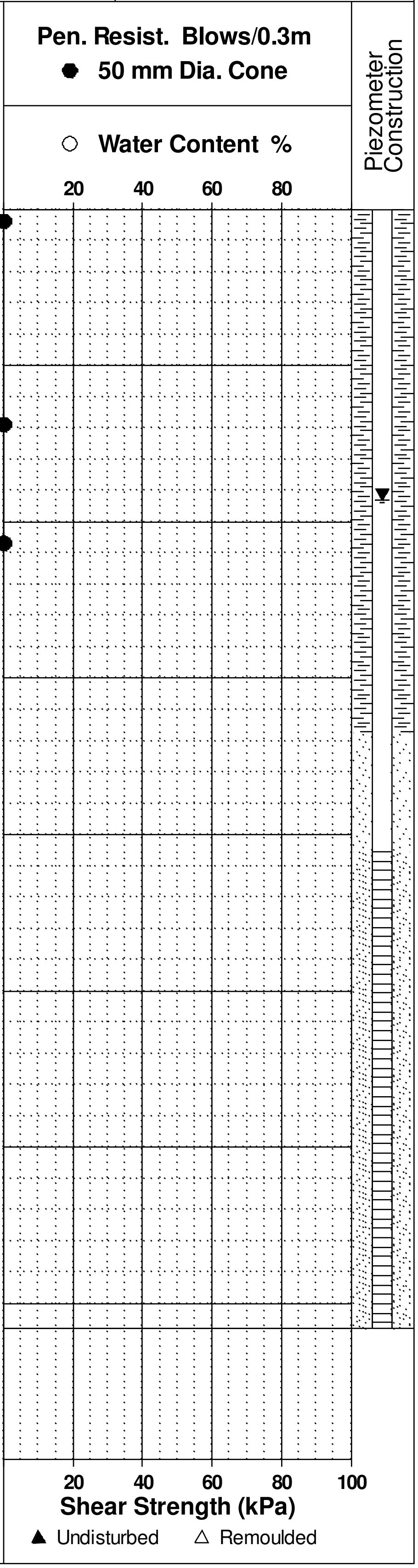
FILE NO. PE3540

HOLE NO. BH 3

SOIL DESCRIPTION	
GROUND SURFACE	
CONCRETE	0.15
FILL: Brown silty sand	0.61
FILL: Brown silt, sand, with clay, gravel and possible topsoil	1.52
Brown **CLAYEY SILT** with weathered bedrock	2.33
Grey **LIMESTONE** with shale beds	7.16
End of Borehole	
GWL @ 1.86m May 12, 2015)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	8
SS	3	58	33
RC	1	100	0
RC	2	93	87
RC	3	100	100
RC	4	100	100

DEPTH (m)	ELEV. (m)
0	99.25
1	98.25
2	97.25
3	96.25
4	95.25
5	94.25
6	93.25
7	92.25



patersongroup

Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1746 Carling Avenue and 828 Boyd Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south-west corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 6, 2015

