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Geotechnical Investigation
Proposed Multi-Storey Building
70 Richmond Road
Ottawa, Ontario

Prepared For

Devtrin (Island Park) Inc.

May 10, 2022

Report PG5501-1 Revision 4

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1.0   Introduction

Paterson Group (Paterson) was commissioned by Devtrin (Island Park) Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 70 Richmond Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

❏   determine the subsurface soil and groundwater conditions 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of writing this report.

2.0   Proposed Project

It is our understanding that the proposed development will consist of a 9 storey building with two levels of underground parking. It is further understood that the building footprint will occupy the majority of the subject site. Additionally, there is a one storey heritage building located at the southern portion of the subject site. It is anticipated that the heritage building will be relocated to the northern portion of the site. Access lanes and landscaped areas are anticipated for the proposed development.

3.0   Method of Investigation

3.1   Field Investigation

Field Program

The field program for the current supplemental geotechnical investigation was carried out on March 23, 2022. At that time, two boreholes were drilled to a maximum depth of 9.1m below existing ground surface. Previous geotechnical/environmental field programs were carried out at the subject site by Paterson between 1997 and 2021, during which a total of seventeen boreholes were advanced to depths ranging between 1.5 to 10.1 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The locations of the boreholes are shown on Drawing PG5501-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a CME 55 Low Clearance 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.

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon or the auger flights. The split-spoon, auger and rock core samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon, auger and rock core samples were recovered from the boreholes are presented as SS, AU and RC, respectively, on the Soil Profile and Test Data sheets.

Standard Penetration Tests (SPT) were conducted and 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 sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at three locations during the previous 2020 investigation, at BH 7-20, BH 8-20 and BH 9-20, and at two location during the current investigation, at BH 1-22 and BH 2-22, to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

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

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

Groundwater

A series of groundwater monitoring wells were installed in previous and recent boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The remaining boreholes were fitting with flexible piezometer to permit ground water level readings.

3.2   Field Survey

The test hole locations and elevations were surveyed in the field by Paterson using a high precision GPS unit. The ground surface elevations at the test hole locations were referenced to a geodetic datum. The borehole locations and the ground surface elevation of the borehole locations are presented on Drawing PG5501-1 - Test Hole Location Plan in Appendix 2.

3.3   Laboratory Testing

Soil and bedrock samples recovered from the subject site were visually examined in our laboratory to review the field logs.

4.0   Observations

4.1   Surface Conditions

The subject site is located at the southwest corner of the intersection of Richmond Road and Island Park Drive. The site is currently occupied by an at grade asphalt parking lot located at the northern, eastern and western portion of the site. There is an existing one-storey building located at the southern portion of the site, to be partially relocated to the northern portion of the site. The ground surface across the subject site is at grade with the bordering streets and slopes down toward the north.

4.2   Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of top soil/pavement structure overlying fill material consisting of silty sand and crushed stone, that extends to depths up to 3.1 m. A dense brown silty sand to sandy silt with trace of gravel was encountered underlying the fill followed by a loose to dense grey silty sand to silt sand layer. A dense to very dense glacial till deposit, consisting of silty sand with gravel, cobbles and boulders was noted below the fill/silty sand layer. Practical refusal to augering was encountered throughout the site. However, limestone bedrock was confirmed at depths ranging from 5.5 m to 6.2 m at BH 1-22, BH 2-22, BH 7-20, BH 8-20 and BH 9-20. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation. The bedrock surface is expected at depths ranging from 3 to 6 m.

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

4.3   Groundwater

Groundwater levels were measured on April 13, 2022 in several of the current and previous test holes. In addition, groundwater measurements were completed during the previous investigations on June 22, 2012 and August 26, 2020. Table 1 provides a summary of the groundwater level measurements completed during the current and previous investigations.

Table 1 - Measured Groundwater Levels				
Test Hole Number	**Ground Surface Elevation (m)**	**Water Level**		**Date**
		Depth (m)	**Elevation (m)**	
BH 2-12	67.12	2.38	64.74	April 13, 2022
BH 3-12	67.32	2.23	65.09	April 13, 2022
BH 8-20	67.27	3.85	63.42	April 13, 2022
BH 9-20	67.20	2.73	64.47	April 13, 2022
BH 1-22	68.19	2.64	65.55	April 13, 2022
BH 2-22	67.90	2.67	65.23	April 13, 2022
BH 7-20	67.34	5.13	62.21	August 26, 2020
BH 8-20	67.27	4.17	63.10	August 26, 2020
BH 9-20	67.20	4.37	62.83	August 26, 2020
BH 1-12	67.49	2.60	64.89	June 22, 2012
BH 2-12	67.12	2.50	64.62	June 22, 2012
BH 3-12	67.32	2.57	64.75	June 22, 2012
BH 4-12	67.85	2.67	65.18	June 22, 2012
BH 5-12	67.80	2.66	65.14	June 22, 2012

Groundwater levels are subject to seasonal fluctuations and therefore levels could differ at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed multi-storey building is expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the two levels of underground parking. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal should be required for the construction of the parking garage levels.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the 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. A minimum of 1 m horizontal bench, should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or a stable base for the overburden shoring system.

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. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. 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.

Bedrock Excavation Face Reinforcement

Angled rock anchors and/or chain link fencing 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. The requirement for rock anchors and chain link fencing will be evaluated during the excavation operations.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the 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 lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3   Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface could be designed for a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and should not contain surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5, can be provided if founded on limestone bedrock which is free of seams, fractures and voids within 1.5 m below the founding level. This should be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within all the footing footprints. A minimum of one probe hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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

Footings placed on an undisturbed, compact glacial till bearing surface or over an approved engineered fill over an undisturbed, glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa.** Engineered fill should be placed in accordance with our recommendations in Subsection 5.2.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious material, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

The bearing resistance value given at SLS will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

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

5.4   Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012 (OBC 2012). The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 3 and 4 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5501-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations were 1, 1.5 and 15 m away from the first geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the bedrock shear wave velocity is **1,600 m/s**. It is understood that the overburden will be completely removed as part of the proposed building and footings will be placed on the bedrock surface. The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the Ontario Building Code (OBC) 2012, and as presented below.

$$V_{s30} = \frac{Depth_{ofinterest}(m)}{\dfrac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)} + \dfrac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}}$$

$$V_{S30} = \frac{30m}{\left(\dfrac{30m}{1,600m/s}\right)}$$

$$V_{S30} = 1,600 m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30} is **1,600 m/s** for conventional shallow footings which are founded on the bedrock surface. Therefore, a **Site Class A** is applicable for design of the proposed building in this case, as per Table 4.1.8.4A of the OBC 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone. Alternatively, excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement.

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

It is understood that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

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 dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating 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   Rock Anchor Design

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

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

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

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

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

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

Grout to Rock Bond

Based on the testing results, the unconfined compressive strength of the limestone bedrock below the subject site ranges between 80 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 59** was assigned to the bedrock. Therefore, Hoek and Brown parameters (**m and s**) were taken as **0.447 and 0.00212**, respectively.

Recommended Rock Anchor Lengths

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

For our calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	30 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	m=0.447 and s=0.00212
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

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

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.2	0.6	2.8	500
	3.0	0.8	3.8	750
	3.6	1.0	4.6	1000
	4.5	1.0	5.5	1200
125	2.3	0.7	3.0	750
	2.8	0.8	3.6	1000
	3.2	1.0	4.2	1500
	4.9	1.0	5.9	2000

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

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

5.8   Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and access lanes, if required.

Table 4 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 5 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be poured against a drainage system placed against the shoring face. It is anticipated that the maximum groundwater in-flow during the spring thaw and rain events will range between 25,000 and 30,000 L/day with the partially tanked groundwater suppression and foundation drainage system. Refer to Figure 2 – Groundwater Suppression and Foundation Drainage System, for specific details of the foundation drainage recommendations attached to the current memorandum.

To manage and control groundwater infiltration to the building's storm sump pump(s) over the long term, the following foundation drainage and water suppression system is recommended to be installed on the exterior perimeter and surface of the building's foundation walls using the following methodology:

❏   Throughout the building excavation and bedrock removal process, the vertical bedrock should be hoe-rammed and grinded to provide a smooth and flat substrate surface approved for the placement of the waterproofing membrane. Shotcrete and/or lean concrete anchored into the bedrock with steel dowels and/or rock anchors may be required to fill in cavities and smooth out angular features and voids. This process and the requirement for shotcrete and/or lean concrete should be periodically reviewed by Paterson personnel during the excavation program.

❏   A waterproofing membrane will be required to lessen the effect of water infiltration for the lower underground parking level between the underside of footing elevation and up to the top of slab of the first level of underground parking. The waterproofing membrane should consist of a 150 miL granular bentonite surface laminated to 20 miL thick HDPE membrane. The membrane should be installed in horizontal lifts and in accordance with the manufacturer's specifications in a shingle fashion with the HDPE side facing the applicator/the building to an adequately prepared substrate surface.

❏   The waterproofing membrane should extend horizontally a minimum of 600 mm below the building's perimeter footings as depicted on Figure 2 - Groundwater Suppression and Foundation Drainage System.

❏   As a secondary measure to capture any groundwater that may, in the unlikely event bypass the waterproofing membrane, it is recommended that a composite foundation drainage membrane, such as 6000 series membrane by DeltaDrain, G100N by MiraDrain or equivalent and approved other, be placed between the waterproofing membrane and exterior foundation wall surfaces. The composite foundation drainage membrane should extend from finished grade to the underside of footing level with the geotextile layer facing the waterproofing membrane.

❏   150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the storm sump pit(s) within the lower basement area. It is recommended that the 150 mm diameter drainage sleeves be installed by carefully cutting an 'X' shaped incision through the composite foundation drainage. In areas where the drainage sleeves are to be installed, care must be taken to ensure that the underlying waterproofing membrane is **NOT** punctured during this process.

