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Geotechnical Investigation

Proposed Multi-Storey Building
1619 and 1655 Carling Avenue
Ottawa, Ontario

Prepared For

Surface Developments

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Report PG5118-1, Revision 1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Surface Developments to prepare the current geotechnical report for a proposed multi-storey building to be located at 1619 and 1655 Carling Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current report are to:

- ❑ determine the subsurface soil and groundwater conditions by means of available subsoil information.
- ❑ provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was carried out as a separate program and is reported under separate cover.

2.0 Proposed Development

It is understood that the development will consist of 5 storey podium base located between a 18 storey structure located within the west portion of the site and a 16 storey structure located within the east portion of the site. It is further understood that the development will include 3 levels of underground parking, which will encompass the entire site.

It is further understood that the proposed buildings will be municipally serviced with sewer and water.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The most recent field program for the subsoil investigation was carried our on August 17, 2020 which consisted of extended a total of four (4) boreholes within the east portion of the site (PE4987, 2020). The previous field investigation was carried out in October, 2008. At that time, a total of six (6) boreholes and fifteen (15) test pits were completed across the subject site to provided general coverage of the proposed development (PE1472, 2008). Also, two (2) boreholes were completed in September, 1994 as part of the previous subsoil investigation conducted by Paterson Group (E1116, 1994).

All available test holes completed by others at the subject site are presented in Appendix 2. The locations of the test holes are presented on Drawing PG5118-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted drill rig operated by a two-person crew while the test pits were excavated using a rubber tired back-hoe. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling and excavation procedure consisted of drilling or excavating to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split spoon sampler. Soil samples were also recovered along the sidewalls of the test pits by hand during excavation.

All soil samples were visually inspected and initially classified on site. The split spoon and auger samples were placed in sealed plastic bags. All rock core was classified on site and placed into a core boxes. All samples were transported to our laboratory for examination and classification. The depths at which the split-spoon, auger and rock core samples were recovered from the boreholes are shown as SS, AU and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

A recovery value and a Rock Quality Designation (RQD) value were calculated for each core run of bedrock and are presented on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled rock core. The RQD value is the ratio, in percentage, of core length greater than 100 mm over the total core run length.

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.

Groundwater

Monitoring wells were installed in four boreholes during the latest subsoil investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 1.5 to 3 m long slotted 32 mm diameter PVC screen sealed at strategic depths.
- 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen to approximately 300 mm from the ground surface.
- Clean backfill from top of bentonite plug to the ground surface.

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

Sample Storage

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

3.2 Field Survey

The ground surface elevation at each test hole location completed during the most recent subsoil investigation in August, 2020 (PE4987, 2020) was referenced to the top of grate of the catch basin manhole located near the northeast corner of the subject site. A geodetic elevation of 77.35 m was assigned to the TBM on the plan prepared by Farley, Smith & Denis Ltd..

The ground surface elevation at each test hole location completed during the previous subsoil investigation in October, 2008 (PE1472, 2008) was referenced to top of the manhole cover located in front of the subject site. A geodetic elevation of 77.51 m was assigned to the TBM on the plan prepared by Farley, Smith & Denis Ltd..

Lastly, the ground surface elevation at each borehole location completed during the initial subsoil investigation completed by Paterson Group in September, 1994 (E1116, 1994) was referenced to the top of spindle of the fire hydrant located at the southwest corner of the property. A geodetic elevation of 78.44 m was assigned to the TBM on the plan prepared by Farley, Smith & Denis Ltd.

The location of the TBMs, test holes and the ground surface elevation at each test hole location are presented in Drawing PG5118-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The west portion of the subject site is currently used as a commercial parking lot with mature trees bordering the north and west property boundaries. A two (2) storey commercial building occupies the east portion of the subject site.

The site is bordered to the north by low-rise apartment buildings, to the west by a commercial plaza and to the east by Shell Station. The site is approximately at grade with the neighboring properties and approximately at grade with Carling Avenue bordering the south property boundary.

It should be noted that the west portion of the site was historically occupied by several buildings. Based on the available information, the former building was centrally located within the west portion of the site and the building was constructed with a basement level.

4.2 Subsurface Profile

Overburden

Generally, the subsoil conditions encountered at the test hole locations consists of a pavement structure overlying a variety of fill material which in turn was found to be overlying a glacial till and/or bedrock surface.

All test holes completed during our subsoil investigations were terminated at depths varying 0.9 and 2.8 m below existing ground surface on a grey limestone bedrock.

Bedrock

Bedrock was cored at 8 borehole locations to a maximum depth of 6.7 m below existing ground surface. The recovery values and RQD values for the bedrock cores were calculated. The recovery values range from 82 to 100%, while the RQD values generally vary between 30 and 100%. Based on these results the quality of the bedrock ranges from poor to excellent.

Based on available geological mapping, the drift thickness in the area ranges in thickness between 2 and 3 m in an area where the bedrock consists of interbedded limestone, dolostone and shale of the Gull River Formation.

4.3 Groundwater

Groundwater levels (GWL) were measured in the monitoring wells on August 23, 2020 (PE4987, 2020) and on November 3, 2008 (PE1472, 2008) as part of the previous subsoil and groundwater investigation. The measured GWL readings are summarized in Table 1 and Table 2 below.

Table 1 (PE4987, 2020) - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-20	77.59	2.23	75.36	August 25, 2020
BH 2-20	77.44	2.69	74.75	August 25, 2020
BH 3-20	77.50	2.06	75.44	August 25, 2020
BH 4-20	77.53	1.71	75.82	August 25, 2020

Note: The ground surface elevation at each test hole location completed during the subsoil investigation in August, 2020 was referenced to the top of grate of catch basin manhole near the northeast corner of the subject site. A geodetic elevation of 77.35 m was assigned to the TBM on the plan prepared by Farley, Smith & Denis Ltd..

Table 2 (PE1472, 2008) - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	77.63	2.50	75.13	November 3, 2008
BH 2	77.72	1.90	75.82	November 3, 2008
BH 3	77.58	2.45	75.13	November 3, 2008
BH 4	77.61	2.91	74.70	November 3, 2008

Note: The ground surface elevation at each test hole location completed during the subsoil investigation in October, 2008 was referenced to top of the manhole cover located in front of the subject site. A geodetic elevation of 77.51 m was assigned to the TBM on the plan prepared by Farley, Smith & Denis Ltd..

Based on our general knowledge of the areas geology, experience with similar development projects in the immediate area, it is expected that the long-term groundwater is located approximately 2 to 3 m below existing ground surface. However, it should be noted that perched water may be encountered within the upper fractured bedrock and at the bedrock/overburden interface which may lead to an initial high infiltration for building excavation.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable from a geotechnical perspective for the proposed development. It is anticipated that the proposed multi-storey buildings with 3 levels of underground parking will be founded on shallow footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground parking levels. Hoe ramming is an option where only small quantities of bedrock need to be removed. 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 further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow depth of the bedrock surface at the subject site and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

Based on the volume of 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 blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be 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 also an experienced blasting consultant.

Excavation side slopes in sound bedrock can 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 to allow for potential sloughing or to provide a stable base for the overburden shoring system. Depending on the design of the shoring system, the final requirement of the bedrock shelf width will be determined.

Vibration Considerations

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

The following construction equipments could be the source of 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 the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, 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.