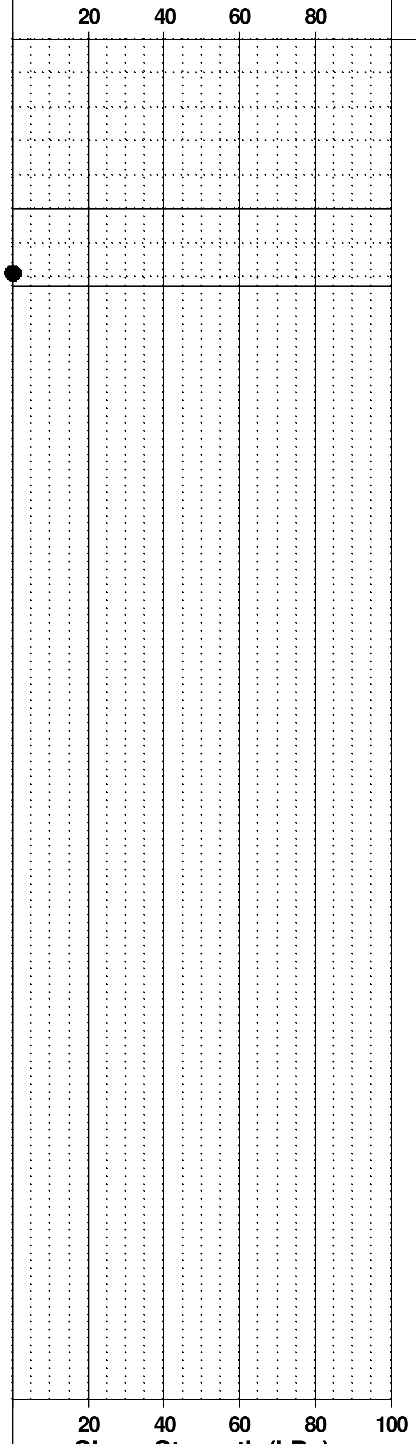
FILE NO. PE3540

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0	99.22	
CONCRETE 0.15								
FILL: Granulars 0.25								
FILL: Brown silt, sand, with clay and trace organic matter						1	98.22	
1.45		SS	1	25	+ 50			
End of Borehole Auger refusal @ 1.45m on inferred bedrock								

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

Phase II - Environmental Site Assessment
1740 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

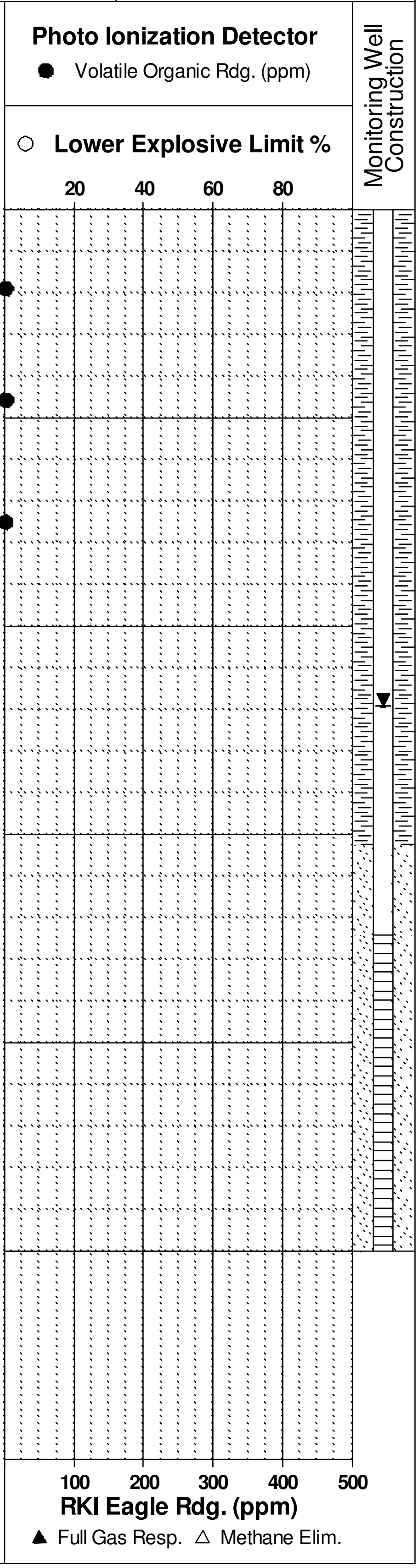
BORINGS BY CME 55 Power Auger

DATE March 24, 2014

FILE NO. PE3241

HOLE NO. BH 1-14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.27
Concrete 0.13							
FILL: Crushed stone 0.18							
		SS	1	56			
FILL: Brown silty clay with gravel, occasional mortar							
		SS	2	50			
1.22						1	98.27
FILL: Brown silty sand, some gravel and clay		SS	3				
1.78						2	97.27
		RC	1	88	71		
						3	96.27
BEDROCK: Grey limestone		RC	2	100	97		
						4	95.27
		RC	3	100	100		
5.00						5	94.27
End of Borehole							
(GWL @ 2.38-June 5, 2014)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1740 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Boyd Avenue and Carling Avenue. Assumed elevation = 100.00m.