Transition from Blind Side to Double Sided Formwork

It is anticipated that the construction of the exterior perimeter foundation walls will transition from a blind sided pour into a conventional double sided pour. The integrity of the water suppression system should be maintained across this transition zone as per the following methodology:

❏   It is critical that the foundation drainage board and/or waterproofing membrane is extended in a suitable manner through this transition zone to maintain the long-term performance of the foundation drainage and waterproofing system. As a result, the contractor should be prepared to provide temporary formwork to ensure this transition is completed in a suitable manner. This would generally consist of the top of the temporary shoring and other surfaces where the foundation wall will be cast in a transition zone from blind side to double side pours.

❏ After the modified temporary formwork is strategically installed at the shoring interface (where required), the foundation drainage and/or waterproofing membrane can be extended over the shoring face and securely fastened to the inside of the custom formwork prior to the placement of the concrete.

❏ Once the foundation wall has been poured and the modified temporary formwork has been removed, the exposed portion of the composite foundation drainage membrane and/or waterproofing membrane can be overlapped in a shingle fashion.

Figure 2 of Appendix 2 presents the proposed groundwater suppression and foundation drainage system. Further design information can be provided at the time of design as required.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. The spacing of the underfloor drainage system should be confirmed at the time of excavation when water infiltration can be better assessed. For design purposes, it's suggested that one 150 mm in diameter perforated drainage pipe be placed in each bay.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Backfill

Above the bedrock surface, 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.

6.2   Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided. Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3   Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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 groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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.

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 designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. 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 impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could 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 included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. 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. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 6 - Soil Parameters	
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
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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

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

6.5   Groundwater Control

Groundwater Control for Building Construction

Due to existing groundwater level and inferred depths of the proposed footings, 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.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package 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, and EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed.   It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

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

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

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, 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.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means. 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.

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

7.0 Recommendations

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

❏   Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

❏   Review the bedrock stabilization and excavation requirements, if applicable.

❏   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.

❏   Periodic observation of the installation of the water suppression system.

❏   Observation of all subgrades prior to backfilling.

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

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

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

8.0   Statement of Limitations

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

A geotechnical investigation 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. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the 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 Devtrin (Island Park) Inc., 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.

Maha K. Saleh, M.A.Sc., P.Eng.   David J. Gilbert, P.Eng.

May 10, 2022

D. J. GILBERT
100116130

Report Distribution:

❏   Devtrin (Island Park) Inc.
❏   Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial/Residential Development
70 Richmond Road - Ottawa, Ontario

DATUM	Geodetic		FILE NO. **PG5501**

REMARKS

BORINGS BY CME-55 Low Clearance Drill	DATE March 23, 2022	HOLE NO. **BH 1-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	68.19
TOPSOIL 0.18		AU	1				
FILL: Brown silty sand with gravel, cobbles and boulders, some clay		SS	2	25	32	1	67.19
1.68		SS	3	42	39	2	66.19
Dense to compact, brown **SILTY SAND to SANDY SILT,** trace gravel		SS	4	38	9		
- grey by 2.0m depth		SS	5	50	10	3	65.19
		SS	6	54	13	4	64.19
		SS	7	50	25	5	63.19
5.26							
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel, cobbles and boulders		SS	8	50	50+		
6.20		SS	9	50	50+	6	62.19
		RC	1	100	83	7	61.19
BEDROCK Fair to good quality, grey limestone							
		RC	2	100	68	8	60.19
9.02						9	59.19
End of Borehole							
(GWL @ 2.64m - April 13, 2022)							

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial/Residential Development
70 Richmond Road - Ottawa, Ontario

DATUM Geodetic	**FILE NO.** **PG5501**
REMARKS	**HOLE NO.**
BORINGS BY CME-55 Low Clearance Drill **DATE** March 23, 2022	**BH 2-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						0	67.90		
Asphaltic concrete　0.13		AU	1						
FILL: Crushed stone with brown silty sand 0.28									
FILL: Brown silty sand, trace gravel and crushed stone		SS	2	38	14	1	66.90		
1.68									
		SS	3	46	38	2	65.90		
Dense, brown **SILTY SAND to SANDY SILT,** trace gravel		SS	4	75	33				▼
- grey by 2.2m depth						3	64.90		
		SS	5	75	35				
3.73									
		SS	6	54	13	4	63.90		
GLACIAL TILL: Compact, grey silty sand to sandy silt with gravel, cobbles and boulders		SS	7	54	20	5	62.90		
		SS	8	50	50				
5.79		RC	1	100	40	6	61.90		
		RC	2	100	83	7	60.90		
BEDROCK: Fair to good quality, grey limestone									
						8	59.90		
		RC	3	100	88				
9.12						9	58.90		
End of Borehole									
(GWL @ 2.67m - April 13, 2022)									

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

Geotechnical Investigation
70 Richmond Road
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation =
REMARKS	68.31m.

FILE NO.
PG5501

HOLE NO.
BH 7-20

BORINGS BY CME-55 Low Clearance Drill   **DATE** July 27, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	67.34		
Asphaltic concrete 0.08		AU	1						
FILL: Brown silty sand with crushed stone 0.66									
		SS	2	47	50+	1	66.34		
FILL: Brown to grey silt and sand with gravel		SS	3	50	53	2	65.34		
		SS	4	58	54				
3.05		SS	5	63	50+	3	64.34		
Dense, grey **SANDY SILT**									
		SS	6	33	31	4	63.34		
4.42		SS	7	50	44				
Dense to very dense, grey **SILTY SAND-GRAVEL**						5	62.34		▼
		SS	8	67	50+			●	
6.00						6	61.34		
		RC	1	94	74	7	60.34		
BEDROCK: Fair to excellent quality, grey limestone						8	59.34		
		RC	2	93	70				
						9	58.34		
		RC	3	100	100				
10.06						10	57.34		
End of Borehole									
(GWL @ 5.13m - August 26, 2020)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
70 Richmond Road
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation =
REMARKS	68.31m.

BORINGS BY CME-55 Low Clearance Drill   **DATE** July 27, 2020

FILE NO.
PG5501

HOLE NO.
BH 8-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						0	67.27		
Asphaltic concrete 0.08		AU	1						
FILL: Brown silty sand with crushed stone		SS	2	42	14	1	66.27		
1.37		SS	3	88	50+				
		G	4			2	65.27		
Very dense, brown **SILTY SAND** with gravel		SS	5	50	50+	3	64.27		
- grey by 3.0m depth		SS	6	90	86	4	63.27		▼
		SS	7	42	84	5	62.27		
		SS	8	84	34				
6.12						6	61.27		
		RC	1	100	36	7	60.27		
BEDROCK: Poor to excellent quality, grey limestone		RC	2	100	67	8	59.27		
						9	58.27		
		RC	3	100	100				
10.08						10	57.27		
End of Borehole									
(GWL @ 3.85m - April 13, 2022)									

Water Content %: 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

**Geotechnical Investigation
70 Richmond Road
Ottawa, Ontario**

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.31m.	**FILE NO.** **PG5501**
REMARKS		
BORINGS BY	CME-55 Low Clearance Drill DATE July 27, 2020	**HOLE NO.** **BH 9-20**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						0 — 67.20			
Asphaltic concrete 0.10		AU	1						
FILL: Brown silty sand with crushed stone		SS	2	4	10	1 — 66.20			
1.52		SS	3	33	50+				
		SS	4	20	50+	2 — 65.20			
		SS	5	0	50+	3 — 64.20			▼
Very dense to compact, brown **SILTY SAND** with gravel		SS	6	38	24	4 — 63.20			
		SS	7	50	24	5 — 62.20			
5.51		SS	8	100	50+				
		RC	1	100	0				
		RC	2	100	25	6 — 61.20			
BEDROCK: Very poor to good quality, grey limestone		RC	3	100	62	7 — 60.20			
		RC	4	100	79	8 — 59.20			
9.07						9 — 58.20			
End of Borehole									
(GWL @ 2.73m - April 13, 2022)									

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

Phase II - Environmental Site Assessment
70 Richmond Road
Ottawa, Ontario

FILE NO.	**PE2637**
HOLE NO.	**BH 1-12**

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation =
REMARKS 68.31m.

BORINGS BY CME 55 Power Auger   **DATE** May 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.49
Asphaltic concrete 0.05							
FILL: Crushed stone 0.15							
FILL: Brown silty sand with gravel		SS	1	33	21	1	66.49
		SS	2	67	50+		
2.13						2	65.49
		RC	1	50	0		
						3	64.49
		RC	2	23	0		
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders						4	63.49
		RC	3	0	0	5	62.49
		RC	4	33	0	6	61.49
6.10							
End of Borehole							
(GWL @ 2.60m-June 22, 2012)							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

Monitoring Well Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
70 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = **REMARKS** 68.31m.	**FILE NO.** **PE2637**
BORINGS BY CME 55 Power Auger **DATE** May 22, 2012	**HOLE NO.** **BH 2-12**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %

GROUND SURFACE

Depth	Soil
0.05	Asphaltic concrete
0.20	**FILL:** Crushed stone

FILL: Brown silty sand with gravel

Type	Number	% Recovery	N Value or RQD
SS	1	25	21
SS	2	0	

2.21

GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders

Type	Number	% Recovery	N Value or RQD
SS	3	42	60
SS	4	40	50+
SS	5	67	50+
SS	6	33	50+

5.38

End of Borehole

Practical refusal to augering @ 5.38m depth

(GWL @ 2.38m - April 13, 2022)

Depth (m) / Elev. (m):
0 / 67.12
1 / 66.12
2 / 65.12
3 / 64.12
4 / 63.12
5 / 62.12

Lower Explosive Limit %: 20 40 60 80

RKI Eagle Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
70 Richmond Road
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.31m.	**FILE NO.** PE2637
REMARKS		
BORINGS BY CME 55 Power Auger	DATE May 22, 2012	**HOLE NO.** BH 3-12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	67.32		
Asphaltic concrete 0.05									
FILL: Crushed stone 0.15									
FILL: Brown silty sand with gravel		SS	1	17	12	1	66.32		
		SS	2	17	6	2	65.32		
2.21									
		SS	3	43	30				
		SS	4	100	50+	3	64.32		
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles, boulders		SS	5	89	50+	4	63.32		
		SS	6	67	50+				
						5	62.32		
		SS	7	44	50+				
5.64									
End of Borehole									
Practical refusal to augering @ 5.64m depth									
(GWL @ 2.23m - April 13, 2022)									