Weathered Bedrock Face Reinforcement

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage. In addition to rock anchors, a bedrock face reinforcement system, such as a chain link fence connected to the bedrock excavation face, may be required where weathered bedrock is exposed and support is required to create a safe excavation area for workers. Evaluation of a bedrock reinforcement system can be evaluated at the time of excavation by the geotechnical consultant.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at a minimum compacted by the heavy equipment tracks to minimize voids. If these materials are to build up the subgrade level for areas to be paved, the material should be compacted in thin lifts to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, limestone bedrock is expected to be encountered at the founding level.

A factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be provided if founded on limestone bedrock which is free of seams, fractures and voids immediately within the founding level.

Settlement

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

5.4 Design for Earthquakes

The site class for seismic site response is a **Class A** for the proposed building. The soil underlying the subject site are not susceptible to liquefaction. A seismic site classification of Class A will require a site specific shear wave velocity testing to be completed to confirm this seismic classification. Refer to the most recent revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and the recommended rigid pavement structure noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 300 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Wall

It is understood that the basement walls are to be poured against a water suppression system, which will be placed against the temporary shoring system and 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 limestone bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. 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 individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should 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) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid 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 servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and an free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops 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 sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 75 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, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 69** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Lengths

Parameters used to calculate grouted rock anchor lengths are provided in Table 3.

Table 3 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	75 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 4. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	1.5	1.0	2.5	450
	1.8	1.2	3.0	600
	2.1	1.4	3.5	750
	2.9	1.6	4.5	900
125	1.2	0.8	2.0	450
	1.5	1.0	2.5	600
	1.6	1.2	2.8	750
	2.2	1.0	3.2	900

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, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 5 below. The flexible pavement structure presented in Table 6 should be used for at grade access lanes and heavy loading parking areas overlying the podium deck.

Table 5 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type **I** or **II** material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 6 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type **II**
SUBGRADE - OPSS Granular B Type **II** overlying the Concrete Podium Deck.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

It is understood that the proposed structure will occupy the entire boundary of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a foundation drainage/waterproofing system placed directly against the temporary shoring system and a suitably prepared bedrock surface. It is suggested that this system should be constructed as follows (refer to Figure 2 - Water Suppression System in Appendix 2 for an illustration of this system cross-section):

- The concrete mud slab will create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation within 10 m of the east property boundary and will consist of a 75 mm thick layer of 25 MPa compressive strength concrete.
- Temporary shoring system and vertical surface should be prepared to receive the proposed foundation drainage and waterproofing membranes for the underground parking structure. The bedrock surface will be prepared by grinding or using shotcrete to smooth out angular sections depending on the manufacturer's requirements of the proposed waterproofing membrane.
- A waterproofing membrane will be applied to the temporary shoring system and prepared vertical bedrock surface from 2 m below grade to the founding elevation. The membrane will serve as a water infiltration suppression system. The membrane will also be placed along the horizontal surface beneath the perimeter footings to provide a better seal at the vertical and horizontal interface. To further reduce the volume of groundwater from the adjacent property to the east, it is recommended to double the bentonite waterproofing membrane along the east property boundary and extending a minimum of 10 m west along the north and south property boundaries.
- A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 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 sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier (minimum 100 mm thick concrete mud slab). For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed in each bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane was recommended to lessen the effects of water infiltration. Any minor dewatering of the site will be within the bedrock layer which is considered relatively shallow at the subject site. Therefore, no adverse effects to the surrounding buildings or properties are expected with the lowering of the groundwater in this 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 required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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.

It is expected that the parking garage will not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may 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

Excavation Side Slopes

The side slopes of the shallow excavations anticipated should either be cut back at acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The subsurface soil at this site 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 be kept away from the excavation sides.

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

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 will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

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

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

Table 7 - 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

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

Underpinning of Adjacent Structures

Based on the relatively shallow depth of the bedrock at the subject site, it is expected that the building neighbouring structure located near the east property boundary of the site is most likely founded on the bedrock surface. Therefore, underpinning is not expected to be required for this project.

6.4 Pipe Bedding and Backfill

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

6.5 Groundwater Control

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 excavation.

Permit to Take Water

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

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

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, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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

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

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects 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.
- ❏ Review proposed waterproofing and foundation drainage design and requirements.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of limitations

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

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

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Surface Developments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Richard Groniger, C. Tech.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
December 1, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Report Distribution

- ❑ Surface Developments (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TEST HOLE LOGS BY OTHERS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1619 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northeast corner of subject site. Geodetic elevation = 77.35m.

REMARKS

BORINGS BY Track-Mouont Power Auger

DATE August 17, 2020

FILE NO. PE4987

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.59
Asphaltic concrete 0.10							
FILL: Brown silty sand with crushed stone		AU	1				
		SS	2	17	13	1	76.59
1.42							
BEDROCK: Poor to excellent quality, grey limestone		RC	1	100	49	2	75.59
		RC	2	100	67	3	74.59
						4	73.59
		RC	3	100	100	5	72.59
5.64							
End of Borehole							
(GWL @ 2.71m - Nov. 17, 2020)							

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1619 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northeast corner of subject site. Geodetic elevation = 77.35m.

REMARKS

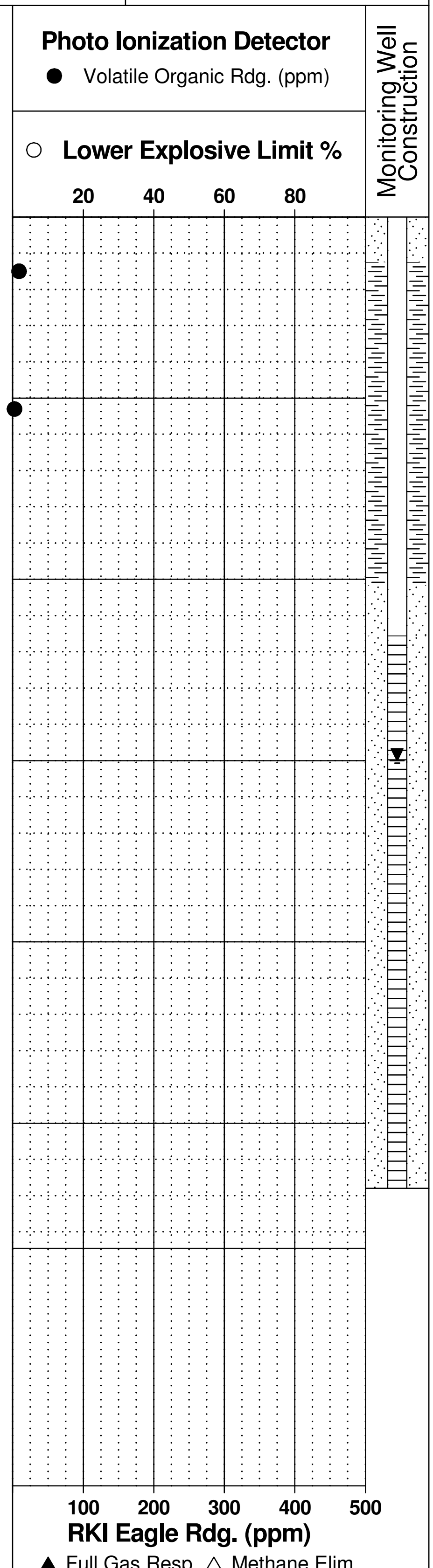
BORINGS BY Track-Mouont Power Auger

DATE August 17, 2020

FILE NO. PE4987

HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.44
Asphaltic concrete (0.10)							
FILL: Brown silty sand with crushed stone (0.59)		AU	1				
FILL: Brown silty sand, some gravel and crushed stone (1.55)		SS	2	38	8	1	76.44
BEDROCK: Fair to excellent quality, grey limestone (5.69)		RC	1	100	65	2	75.44
						3	74.44
		RC	2	82	38	4	73.44
		RC	3	100	100	5	72.44
End of Borehole							
(GWL @ 3.00m - Nov. 17, 2020)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1619 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northeast corner of subject site. Geodetic elevation = 77.35m.