REMARKS

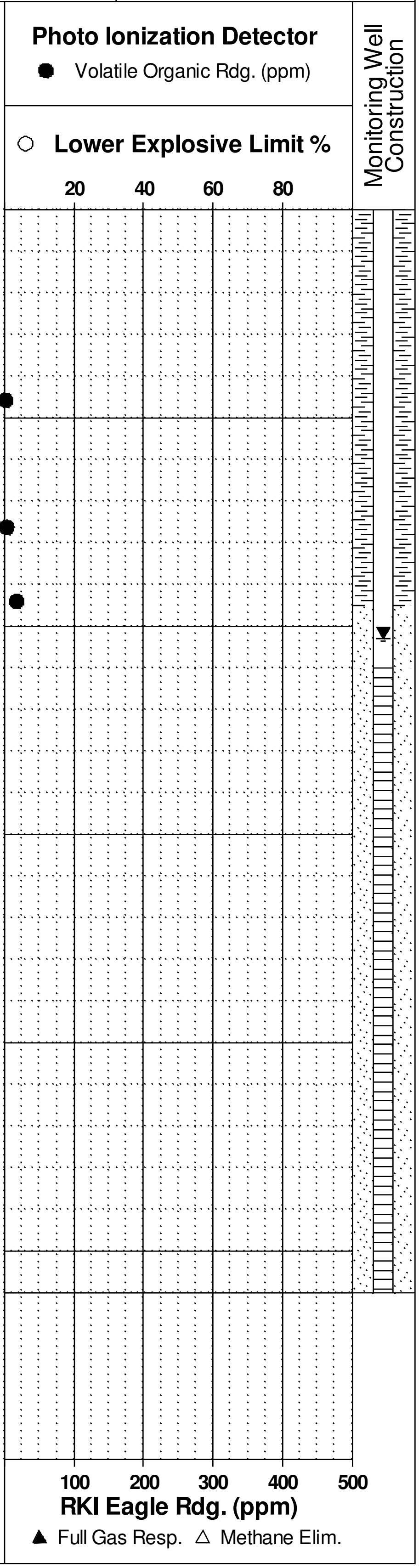
BORINGS BY CME 55 Power Auger

DATE March 24, 2014

FILE NO. PE3241

HOLE NO. BH 2-14

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.27
Concrete	0.13						
FILL: Brown silty sand with gravel	1.22	SS	1	83		1	98.27
FILL: Brown silty clay with gravel and organics	1.83	SS	2	83			
FILL: Weathered limestone	1.93	SS	3	100		2	97.27
BEDROCK: Grey limestone		RC	1	81	68	3	96.27
		RC	2	100	100	4	95.27
	5.20	RC	3	100	100	5	94.27
End of Borehole							
(GWL @ 2.06-June 5, 2014)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1740 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Assumed elevation = 100.00m.