Lower Explosive Limit %: 20 40 60 80

RKI Eagle Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
70 Richmond Road
Ottawa, Ontario

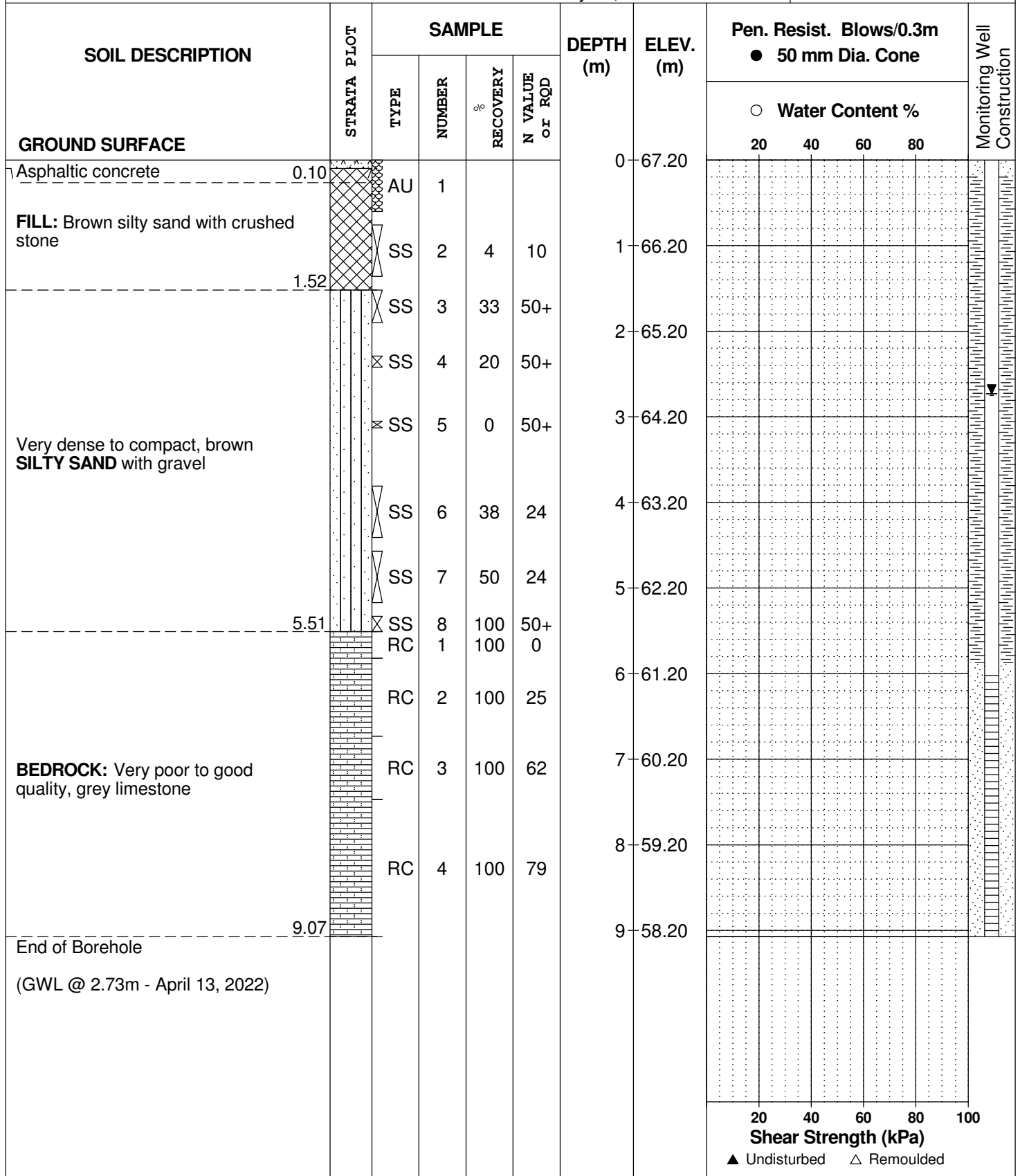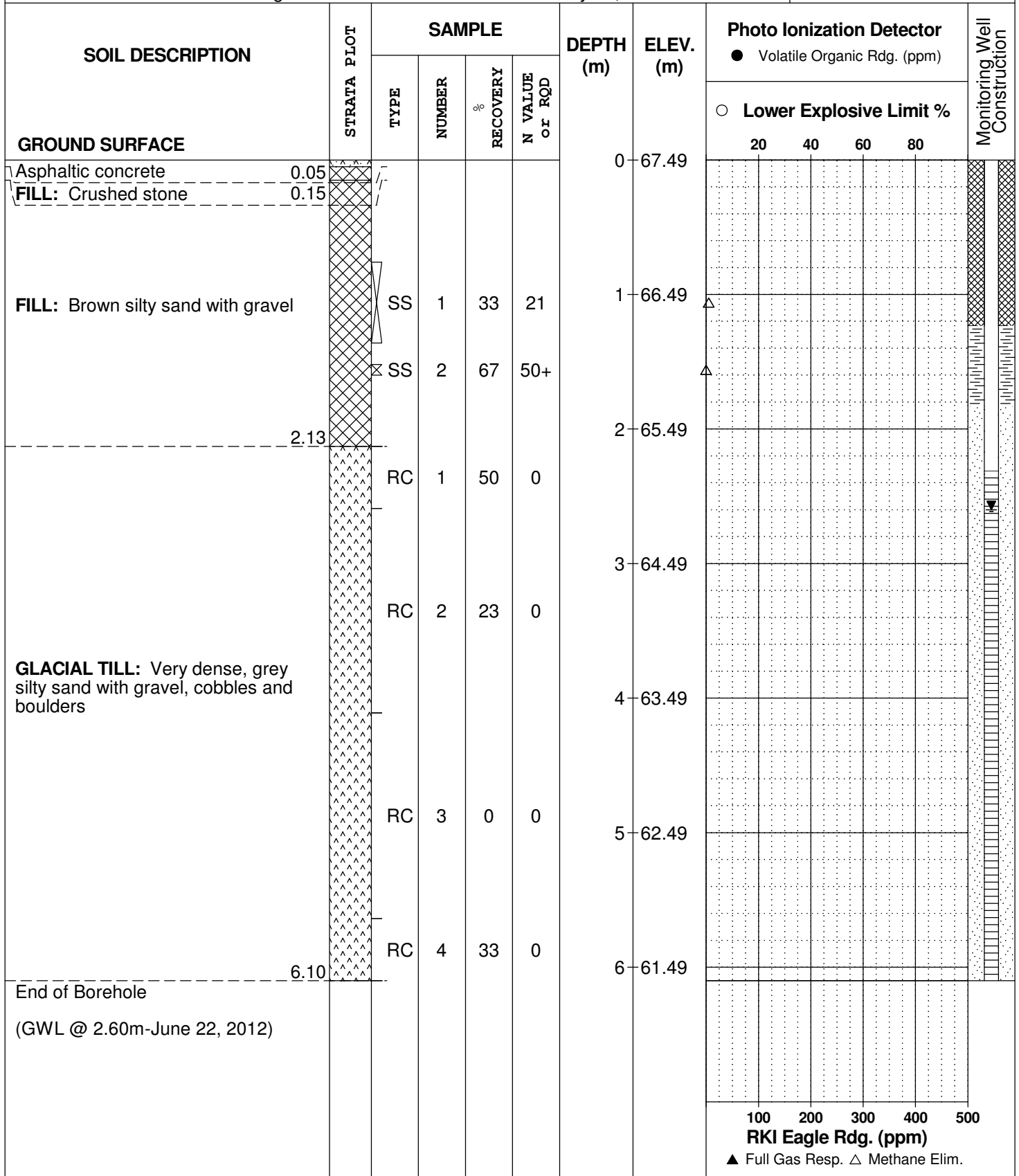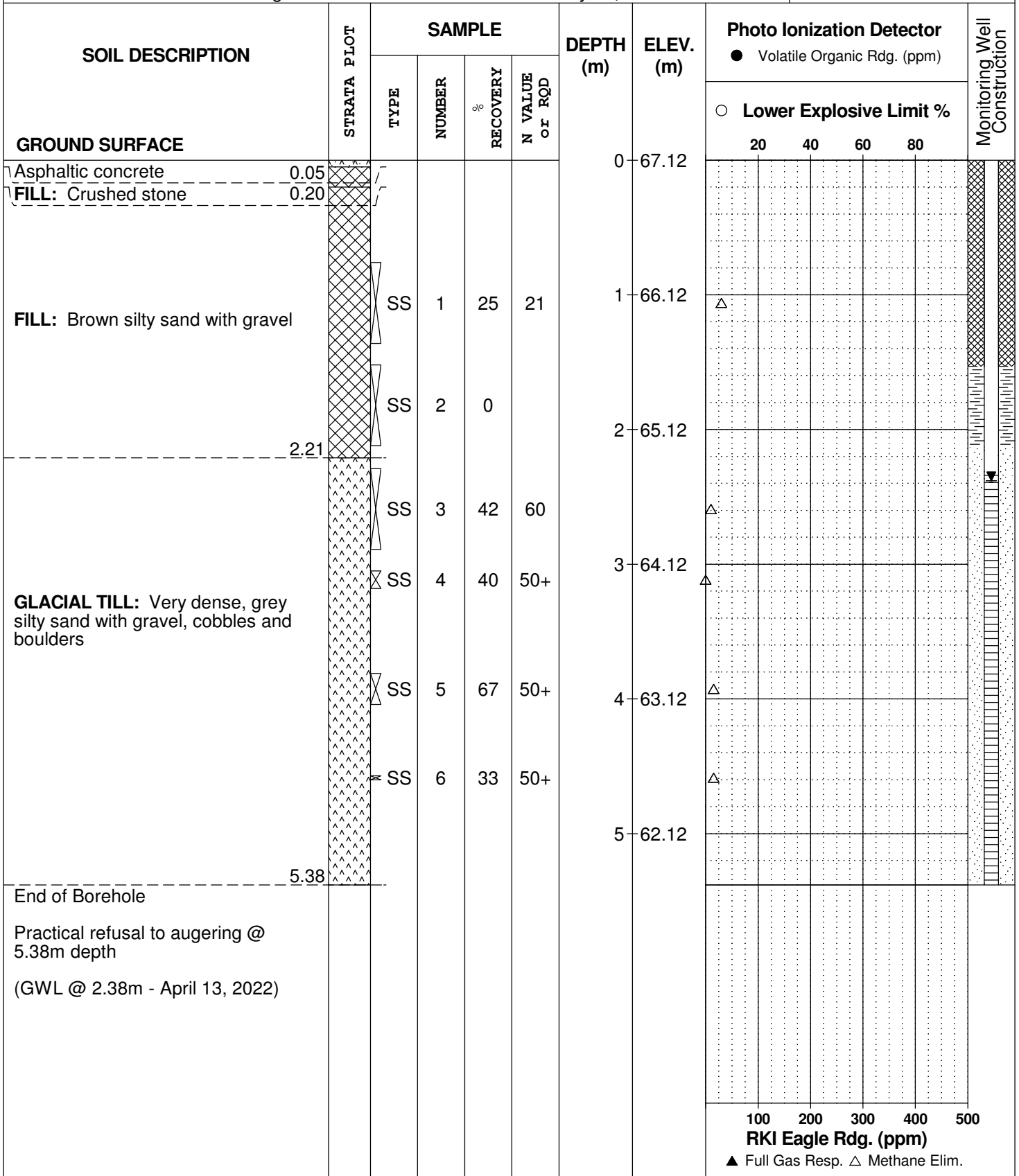| DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.31m.	FILE NO. **PE2637**
REMARKS		HOLE NO. **BH 4-12**