REMARKS

BORINGS BY Track-Mouont Power Auger

DATE August 17, 2020

FILE NO. PE4987

HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand with crushed stone	0.66	AU	1		
FILL: Brown silty sand, trace organics	1.58	SS	2	46	9
BEDROCK: Fair to excellent quality, grey limestone	5.51	RC	1	100	70
		RC	2	100	58
		RC	3	100	100
End of Borehole					
(GWL @ 2.45m - Nov. 17, 2020)					

DEPTH (m)	ELEV. (m)
0	77.50
1	76.50
2	75.50
3	74.50
4	73.50
5	72.50

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

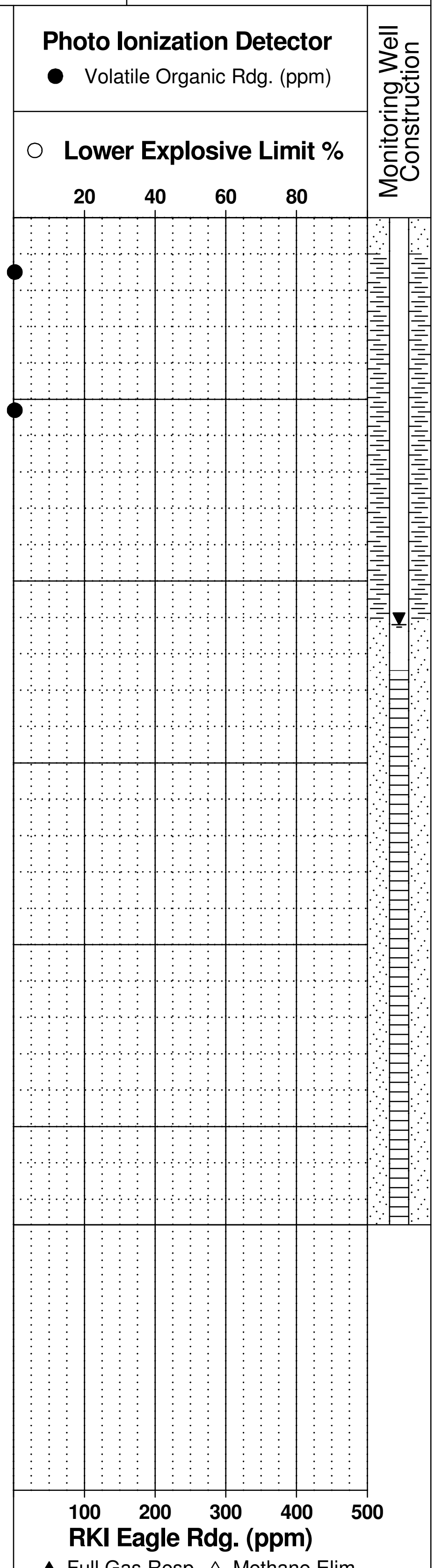
SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1619 Carling Avenue
Ottawa, Ontario

DATUM	TBM - Top of grate of catch basin located near the northeast corner of subject site. Geodetic elevation = 77.35m.
REMARKS	
BORINGS BY	Track-Mouont Power Auger
DATE	August 17, 2020
FILE NO.	PE4987
HOLE NO.	BH 4-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Brown silty sand with crushed stone	0.61	AU	1		
FILL: Brown silty sand with gravel and crushed stone	1.65	SS	2	17	20
BEDROCK: Poor to excellent quality, grey limestone	5.54	RC	1	100	29
		RC	2	100	66
		RC	3	100	100
End of Borehole					
(GWL @ 2.24m - Nov. 17, 2020)					

DEPTH (m)	ELEV. (m)
0	77.53
1	76.53
2	75.53
3	74.53
4	73.53
5	72.53



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. **PE1472**

REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 75 Power Auger

DATE October 31, 2008

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.20	AU	1		
FILL: Sand and gravel	0.51				
FILL: Dark brown, mixture of clay, silt, sand, gravel and brick fragments	2.80	SS	2	17	5
		SS	3	25	5
		SS	4	38	50+
BEDROCK: Grey limestone	6.17	RC	1	98	88
		RC	2	98	82
		RC	3	100	100
End of Borehole					
(GWL @ 2.50m-Nov. 3/08)					

DEPTH (m)	ELEV. (m)
0	77.63
1	76.63
2	75.63
3	74.63
4	73.63
5	72.63
6	71.63

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. BH 2

BORINGS BY CME 75 Power Auger **DATE** October 31, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.72
FILL: 25mm Asphaltic concrete over dark brown silty sand with gravel and asphalt		AU	1				
		SS	2	8	9	1	76.72
1.98		SS	3	17	10		
BEDROCK: Grey limestone						2	75.72
		RC	1	100	73		
						3	74.72
		RC	2	89	30		
						4	73.72
		RC	3	98	87		
						5	72.72
6.12		RC	4	100	78	6	71.72
End of Borehole							
(GWL @ 1.90m-Nov. 3/08)							

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

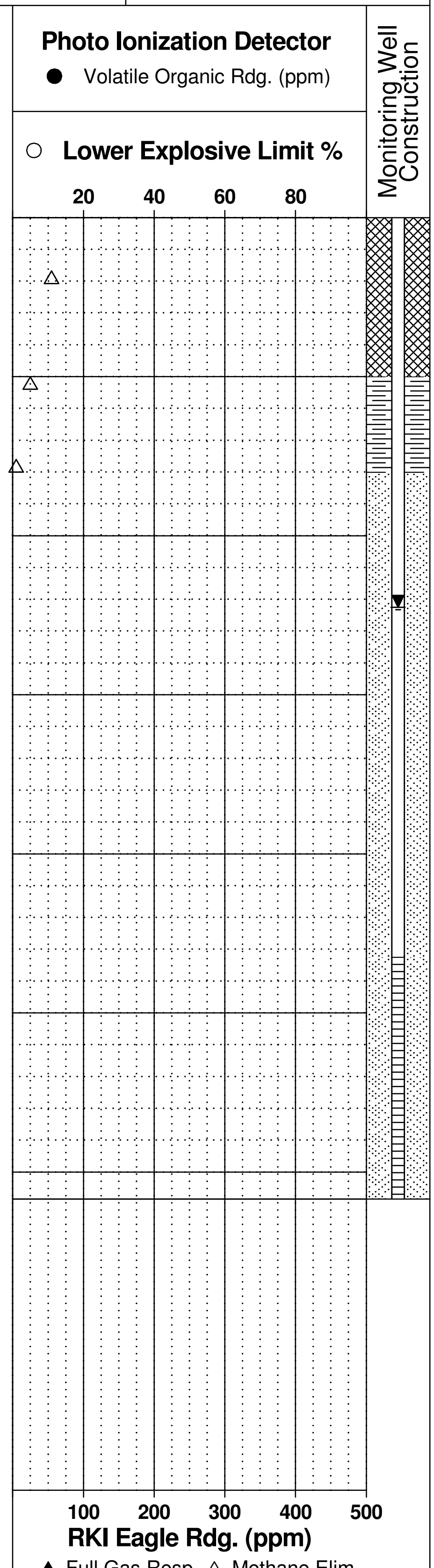
HOLE NO. BH 3

BORINGS BY CME 75 Power Auger

DATE October 31, 2008

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.25	AU	1		
FILL: Sand and gravel	0.56				
FILL: Brown sand	0.66				
GLACIAL TILL: Brown silty sand with gravel, cobbles	1.22	SS	2	36	14
BEDROCK: Grey limestone	6.17	SS	3	67	50+
		RC	1	97	69
		RC	2	95	82
		RC	3	99	70
		RC	4	100	100
End of Borehole					
(GWL @ 2.45m-Nov. 3/08)					