REMARKS

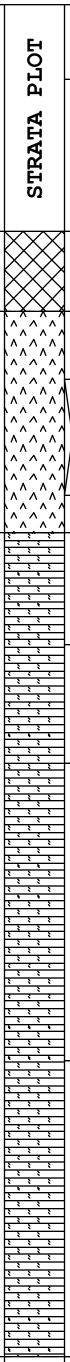
BORINGS BY CME 55 Power Auger

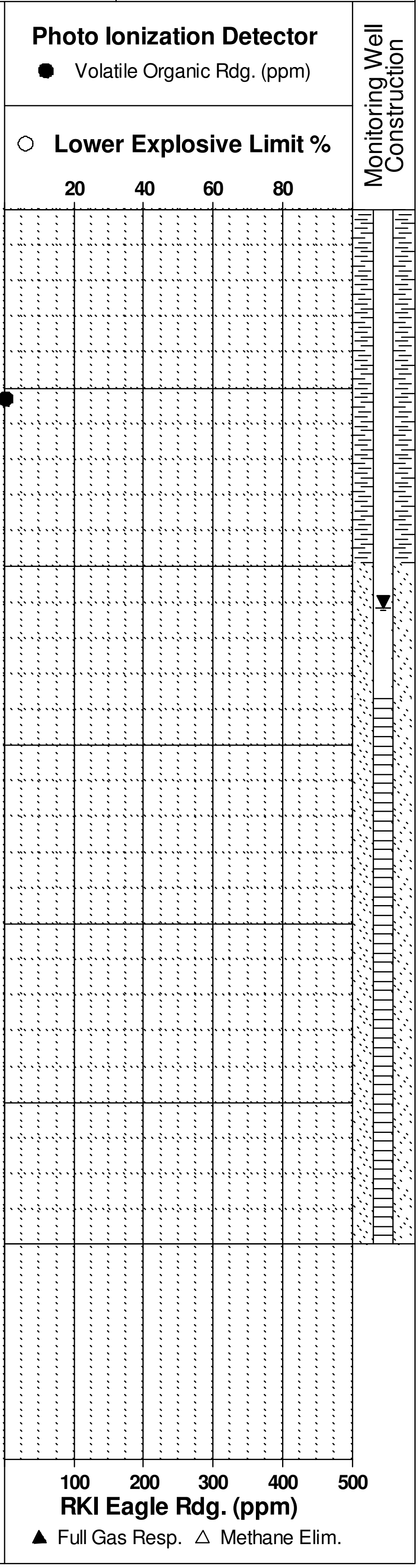
DATE October 30, 2013

FILE NO. PE3133

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.94
FILL: Interlock brick over crushed stone with gravel 0.41						
GLACIAL TILL: Brown silty sand with gravel 1.55	SS	1	42	8	1	97.94
BEDROCK: Grey limestone					2	96.94
	RC	2	71	46		
					3	95.94
	RC	3	100	42		
					4	94.94
	RC	4	100	85	5	93.94
5.79 End of Borehole (GWL @ 2.23m-Nov. 4, 2013)						





SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1740 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Assumed elevation = 100.00m.