BORINGS BY Geo Probe   **DATE** June 18, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector / Monitoring Well Construction
GROUND SURFACE						0	67.85	Photo Ionization Detector — ● Volatile Organic Rdg. (ppm) — ○ Lower Explosive Limit %
FILL: Brown silty sand with gravel		SS	1	75				
1.22						1	66.85	
FILL: Brown silty sand with gravel, cobbles and boulders		SS	2	100		2	65.85	
2.44								
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	3	100		3	64.85	
		SS	4	100		4	63.85	
		SS	5	100		5	62.85	
6.10						6	61.85	
End of Borehole								
(GWL @ 2.67m-June 22, 2012)								

RKI Eagle Rdg. (ppm) — scale: 100, 200, 300, 400, 500

▲ Full Gas Resp.   △ Methane Elim.

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
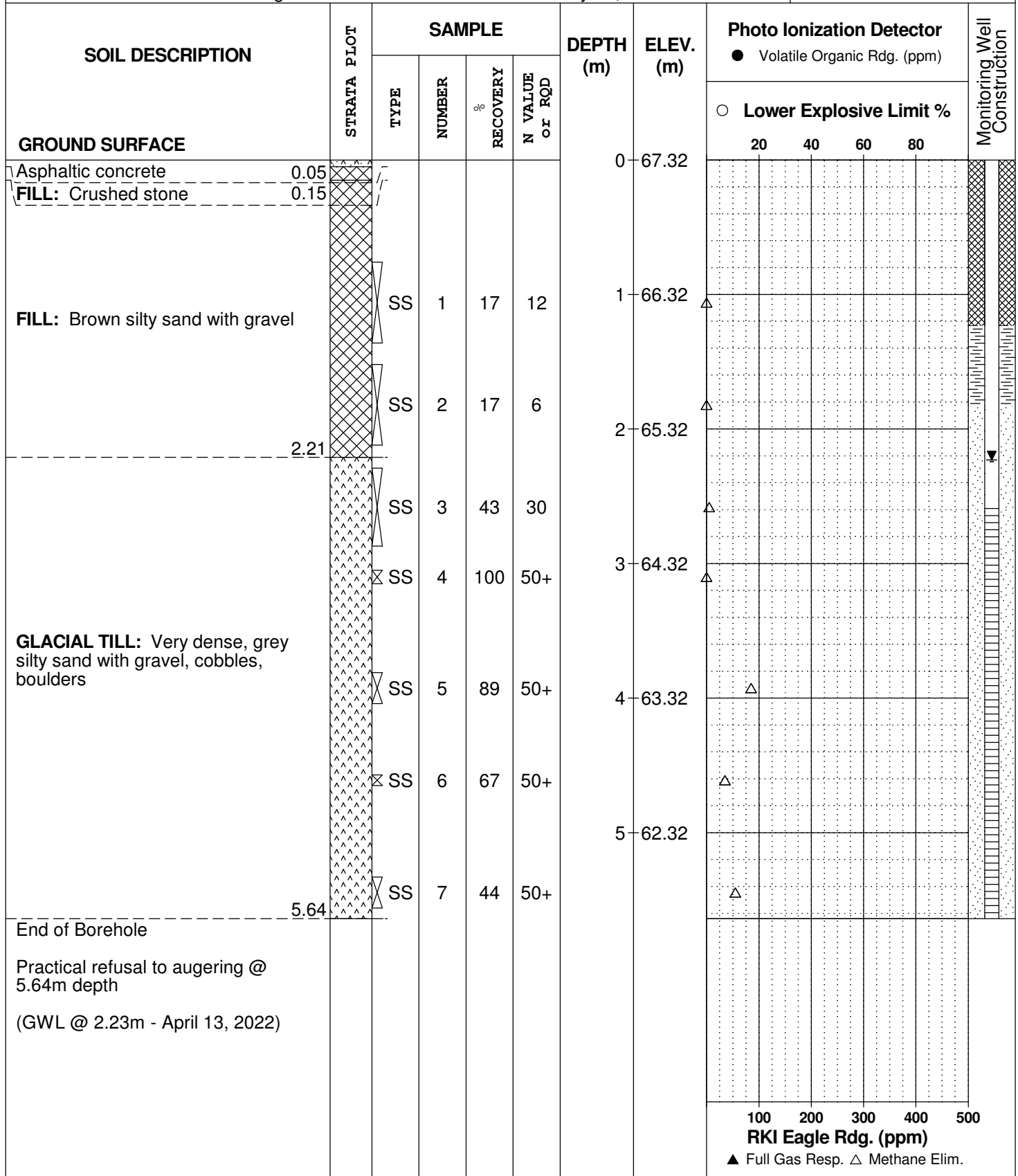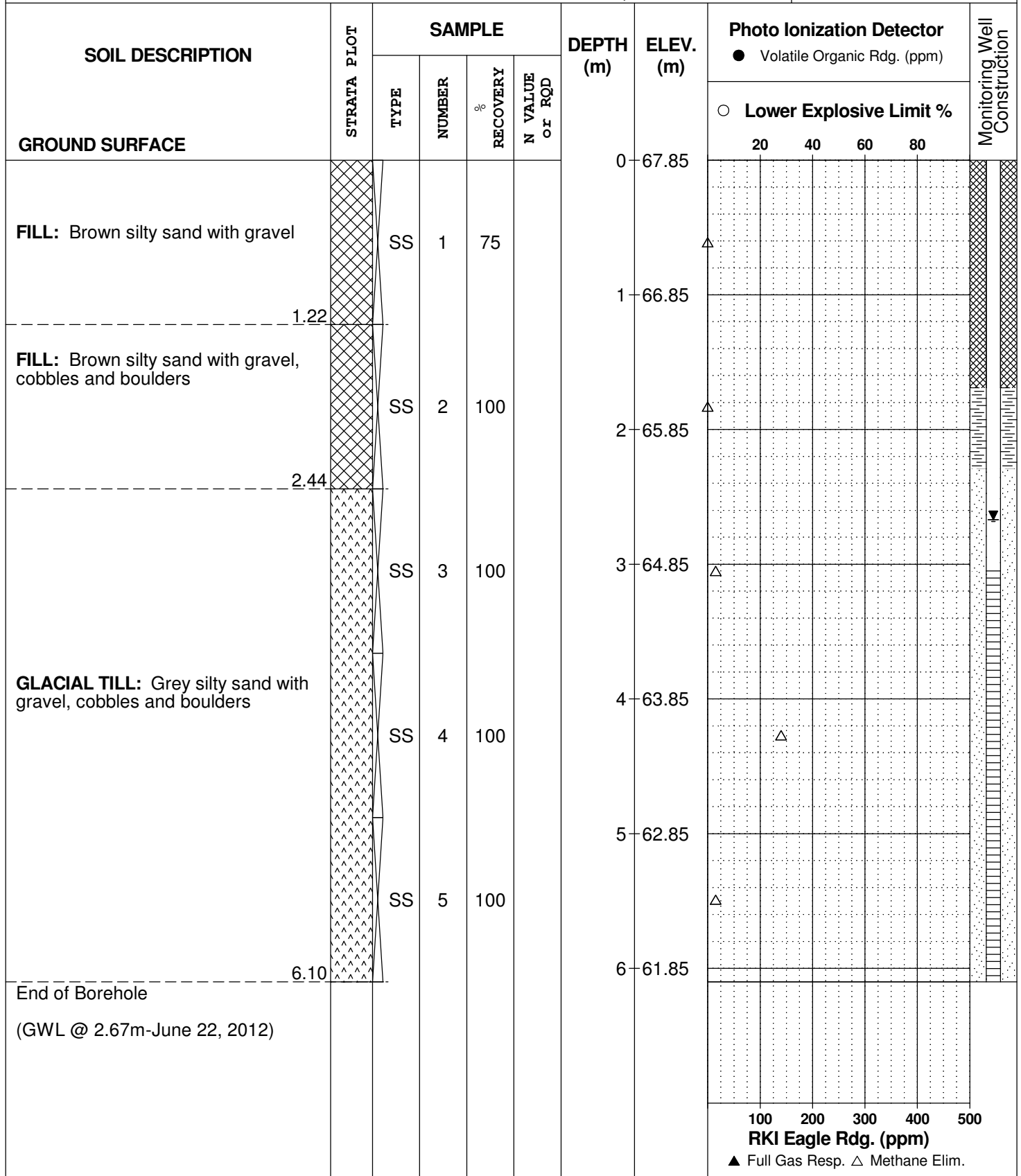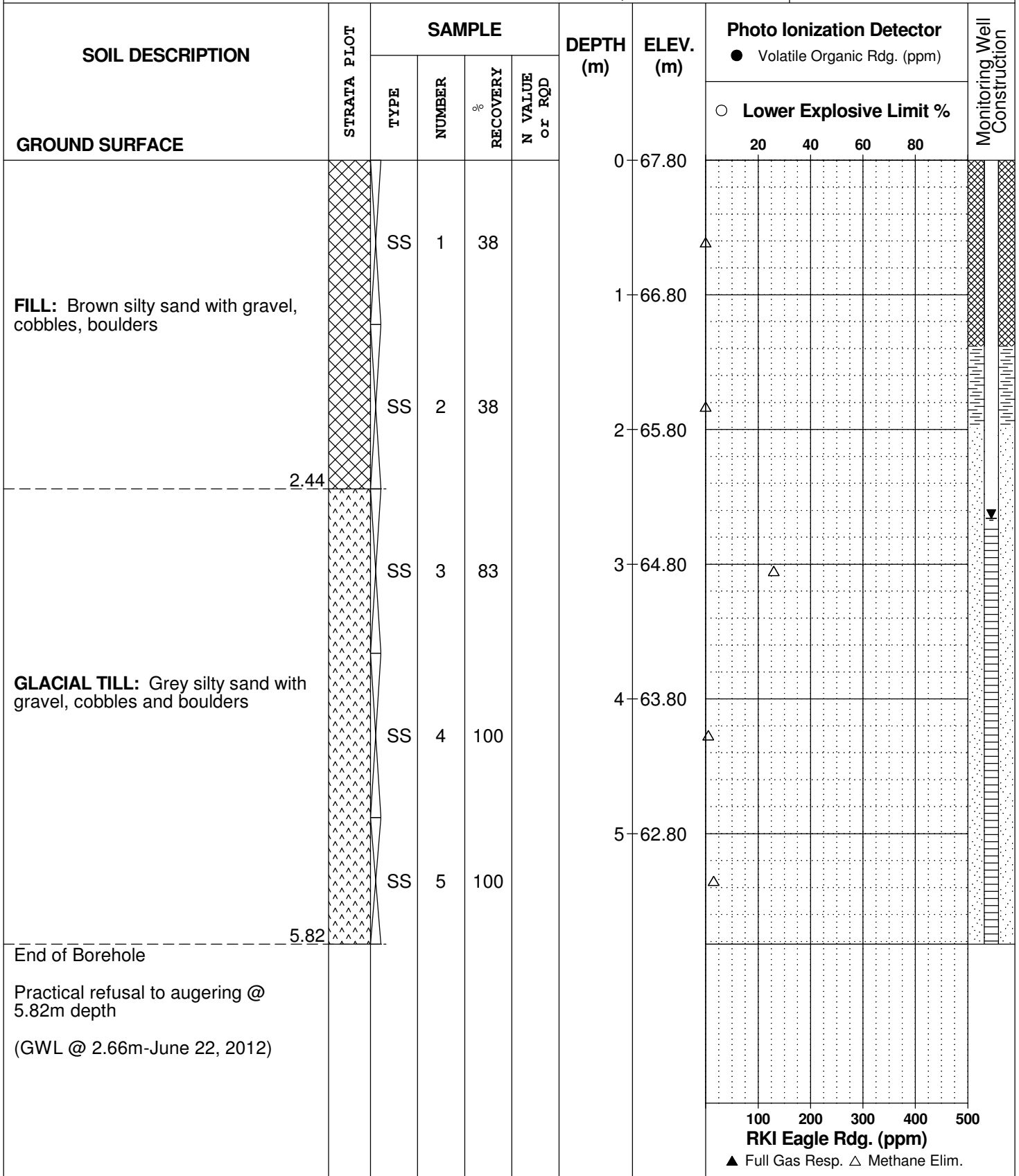**70 Richmond Road**
Ottawa, Ontario