DEPTH (m)	ELEV. (m)
0	77.58
1	76.58
2	75.58
3	74.58
4	73.58
5	72.58
6	71.58



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SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

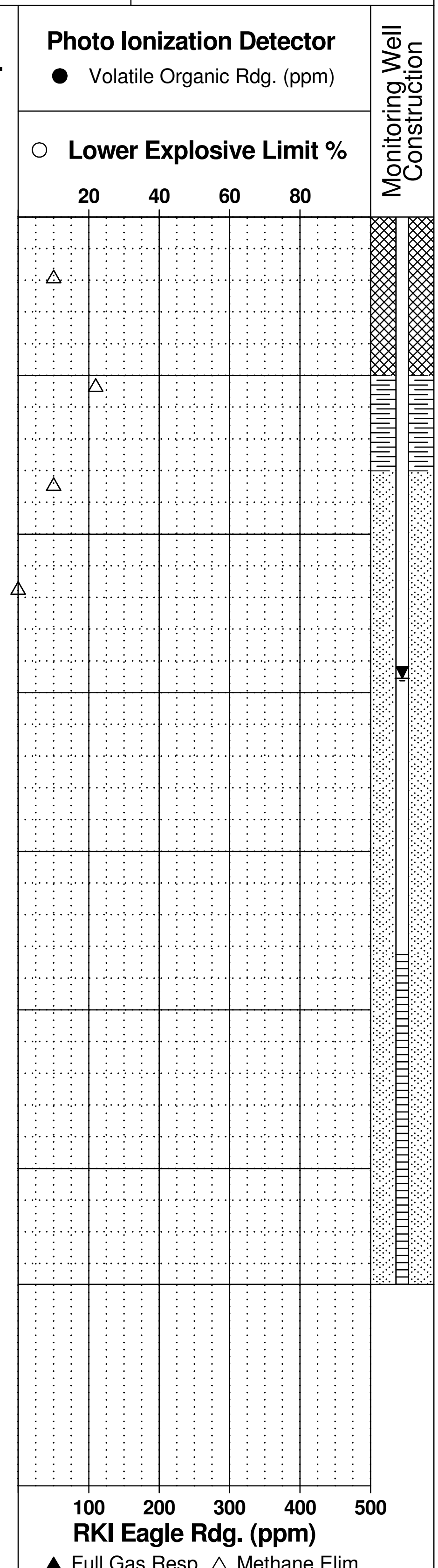
REMARKS

HOLE NO. BH 4

BORINGS BY CME 75 Power Auger

DATE October 31, 2008

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.61
FILL: 38mm Asphaltic concrete over crushed stone	0.20						
FILL: Sand and gravel	0.51	AU	1				
FILL: Brown silty clay, some sand and gravel	1.47	SS	2		7	1	76.61
GLACIAL TILL: Brown silty sand with gravel, cobbles	1.68	SS	3		50+	2	75.61
BEDROCK: Grey limestone	6.73	SS	4		50+		
		RC	1	100		3	74.61
		RC	2	100	92	4	73.61
		RC	3	98	80	5	72.61
		RC	4	97	68	6	71.61
End of Borehole							
(GWL @ 2.91m-Nov. 3/08)							



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SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

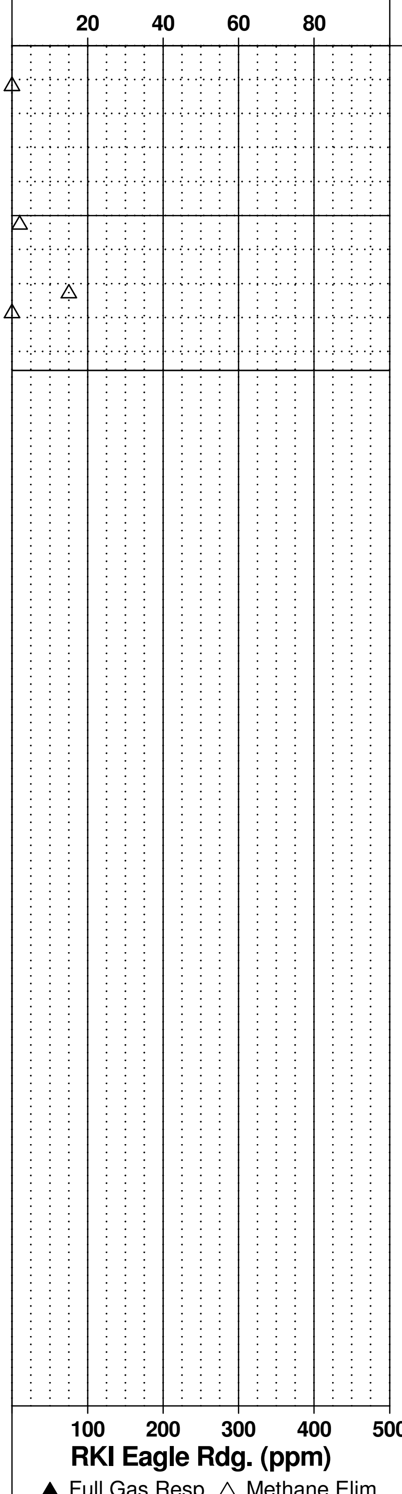
FILE NO. PE1472

REMARKS

HOLE NO. BH 5

BORINGS BY CME 75 Power Auger **DATE** October 31, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	77.64		
FILL: Sand and gravel		AU	1						
		SS	2	14	6	1	76.64		
1.22									
FILL: Brown silty sand, trace clay and gravel		AU	4						
		SS	3	33	50+				
1.91									
End of Borehole Practical refusal to augering @ 1.91m depth									



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SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. BH 6

BORINGS BY CME 75 Power Auger

DATE October 31, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.50
FILL: 25mm Asphaltic concrete over sand and gravel	▨	AU	1				
0.66							
FILL: Brown silty sand, trace organic matter	▨	SS	2	33	6	1	76.50
	▨	SS	3		8		
2.11						2	75.50
End of Borehole Practical refusal to augering @ 2.11m 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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 1

BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	Depth to base (m)	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	77.54		
Asphaltic concrete	0.06									
FILL: Crushed stone	0.15									
FILL: Sand and gravel	0.26									
FILL: Brown silty sand	0.50		G	1					△	
GLACIAL TILL: Brown silty sand, some clay and gravel	0.90		G	2					△	
Weathered **BEDROCK**	1.15						1	76.54		
End of Test Pit										
(TP dry upon completion)										

Lower Explosive Limit % scale: 20, 40, 60, 80

RKI Eagle Rdg. (ppm) scale: 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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 2

BORINGS BY Backhoe **DATE** October 20, 2008

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 % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	77.56		
Asphaltic concrete 0.10									
FILL: Crushed stone 0.55									
FILL: Brown silty sand 0.80		G	1					△	
						1	76.56		
FILL: Brown silty clay, some sand and gravel									
		G	2			2	75.56	△	
2.25									
End of Test Pit TP terminated on bedrock surface @ 2.25m depth (TP dry upon completion)									

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
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Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 3

BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	77.53		
Asphaltic concrete 0.12									
FILL: Sand and gravel 0.36									
GLACIAL TILL: Brown silty sand, trace clay and gravel		G	1						
		G	2						
1.10						1	76.53		
Weathered **BEDROCK** 1.25									
End of Test Pit									
(TP dry upon completion)									

Chart scale: 20 40 60 80 (top); 100 200 300 400 500 (bottom)

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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

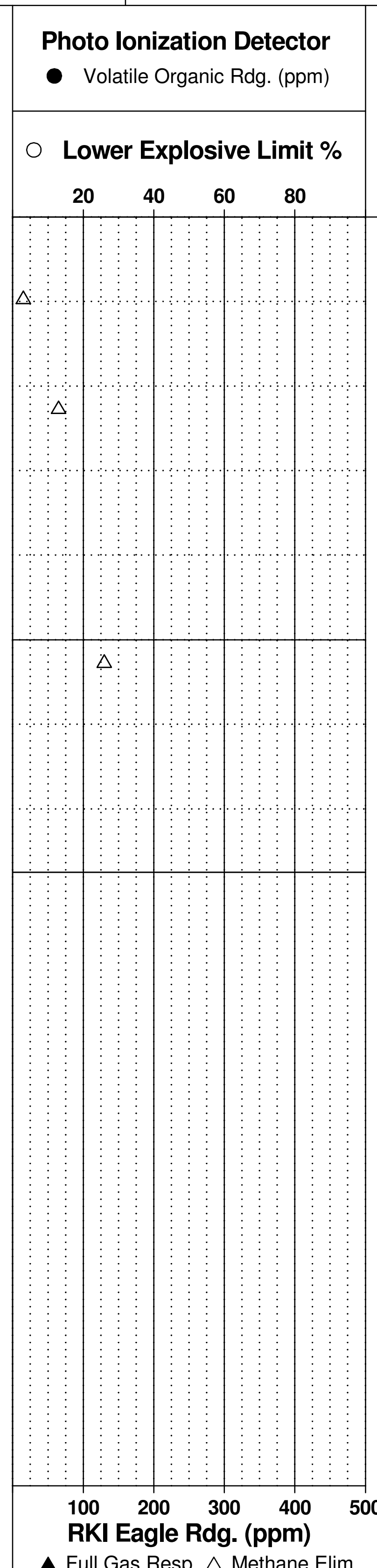
REMARKS

HOLE NO. TP 4

BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.64
Asphaltic concrete	0.05						
FILL: Sand and gravel	0.15						
FILL: Brown sand and gravel, trace cobbles and boulders	0.90	G	1				
		G	2				
Dark brown **SILTY SAND**, trace topsoil	1.20	G	3			1	76.64
Weathered **BEDROCK**	1.55						
End of Test Pit							
(TP dry upon completion)							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 5

BORINGS BY Backhoe **DATE** October 20, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	77.60		
Asphaltic concrete 0.08									
FILL: Sand and gravel 0.35									
FILL: Brown silty sand 0.57		G	1					△	
FILL: Brown sandy silt, trace clay and gravel		G	2					△	
						1	76.60		
1.25		G	3					△	
End of Test Pit TP terminated on bedrock surface @ 1.25m depth (TP dry upon completion)									

Upper scale: 20 40 60 80

Lower scale: 100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 6

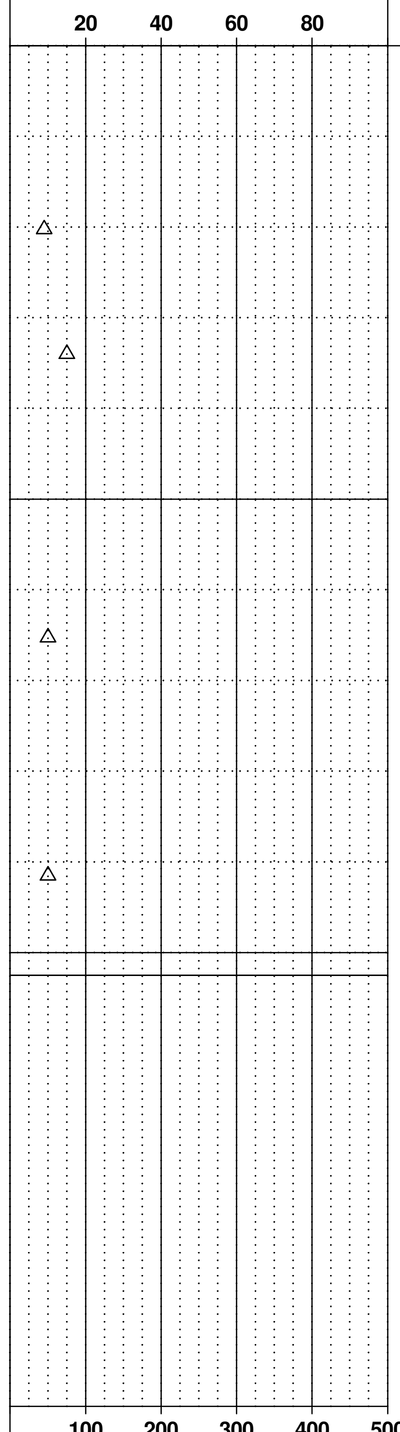
BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.56
Asphaltic concrete — 0.08							
FILL: Sand and gravel — 0.26							
FILL: Brown silty sand, piece of shingle — 0.60		G	1				
FILL: Clayey silt, trace topsoil — 1.15		G	2			1	76.56
GLACIAL TILL: Brown clayey silt, trace sand and gravel — 1.90		G	3				
		G	4				
Weathered **BEDROCK** — 2.05						2	75.56
End of Test Pit							
(TP dry upon completion)							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**



RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

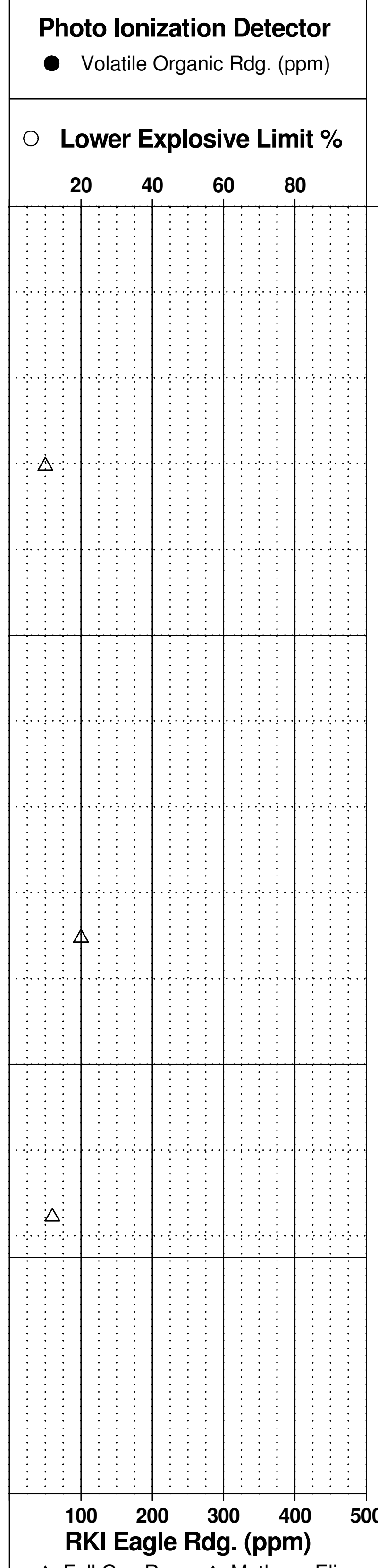
HOLE NO. TP 7

BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06				
FILL: Sand and gravel	0.45				
FILL: Silty sand, trace gravel, cinders blocks, rotten wood	2.45	G	1		
		G	2		
		G	3		
End of Test Pit TP terminated on bedrock surface @ 2.45m depth					