REMARKS

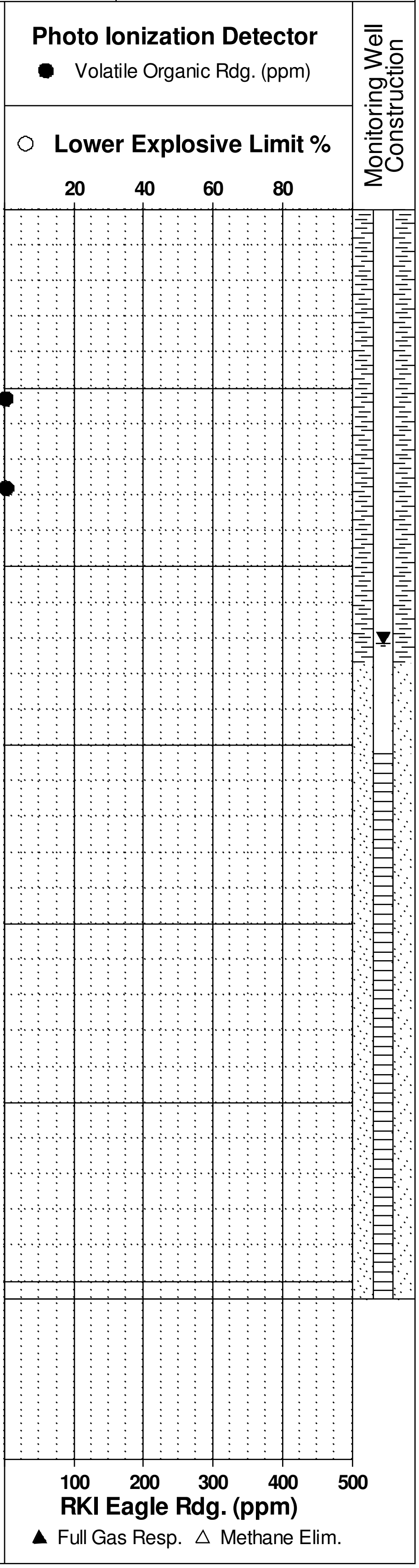
BORINGS BY CME 55 Power Auger

DATE October 30, 2013

FILE NO. PE3133

HOLE NO. BH 2

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.04
Asphaltic concrete	0.08						
FILL: Crushed stone	0.25						
FILL: Brown silty clay, trace sand and gravel		SS	1	42	6	1	98.04
	1.70	SS	2	33	50+		
BEDROCK: Grey limestone						2	97.04
						3	96.04
						4	95.04
		RC	1	100	67		
						5	94.04
		RC	2	100	100		
	6.10					6	93.04
End of Borehole							
(GWL @ 2.43m-Nov. 4, 2013)							



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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 = $(D30)^2 / (D10 \times D60)$
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)

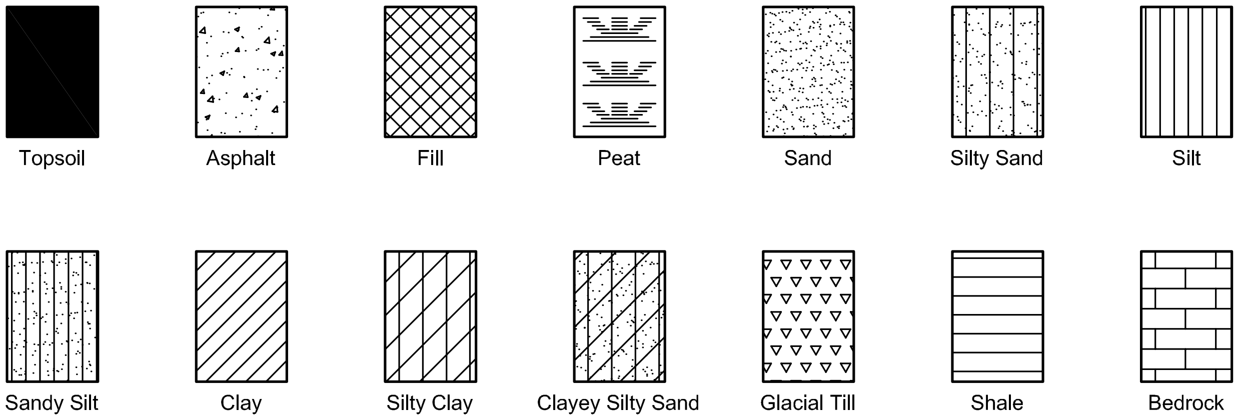
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

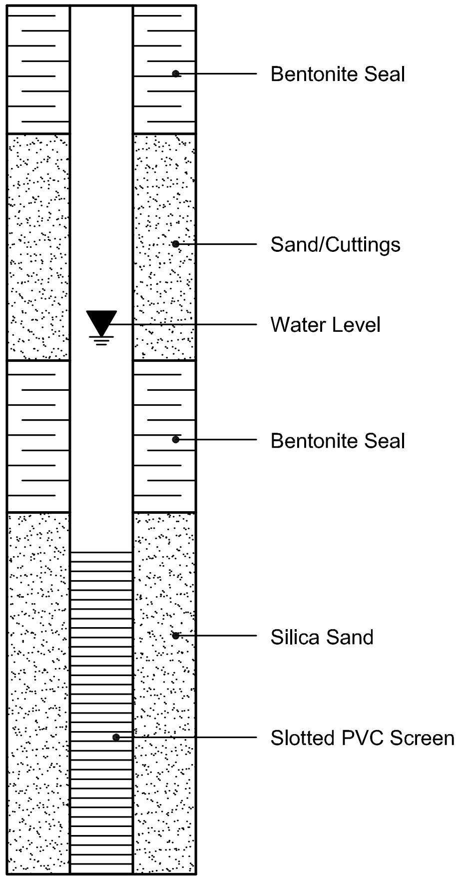
SYMBOLS AND TERMS (continued)