| DATUM | TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = | **FILE NO.** **PE2637** |
| REMARKS | 68.31m. | |

BORINGS BY Geo Probe   **DATE** June 18, 2012   **HOLE NO.** **BH 5-12**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	67.80		
FILL: Brown silty sand with gravel, cobbles, boulders		SS	1	38					
		SS	2	38		1	66.80		
2.44						2	65.80		
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	3	83		3	64.80		
		SS	4	100		4	63.80		
		SS	5	100		5	62.80		
5.82									
End of Borehole									
Practical refusal to augering @ 5.82m depth									
(GWL @ 2.66m-June 22, 2012)									

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)
○ **Lower Explosive Limit %**
20   40   60   80

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

SOIL PROFILE AND TEST DATA

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Phase II - Environmental Site Assessment
70 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = **REMARKS** 68.31m.	**FILE NO.** **PE2637**
	HOLE NO. **BH 6-12**

BORINGS BY Geo Probe   **DATE** June 18, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	

GROUND SURFACE

Soil Description	Depth	Sample	Depth (m)	Elev (m)
Asphaltic concrete	0.05		0	67.76
FILL: Crushed stoned	0.20			
FILL: Brown silty sand with gravel, cobbles, boulders		SS 1 63		
			1	66.76
		SS 2 50	2	65.76
	2.44			
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS 3 100	3	64.76
		SS 4 42	4	63.76
		SS 5 50	5	62.76
	5.59			

End of Borehole

Practical refusal to augering @ 5.59m depth

Lower Explosive Limit %: 20 40 60 80

RKI Eagle Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp.   △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.31m	**FILE NO.** E1399
REMARKS Richmond Road at Island Park Drive	**HOLE NO.** BH 2-3
BORINGS BY Truck-mount Drill **DATE** January 7, 1997	

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.27
Asphaltic concrete 0.08							
Concrete 0.23							
FILL: Brown sand-gravel							
1.12		SS	3	83	8	1	66.27
FILL: Mixture of silt, sand, gravel and organics (possible glacial till)		SS	4	83	38		
1.90		AU	5				
End of Borehole							
Auger refusal on inferred bedrock at 1.90m depth							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

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

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

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

FILE NO.	**E1399**
HOLE NO.	**BH 3-3**

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.81m

REMARKS Richmond Road at Island Park Drive

BORINGS BY Truck-mount Drill

DATE January 7, 1997

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)
○ **Lower Explosive Limit %**
20 40 60 80

GROUND SURFACE

Description	Depth	Type	Number	% Recovery	N Value	Depth	Elev.
Asphaltic concrete	0.08					0	67.69
Grey crushed limestone	0.23						
FILL: Brown sand-gravel	1.07	SS	6	75	7	1	66.69
Dark brown silty **TOPSOIL**	1.30						
		SS	7	42	24	2	65.69
GLACIAL TILL: Compact, grey silty sand-gravel		SS	8	83	21		
		SS	9	92	23	3	64.69
End of Borehole	3.35						

Monitoring Well Construction

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

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.9m

FILE NO. E1399

REMARKS Richmond Road at Island Park Drive

HOLE NO. BH 4-3

BORINGS BY Truck-mount Drill   **DATE** February 6, 1997

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.70
Asphaltic concrete 0.08							
FILL: Grey crushed limestone to brown sand-gravel							
1.09						1	66.70
FILL: Dark brown to black, mixture of silt, sand and gravel		AU	10				
1.67							
GLACIAL TILL: Very dense, light grey sand-gravel 1.88		SS	11		50+		
End of Borehole							
Auger refusal on inferred bedrock at 1.88m depth							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20	40	60	80

Monitoring Well Construction

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

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.8m	**FILE NO.** E1399
REMARKS Richmond Road at Island Park Drive	**HOLE NO.** BH 5-3

BORINGS BY Truck-mount Drill   **DATE** February 6, 1997

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector
GROUND SURFACE						0	67.70	
Asphaltic concrete 0.08								
FILL: Grey crushed stone 0.41								
FILL: Brown sand-gravel 1.07						1	66.70	
FILL: Dark brown, mixture of silt, sand and gravel 1.52		AU	12					△
GLACIAL TILL: Very dense, brownish grey sand-gravel 1.98		SS	13	67	50+			△
End of Borehole								
Auger refusal on inferred bedrock at 1.98m depth								

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)
○ **Lower Explosive Limit %**
20   40   60   80

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.81m
REMARKS	Richmond Road at Island Park Drive
BORINGS BY Truck-mount Drill	**DATE** February 6, 1997

FILE NO. E1399

HOLE NO. BH 6-3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	67.61		
Asphaltic concrete 0.08									
FILL: Grey crushed stone 0.30									
FILL: Brown sand-gravel									
1.07						1	66.61		
FILL: Dark brown to black, mixture of silt, sand and gravel 1.32		AU	14					△	
GLACIAL TILL: Very dense, grey sand-gravel		SS	15		50+			760 △	
1.88									
End of Borehole									
Auger refusal on inferred bedrock at 1.88m depth									

Lower Explosive Limit scale: 20 40 60 80

RKI Eagle Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

FILE NO. E1399

HOLE NO. BH 7-3

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.8m

REMARKS Richmond Road at Island Park Drive

BORINGS BY Truck-mount Drill

DATE February 6, 1997

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.50
Asphaltic concrete 0.08							
FILL: Crushed stone 0.25							
FILL: Brown sand-gravel 0.86							
FILL: Dark brown to black, mixture of silt, sand and gravel 1.14		AU	16			1	66.50
GLACIAL TILL: Very dense, grey sand-gravel		SS	17	75	92	2	65.50
2.13							
End of Borehole							
Auger refusal on inferred bedrock at 2.13m depth							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