DEPTH (m)	ELEV. (m)
0	77.67
1	76.67
2	75.67



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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP 8

BORINGS BY Backhoe

DATE October 20, 2008

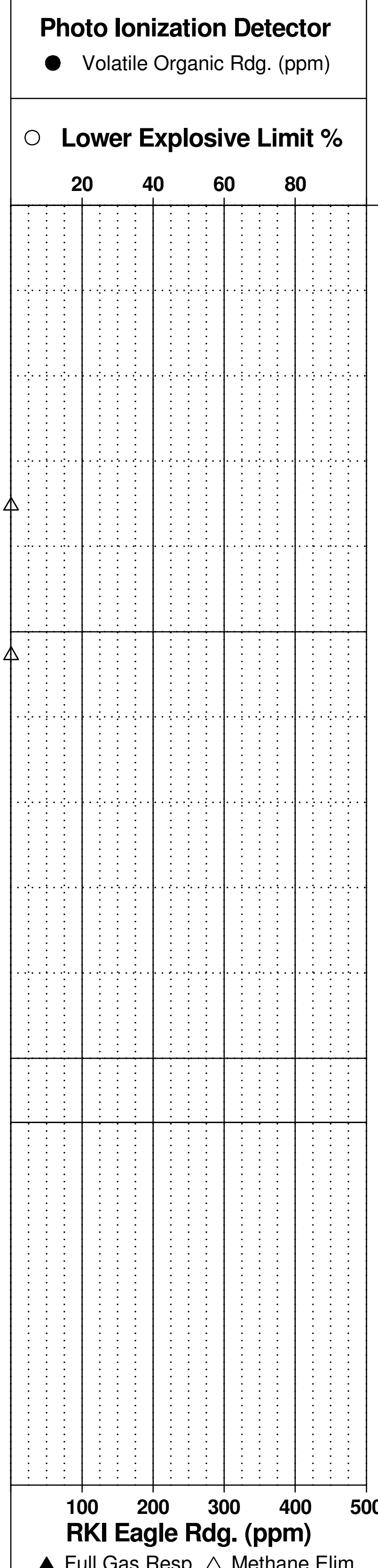
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Sand and gravel	0.20				
FILL: Crushed stone	0.30				
FILL: Sand and gravel	2.15	G	1		
		G	2		

DEPTH (m)	ELEV. (m)
0	77.69
1	76.69
2	75.69

End of Test Pit

Cinder block foundation wall on east side of test pit.

(TP dry upon completion)



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

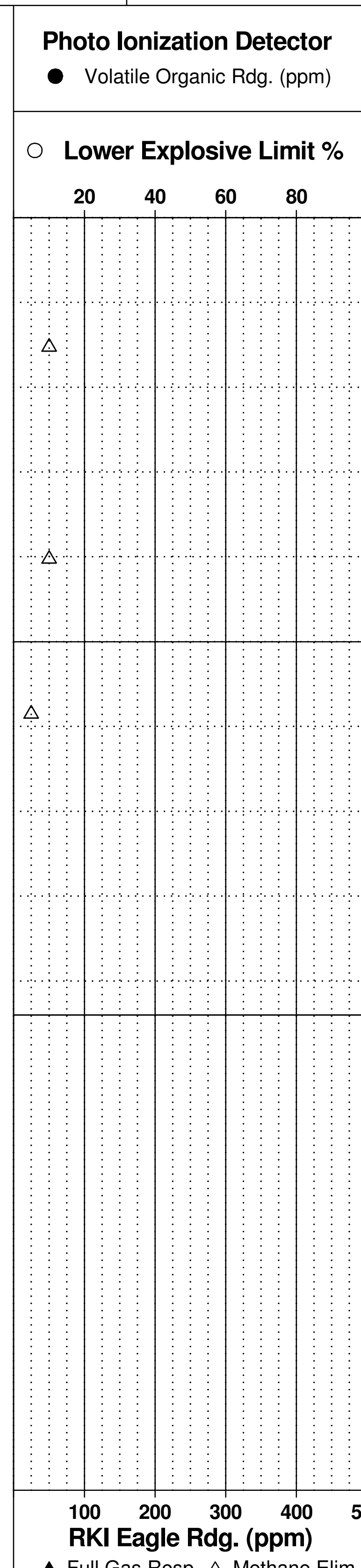
REMARKS

HOLE NO. TP 9

BORINGS BY Backhoe

DATE October 20, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.37
Asphaltic concrete — 0.10							
FILL: Sand and gravel — 0.40		G	1				
FILL: Brown silty sand with brick, wire, rope, plastic, pipe and concrete pieces		G	2				
						1	76.37
		G	3				
— 1.88							
End of Test Pit (TP dyr upon completion)							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

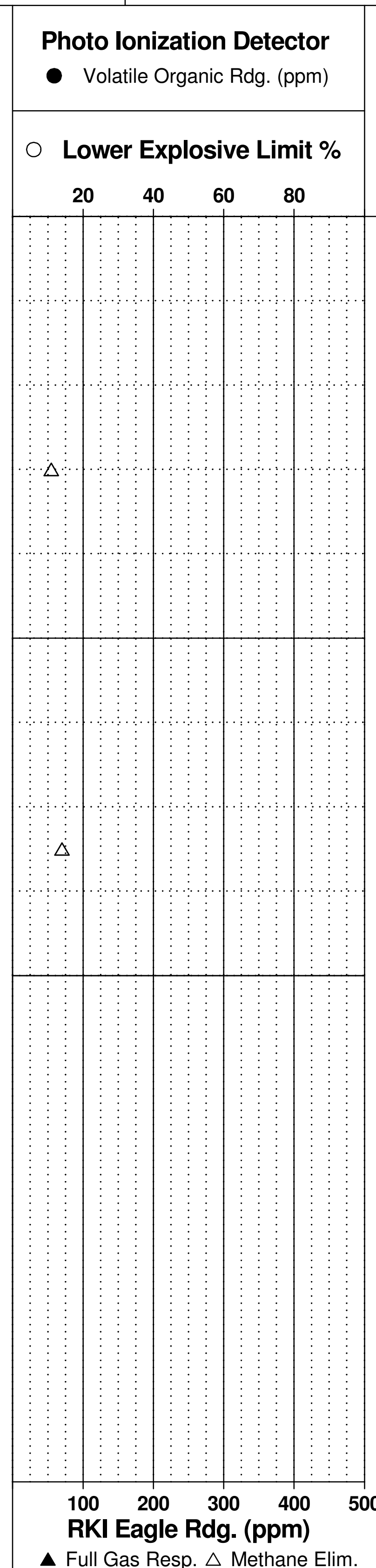
REMARKS

HOLE NO. TP10

BORINGS BY Backhoe

DATE October 21, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.66
FILL: Sand and gravel							
0.43							
FILL: Brown silty sand, some clay and gravel		G	1				
						1	76.66
		G	2				
1.80							
End of Test Pit TP terminated on bedrock surface @ 1.80m depth (TP dry upon completion)							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

REMARKS

BORINGS BY Backhoe

DATE October 21, 2008

FILE NO. PE1472

HOLE NO. TP11

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	77.61		
Asphaltic concrete 0.07									
FILL: Sand and gravel 0.30									
FILL: Dark brown to light brown silty sand 0.70		G	1						
		G	2						
FILL: Brown clayey silt, trace gravel, topsoil 1.75		G	3			1	76.61		
		G	4						
		G	5						
End of Test Pit									