STRATA PLOT

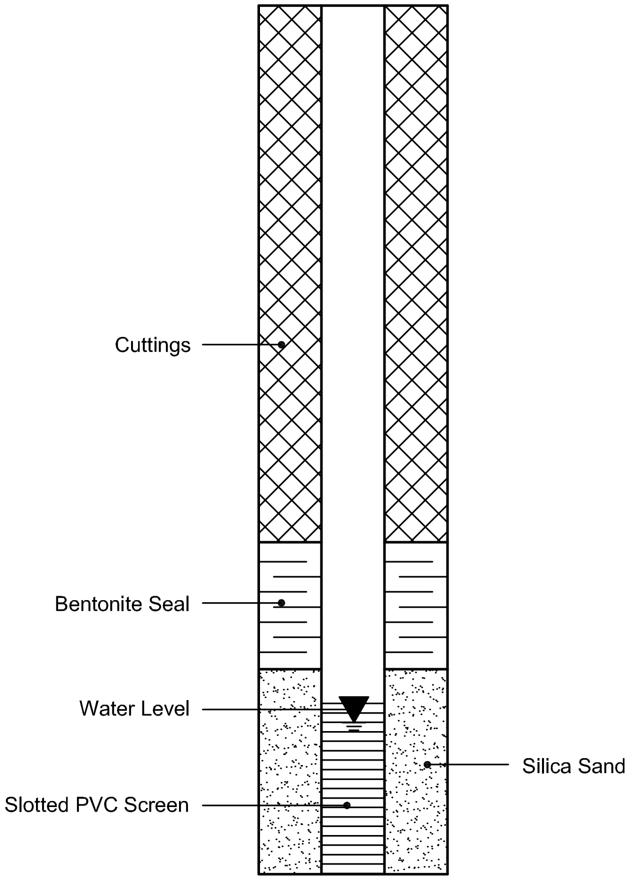


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG7336-1 – TEST HOLE LOCATION PLAN