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

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

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.81m
REMARKS	Richmond Road at Island Park Drive
BORINGS BY	Truck-mount Drill
DATE	February 6, 1997

FILE NO. E1399

HOLE NO. BH 8-3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	67.57		
Asphaltic concrete 0.08									
FILL: Grey crushed stone 0.25									
FILL: Brown sand-gravel									
						1	66.57	△	
Black, silty **TOPSOIL** 1.07 / 1.19		AU	18						
GLACIAL TILL: Dense to very dense, brown to grey sand-gravel		SS	19	100	45			△	
1.98									
End of Borehole									
Auger refusal on inferred bedrock at 1.98m depth									

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

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.91m	FILE NO. **E1399**
REMARKS	Richmond Road at Island Park Drive	HOLE NO. **BH 9-3**
BORINGS BY	Truck-mount Drill DATE February 6, 1997	

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0 — 67.17	
Asphaltic concrete 0.08							
FILL: Grey crushed stone 0.23							
FILL: Brown sand-gravel							
1.02						1 — 66.17	
FILL: Brown, mixture of silt, sand and gravel 1.24		AU	20				
Dark brown, silty **TOPSOIL** 1.32							
GLACIAL TILL: Very dense, brown to grey sand-gravel							
1.68		SS	21	100	60		
End of Borehole							
Auger refusal on inferred bedrock at 1.68m depth							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

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

720 △

Monitoring Well Construction

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

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.8m	FILE NO. **E1399**
REMARKS	Richmond Road at Island Park Drive	
BORINGS BY Truck-mount Drill	DATE February 6, 1997	HOLE NO. **BH10-3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	

GROUND SURFACE

Asphaltic concrete 0.10

FILL: Grey crushed stone over brown to grey sand-gravel

AU 22 — DEPTH 1 — ELEV. 66.58

SS 23 75 5

1.98

GLACIAL TILL: Compact to dense, dark grey to grey sand, some silt and gravel to silty sand, some gravel

SS 24 83 34

SS 25 100 58 — DEPTH 3 — ELEV. 64.58

3.20

End of Borehole

Auger refusal on inferred bedrock at 3.20m depth

DEPTH (m): 0 — ELEV. 67.58
DEPTH 2 — ELEV. 65.58

Lower Explosive Limit % scale: 20 40 60 80

RKI Eagle Rdg. (ppm) scale: 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.91m	**FILE NO.** E1399
REMARKS Richmond Road at Island Park Drive	**HOLE NO.** BH11-3
BORINGS BY Truck-mount Drill **DATE** February 6, 1997	

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector
GROUND SURFACE						0	67.70	
Asphaltic concrete 0.10								
FILL: Grey crushed stone 0.23								
FILL: Brown sand-gravel						1	66.70	
1.27								
Dark brown, silty **TOPSOIL** 1.37		AU	26					△
GLACIAL TILL: Compact to dense, grey sand-gravel to silty sand with some gravel		SS	27	25	16	2	65.70	800 △
		SS	28	100	40			△
2.74								
End of Borehole								

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)
○ **Lower Explosive Limit %**
20   40   60   80

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Environmental Site Characterization
Former Service Stations
Ottawa, Ontario

DATUM	TBM - Top spindle of fire hydrant located on the north side of Richmond Road, approx. 25m west of west property line of subject site. Geodetic elevation = 68.81m
REMARKS	Richmond Road at Island Park Drive
BORINGS BY Truck-mount Drill	**DATE** February 6, 1997

FILE NO. E1399

HOLE NO. BH12-3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Photo Ionization Detector	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	

GROUND SURFACE

						0	67.68	
Asphaltic concrete — 0.10								
FILL: Grey crushed stone — 0.23								
FILL: Brown sand-gravel — 1.04						1	66.68	
Brown, sandy **TOPSOIL** — 1.19								
GLACIAL TILL: Dense, grey sand-gravel, some silt — 1.52		AU	29					○

End of Borehole

Auger refusal on inferred bedrock at 1.52m depth

Lower Explosive Limit %: 20 40 60 80

RKI Eagle Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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

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.

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

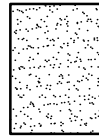|		
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

SYMBOLS AND TERMS (continued)

STRATA PLOT
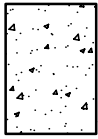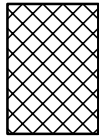
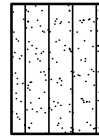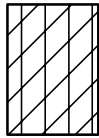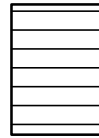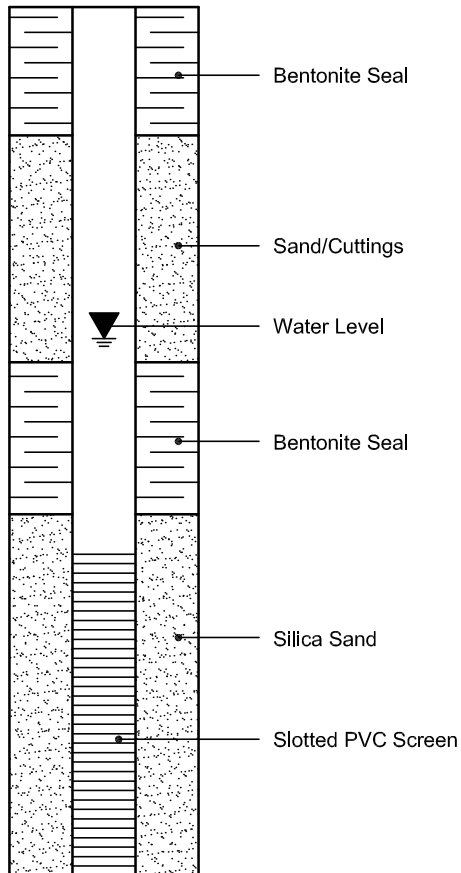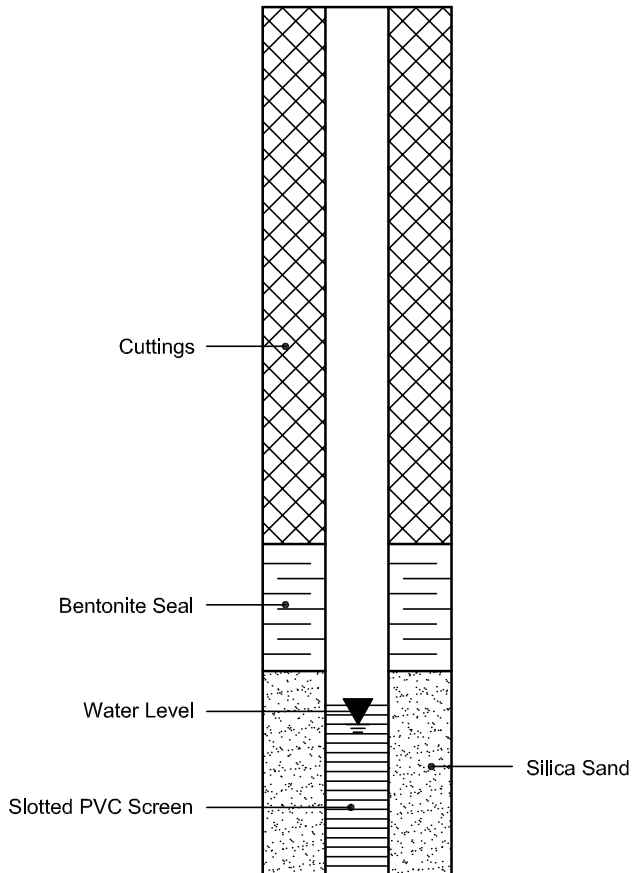|						
Topsoil	Asphalt	Fill	Peat	Sand	Silty Sand	Silt
Sandy Silt	Clay	Silty Clay	Clayey Silty Sand	Glacial Till	Shale	Bedrock
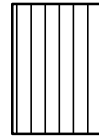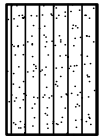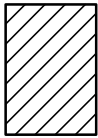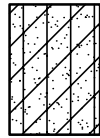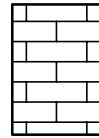
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION

- Bentonite Seal
- Sand/Cuttings
- Water Level
- Bentonite Seal
- Silica Sand
- Slotted PVC Screen