TP terminated on bedrock surface @ 1.75m depth

(TP dry upon completion)

Chart scale: 20 40 60 80 (top); 100 200 300 400 500 (bottom)

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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP12

BORINGS BY Backhoe

DATE October 21, 2008

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 % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	77.67		
Asphaltic concrete — 0.04									
FILL: Sand and gravel — 0.30									
		G	1					△	
FILL: Brown silty clay, trace sand and gravel						1	76.67		
		G	2					△	
1.80									
End of Test Pit TP terminated on bedrock surface @ 1.80m depth (TP dry upon completion)									

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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP13

BORINGS BY Backhoe

DATE October 21, 2008

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	77.64		
Asphaltic concrete 0.04									
FILL: Sand and gravel 0.16									
FILL: Brown silty clay, some sand, trace gravel		G	1					△ 20	
						1	76.64		
1.73		G	2					△ 20	
End of Test Pit TP terminated on bedrock surface @ 1.73m depth Cinder block foundation wall and footing on west side of test pit									

Lower Explosive Limit % scale: 20, 40, 60, 80

RKI Eagle Rdg. (ppm) scale: 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
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP14

BORINGS BY Backhoe

DATE October 21, 2008

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	77.66		
Asphaltic concrete 0.15									
FILL: Sand and gravel 0.40									
FLL: Brown silty sand, some clay, trace gravel 0.88		G	1					△	
		G	3					△	
						1	76.66		
FILL: Brown silty clay with sand and gravel 2.30		G	2					△	
						2	75.66		
		G	4					△	
End of Test Pit TP terminated on bedrock surface @ 2.30m depth (TP dry upon completion)									

Upper scale: 20, 40, 60, 80

Lower scale: 100, 200, 300, 400, 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Phase II-Environmental Site Assessment
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top of manhole cover, located in front of subject site. Elevation = 77.51m.

FILE NO. PE1472

REMARKS

HOLE NO. TP15

BORINGS BY Backhoe

DATE October 21, 2008

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.58
Asphaltic concrete 0.12							
FILL: Sand and gravel 0.22							
FILL: Brown silty sand, trace gravel 0.80		G	1				
						1	76.58
FILL: Brown silty clay, some sand and gravel		G	2				
						2	75.58
		G	3				
2.62		G	4				
End of Test Pit TP terminated on bedrock surface @ 2.62m depth (Water infiltration @ 2.4m 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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I and II Environmental Investigation
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant at southwest corner of property. Geodetic elevation = 78.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE September 21, 1994

FILE NO. E1116

HOLE NO. BH 1

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 % (20, 40, 60, 80) / RKI Eagle Rdg. (ppm) (100, 200, 300, 400, 500) ▲ Full Gas Resp. △ Methane Elim.	Monitoring Well Construction
GROUND SURFACE						0	77.42		
Asphaltic concrete 0.05									
FILL: Grey crushed stone 0.20									
FILL: Brown, mixture of silt, sand, gravel and organics 1.00		SS	1	50	13			△	
Dark brown silty **TOPSOIL** 1.10		SS	2	100	12	1	76.42	△	
GLACIAL TILL: Compact, yellowish brown silty sand-gravel 1.25		AU	3					△	
End of Borehole Auger refusal on inferred bedrock at 1.25m depth. (BH dry upon completion)									

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I and II Environmental Investigation
1655 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant at southwest corner of property. Geodetic elevation = 78.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE September 21, 1994

FILE NO. E1116

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	77.40		
Asphaltic concrete — 0.10									
FILL: Brown sand-gravel, some silt — 0.25									
FILL: Brown, mixture of silt, sand and gravel — 1.10		AU	4					△	
						1	76.40		
End of Borehole Auger refusal on inferred bedrock at 1.10m depth. (BH dry upon completion)									

Upper scale: 20 40 60 80

Lower scale: 100 200 300 400 500

RKI Eagle Rdg. (ppm)

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

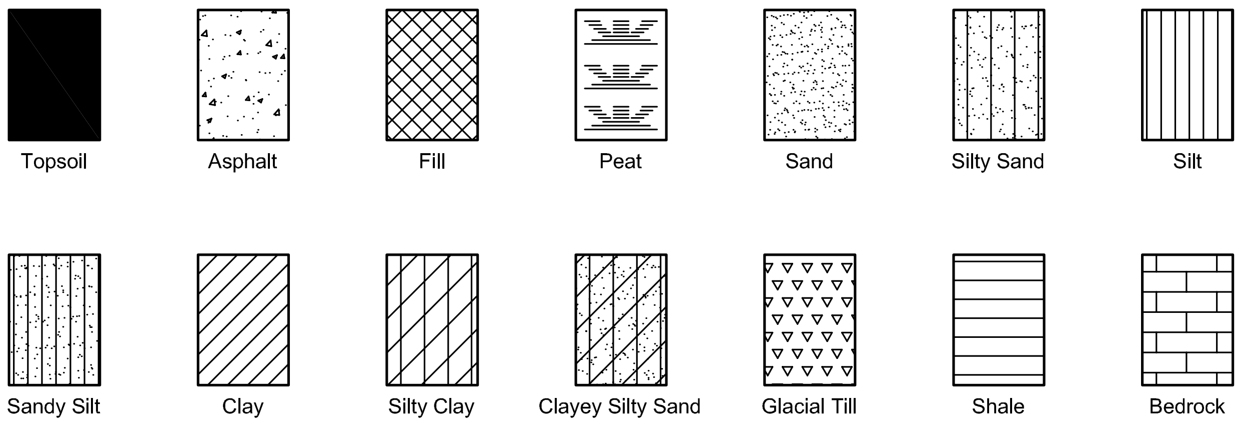
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

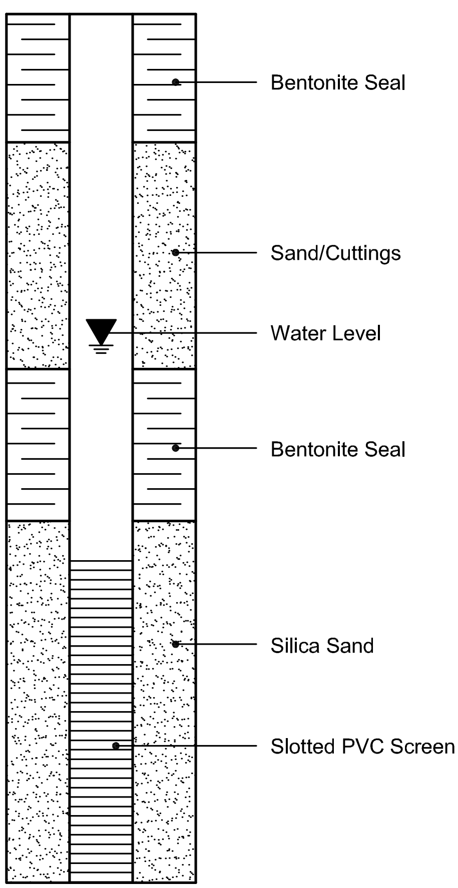
SYMBOLS AND TERMS (continued)