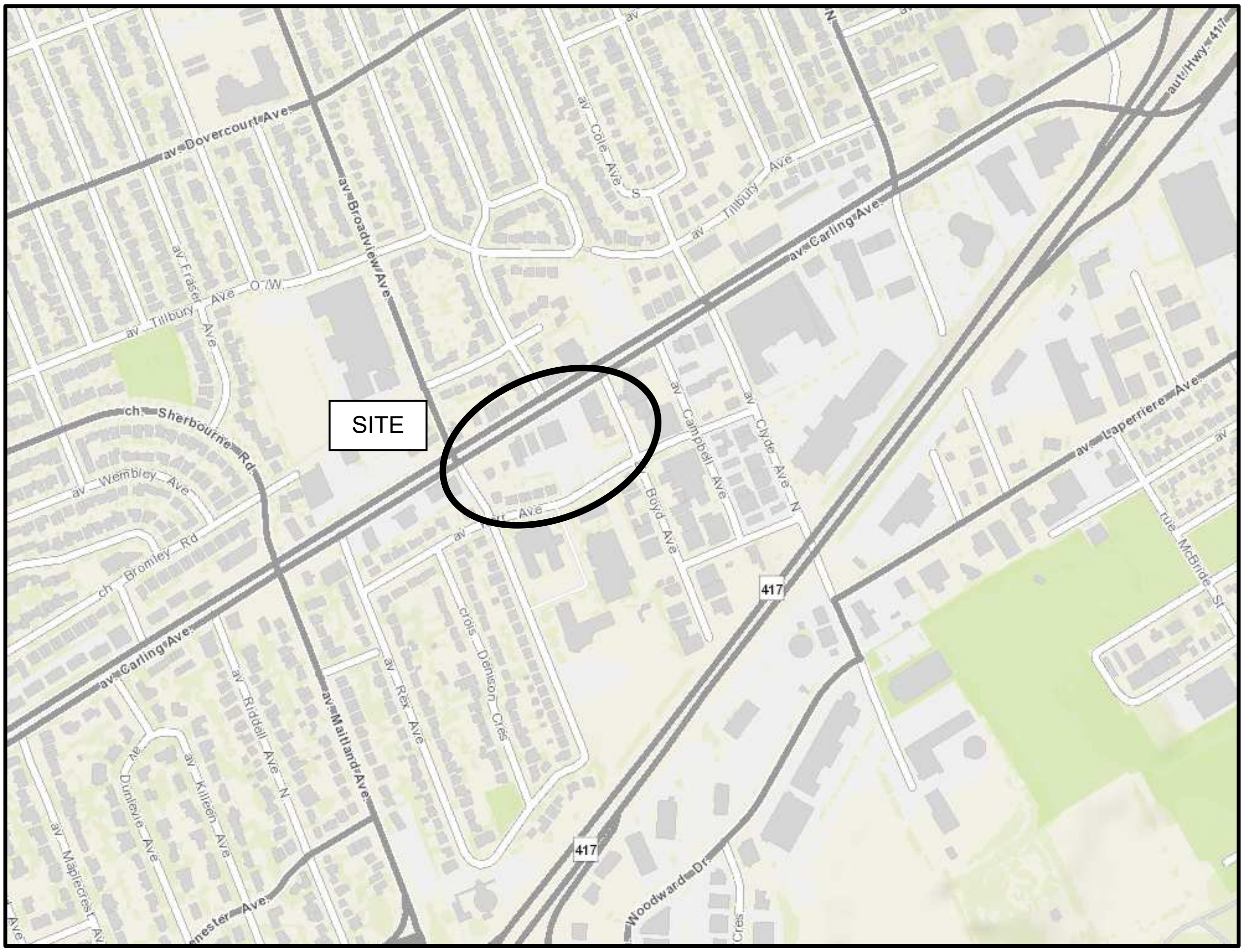
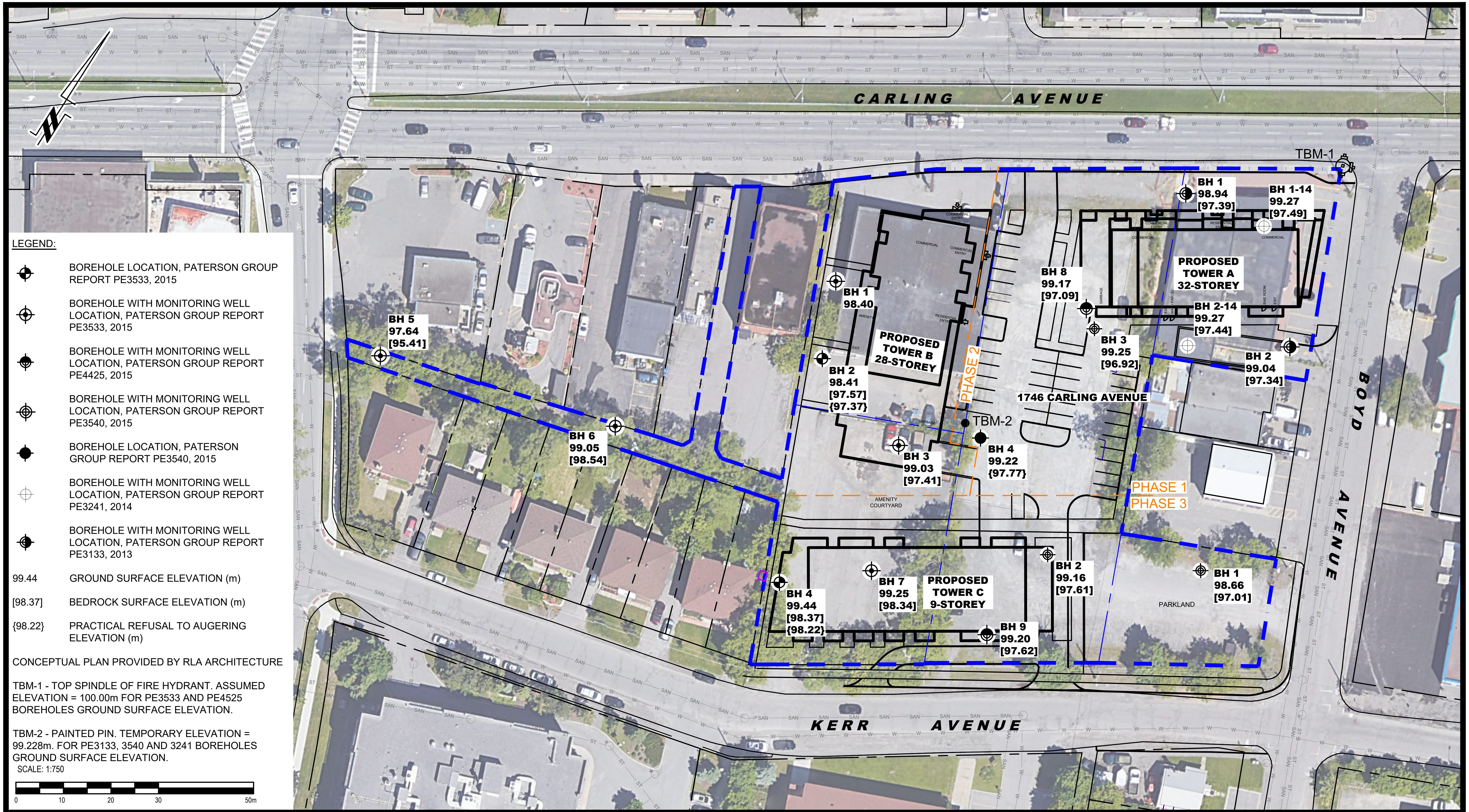


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PE3533, 2015
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3533, 2015
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE4425, 2015
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3540, 2015
●	BOREHOLE LOCATION, PATERSON GROUP REPORT PE3540, 2015
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3241, 2014
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3133, 2013
99.44	GROUND SURFACE ELEVATION (m)
[98.37]	BEDROCK SURFACE ELEVATION (m)
{98.22}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTURE

TBM-1 - TOP SPINDLE OF FIRE HYDRANT. ASSUMED ELEVATION = 100.00m FOR PE3533 AND PE4525 BOREHOLES GROUND SURFACE ELEVATION.

TBM-2 - PAINTED PIN. TEMPORARY ELEVATION = 99.228m. FOR PE3133, 3540 AND 3241 BOREHOLES GROUND SURFACE ELEVATION.

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
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TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

KERR BROADVIEW PROPERTIES LTD.
GEOTECHNICAL DESKTOP REVIEW
PROPOSED MIXED-USE DEVELOPMENT
1746 CARLING AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 10/2024
Drawn by: GK	Report No.: PG7336-1
Checked by: YZ	Dwg. No.: PG7336-1
Approved by: FA	Revision No.:

p:\autocad drawings\geotechnical\pg73xx\pg7336\pg7336-1 test hole location plan.dwg

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