PIEZOMETER CONSTRUCTION

- Cuttings
- Bentonite Seal
- Water Level
- Slotted PVC Screen
- Silica Sand

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - GROUNDWATER SUPPRESSION AND FOUNDATION DRAINAGE SYSTEM

FIGURES 3 & 4 - SEISMIC SHEAR WAVE VELOCITY PROFILES

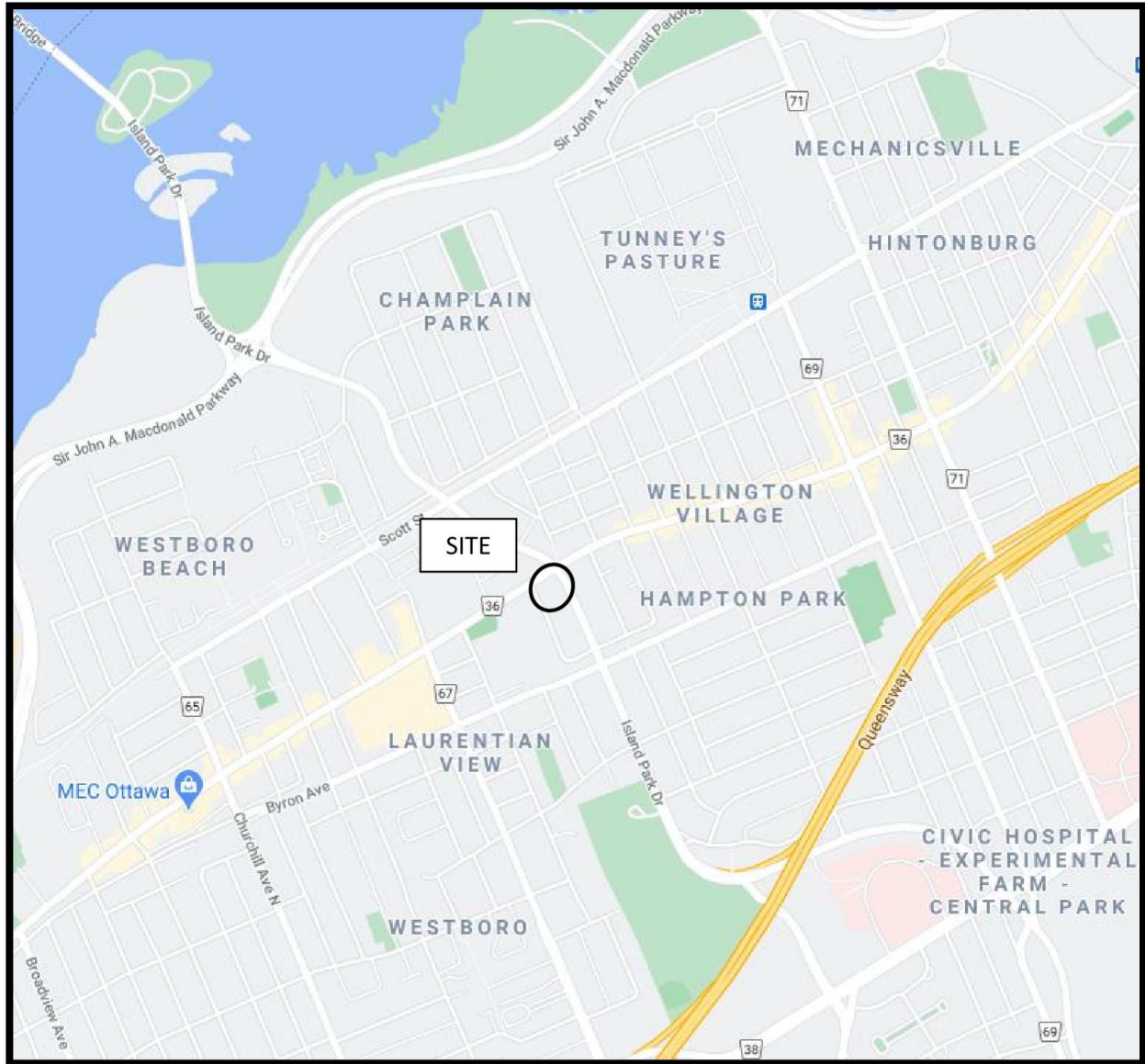**DRAWING PG5501-1 - TEST HOLE LOCATION PLAN**

FIGURE 1

KEY PLAN

patersongroup

IT IS RECOMMENDED THAT THE WATERPROOFING MEMBRANE AND/OR DRAINAGE BOARD BE INSTALLED ON THE INSIDE OF THE TEMPORARY FRAMEWORK IN THE DOUBLE SIDED CONCRETE WALL POUR TO ENSURE THE DRAINAGE BOARD MAINTAINS A FLAT AND VERTICAL INSTALL.

STABILIZED EARTH

DUE TO THE VARIABILITY OF THE GROUND CONDITIONS AT THE TOP OF THE TEMPORARY SHORING, DOUBLE-SIDED POURS MAY BE REQUIRED TO CARRY OUT FOUNDATION WALL CONSTRUCTION. IF REQUIRED, TEMPORARY FORMWORK CUT TO MATCH THE TOP OF THE SHORING SURFACE OUTLINE WILL BE REQUIRED TO SEAMLESSLY TRANSITION FROM THE BLIND SIDE TO DOUBLE SIDED FORMWORK TO MAINTAIN THE DESIGN THICKNESS OF THE FOUNDATION WALL AND TO ENSURE THAT THE FOUNDATION DRAINAGE AND/OR WATERPROOFING MEMBRANE IS INSTALLED IN A RELATIVELY FLAT AND VERTICAL FASHION.
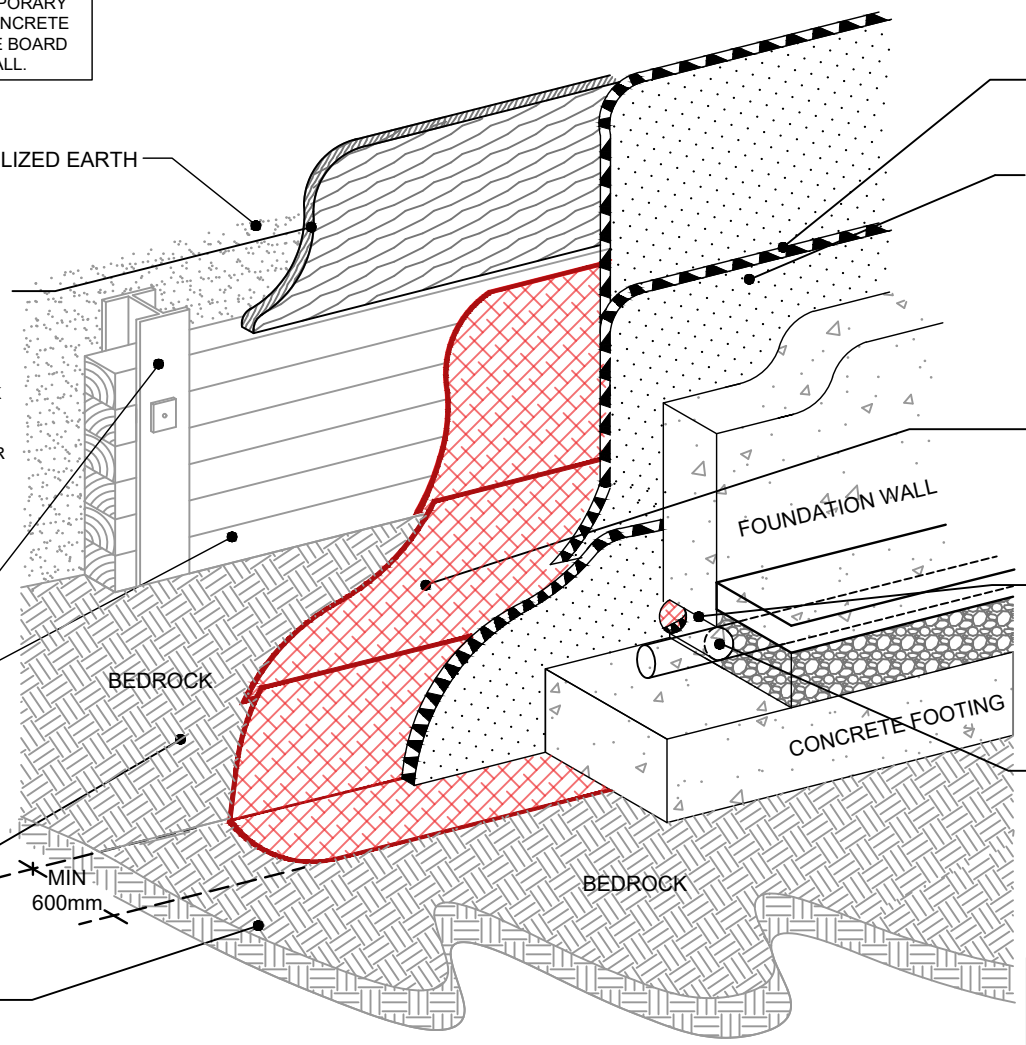
SOLDIER PILES

WOOD LAGGING

BEDROCK

SMOOTH AND FLAT TEMPORARY SHORING AND / OR BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

MIN 600mm

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE WATERPROOFING MEMBRANE

COMPOSITE FOUNDATION DRAINAGE BOARD 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO EXTEND FROM A MINIMUM HEIGHT OF 1.0m ABOVE THE LONG-TERM GROUNDWATER TABLE.

FOUNDATION WALL

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

CONCRETE FOOTING

BEDROCK

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE GEOTEXTILE LINED SIDE OF THE COMPOSITE FOUNDATION DRAINAGE BOARD AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM .

| **paterson**group	DEVTRIN (ISLAND PARK) INC.		Scale:	Date:
consulting engineers	**PROPOSED COMMERCIAL / RESIDENTIAL BUILDING**		N.T.S.	06/09/2021
	70 RICHMOND ROAD		Drawn by:	Report No.:
			YA	PG5501-1 REV.02
154 Colonnade Road South Ottawa, Ontario K2E 7J5 Tel: (613) 226-7381 Fax: (613) 226-6344 www.patersongroup.ca	OTTAWA, ONTARIO		Checked by:	Drawing No.:
	Title: **GROUNDWATER SUPPRESSION AND FOUNDATION DRAINAGE SYSTEM**		AB	**FIGURE 2**
			Approved by:	
			JV	Revision No.:
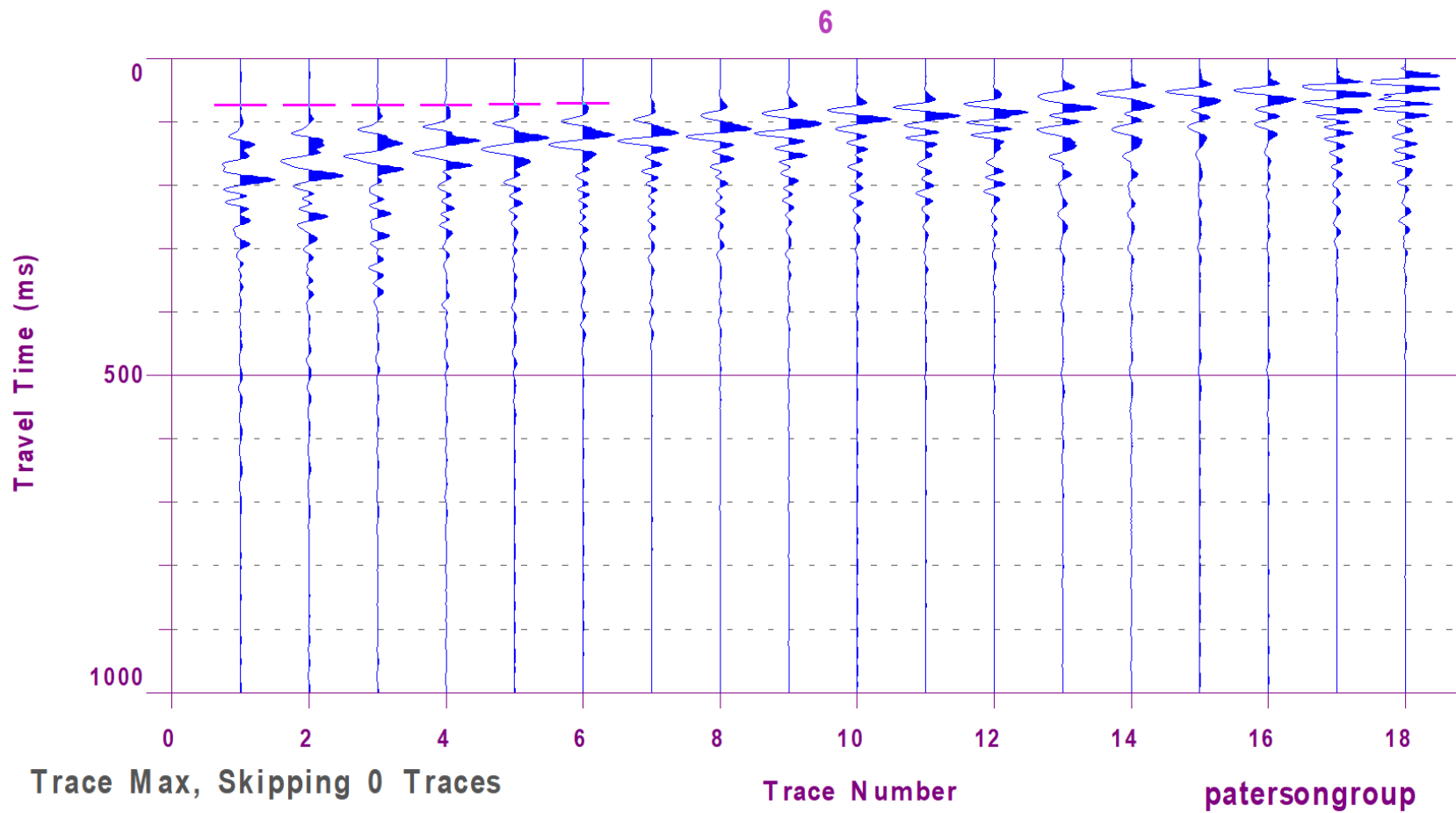
Figure 3 – Shear Wave Velocity Profile at Shot Location 18.5 m
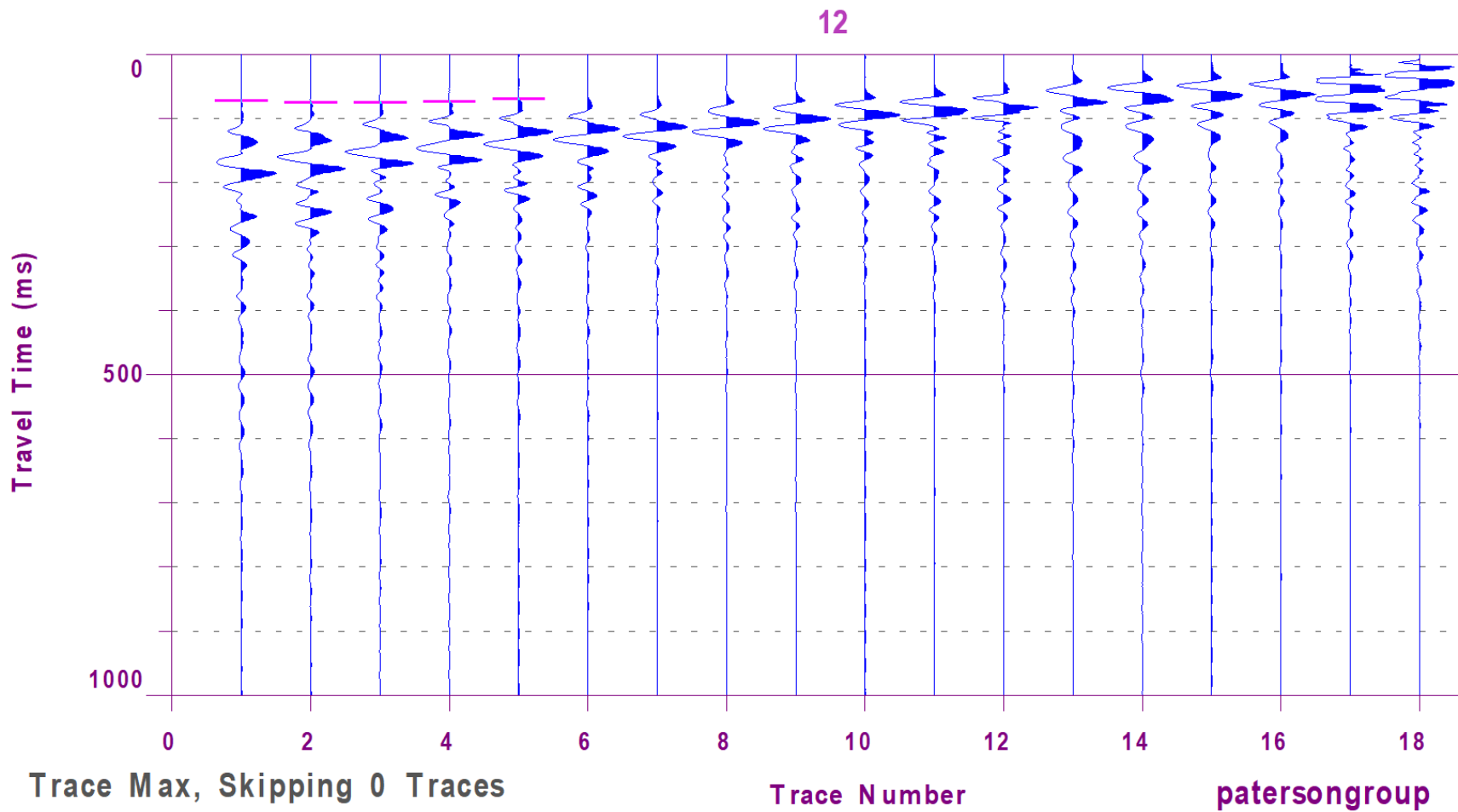
Figure 4 – Shear Wave Velocity Profile at Shot Location 18 m

ISLAND PARK DRIVE

SEISMIC SHEAR WAVE VELOCITY TEST LOCATION

1.0m TYP.

8.5

18.5 18.0

32.2

TERRACE LEV 6

TERRACE LEV 8

ROOF

(18) (16) (13) (10) (5) (1)

BH 1-22
68.19
[61.99]

BH 2-22
67.90
[62.11]

BH 12-3
67.68
(66.16)

BH 8-20
67.27
[61.15]

BH 3-3
67.69

BH 6-12
67.76
(62.17)

BH 2-12
67.12
(64.62)

BH 10-3
67.58
(64.38)

BH 11-3
67.70

BH 3-12
67.32
(64.75)

BH 9-3
67.17
(65.49)

BH 2-3
67.27
(65.37)

BH 9-20
67.20
[61.69]

**70 RICHMOND ROAD
9-STOREY
MIXED-USE RESIDENTIAL BLDG.**

ROOF AMENITIES

ROOF TERRACE

TERRACE LEV 9

TERRACE LEV 8

TERRACE LEV 6

TERRACE LEV 2

BH 5-12
67.80
(65.14)

RICHMOND ROAD

BH 7-3
67.50
(65.37)

BH 6-3
67.61
(65.73)

BH 5-3
67.70
(65.72)

MH HYDRO
T/G=66.83

TERRACE LEV 6

TERRACE LEV 8

ROOF TERRACE

BH 7-20
67.34
[61.34]

BH 1-12
67.49
(64.89)

BH 4-3
67.70
(65.82)

BH 8-3
67.57
(65.59)

BH 4-12
67.85

PATHWAY

MH
T/G = 67.02

LANEWAY
(HEATED LANEWAY FOR SNOW REMOVAL)

LEGEND:

- BOREHOLE WITH MONITORING WELL LOCATION
- BOREHOLE LOCATION WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE2637, 2012
- BOREHOLE LOCATION, PATERSON GROUP REPORT PE2637, 2012
- BOREHOLE LOCATION, PATERSON GROUP REPORT E1399, 1997
- GEOPHONE LOCATIONS
- (5) GEOPHONE NUMBER
- $+$ 8.5 SHOT LOCATION
- 67.80 GROUND SURFACE ELEVATION (m)
- [61.15] BEDROCK SURFACE ELEVATION (m)
- (65.14) PRACTICAL REFUSAL TO AUGERING ELEV. (m)

GROUND SURFACE ELEVATION AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:250

0 1 2 3 4 5   10   15m

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2022 BOREHOLES SHEAR WAVE VELOCITY TEST ADDED TO DRAWING LATEST CONCEPTUAL PLAN UPDATED TO DRAWING	25/04/2022	BN
1	UPDATE SITE BOUNDARIES (TO INCLUDE 376 ISLAND PARK DRIVE)	19/07/2021	JV

DEVTRIN (ISLAND PARK) INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
70 RICHMOND ROAD AND 376 ISLAND PARK DRIVE

OTTAWA,   ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:250	Date:	08/2020
Drawn by:	MPG	Report No.:	PG5501-1
Checked by:	JV	Dwg. No.:	
Approved by:	DJG	**PG5501-1** Revision No.: 2	

11x17

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