STRATA PLOT

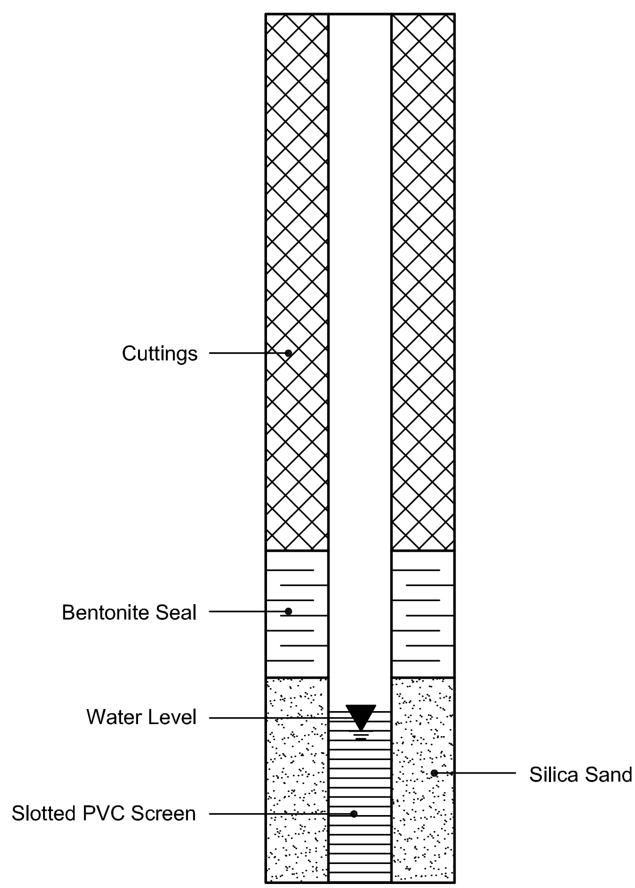


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 011-2812

LOCATION:

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 01-1

BORING DATE: 02 17, 2001

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m
		GROUND SURFACE	77.45			
0	Power Auger, 200mm Diam (Hollow Stem)	grey crushed stone (FILL)				
		ASPHALTIC CONCRETE	0.06			
		Grey crushed stone (FILL)	0.15			
		Frozen brown sandy silt, some gravel, trace clay (FILL)	0.30	1	AS	-
1		Dark brown TOPSOIL	76.38 / 1.07			
		Very dense brown sandy silt, some gravel and clay (GLACIAL TILL)	1.22	2	50 DO	>100
				3	50 DO	64
2	Rotary Drill, NQ Core	Slightly weathered grey LIMESTONE BEDROCK, some brown staining, occasional angular fracture and calcite seam	75.38 / 2.07	4	NQ RC	DD
3						
				5	NQ RC	DD
4						
		END OF BOREHOLE	72.97 / 4.48			
5						
6						
7						
8						
9						
10						

Photovac ppm ⊕ 100 200 300 400

ppm □ 100 200 300 400

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—⊖W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal

Granular Filter

32mm PVC #10 Slot Screen

W.L. in Screen at Elev. 75.68 m Feb. 24, 2001

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: D.J.S.

CHECKED: [signature]

PROJECT: 011-2812

RECORD OF BOREHOLE: 01-2

SHEET 1 OF 1

LOCATION:

BORING DATE: 02 17, 2001

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
		ASPHALT SURFACE	77.87				
0	Power Auger, 200mm Diam (Hollow Stem)	ASPHALTIC CONCRETE	0.00				
		Grey crushed stone (FILL)	0.09				
		Frozen brown sand silt, some gravel (FILL)	77.57 / 0.30				
1		Dark brown silty TOPSOIL	76.89 / 0.98	1	SS DO	21	
		Compact brown SANDY SILT, some gravel	76.71 / 1.16				Bentonite Seal
2				2	SS DO	27	
	Rotary Drill, NQ Core	Slightly weathered grey LIMESTONE BEDROCK with occasional brown staining and 25-50 mm seam @ 3.63-3.69 m depth	75.80 / 2.07	3	NQ RC	DD	
3							Granular Filter
4				4	NQ RC	DD	32mm PVC #10 Slot Screen
		END OF BOREHOLE	73.01 / 4.86				
5							W.L. in Screen at Elev. 75.40 m Feb. 24, 2001
6							
7							
8							
9							
10							

Photovac ppm ⊕ 100 200 300 400

ppm □ 100 200 300 400

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp —O W— Wl 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: D.J.S.

CHECKED: [illegible]

PROJECT: 011-2812

LOCATION:

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 01-3

BORING DATE: 02 24, 2001

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
		ASPHALT SURFACE	77.60			
0	Power Auger, 200mm Diam (Hollow Stem)	ASPHALTIC CONCRETE	0.00			
		Grey crushed stone (FILL)	0.08			
		Frozen brown sandy silt, some gravel (FILL)	77.30 / 0.30			
		TOPSOIL	76.84 / 0.76			
1		Compact brown SILTY SAND	0.81	1	50 DO	16
		Very dense brown sandy silt, some gravel and clay (GLACIAL TILL)	76.26 / 1.34	2	50 DO	>100
2	Rotary Drill, NQ Core	Fractured and weathered brown and grey LIMESTONE BEDROCK with mud seams	75.83 / 1.77	3	NQ RC	DD
3		Slightly weathered grey LIMESTONE BEDROCK with fracture @ 3.32 m depth and mud seam from 3.87-3.93 m depth	74.86 / 2.74			
4				4	NQ RC	DD
		END OF BOREHOLE	72.94 / 4.66			
5						
6						
7						
8						
9						
10						

Photovac ppm ⊕ 100 200 300 400

ppm □ 100 200 300 400

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal

Granular Filter

32mm PVC #10 Slot Screen

W.L. in Screen at Elev. 75.22 m Feb. 24, 2001

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: D.J.S.

CHECKED:

PROJECT: 011-2812

LOCATION:

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 01-4

BORING DATE: 02 24, 2001

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	Photovac ppm	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE		77.65							
	Power Auger, 200mm Diam (Hollow Stem)	Grey crushed stone (FILL)		0.00							
		Frozen to compact brown sandy silt, some gravel, trace concrete and ash (FILL)		77.44 / 0.21							Bentonite Seal
1					1	50 DO	18	⊕			
					2	50 DO	27	⊕			Native Backfill
2		Very dense brown sandy silt, some gravel, trace clay (GLACIAL TILL)		75.82 / 1.83							
					3	50 DO	>100	⊕			Bentonite Seal
3	Rotary Drill, NQ Core	Slightly weathered grey LIMESTONE BEDROCK with occasional black shale interbed		74.94 / 2.71							
4					4	NQ RC	DD				Granular Filter
5					5	NQ RC	DD				32mm PVC #10 Slot Screen
6		END OF BOREHOLE		71.71 / 5.94							
7											W.L. in Screen at Elev. 75.22 m Feb. 24, 2001
8											
9											
10											

Photovac ppm scale: 100, 200, 300, 400

Hydraulic conductivity scale: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20, 40, 60, 80

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.J.S.

CHECKED:

BOREHOLE 011-2812.GPJ GLDR_CAN.GDT 4 23 01

PROJECT: 021-2402

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 02-1

BORING DATE: June 17, 2002

SHEET 1 OF 2

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PID ppm (20, 40, 60, 80) / ppm (100, 200, 300, 400)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) / WATER CONTENT PERCENT Wp W Wl (10, 20, 30, 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE		77.46							
	Power Auger 200mm Diam (Hollow Stem)	Brown sandy silt, some gravel, trace concrete, brick and asphalt, occasional large lumps of concrete and asphalt (FILL)		0.00							Flush mount protective casing set in Bentonite Seal
					1	50 DO	WH				
1					2	50 DO	8				Native Backfill
					3	50 DO	WH				
		borehole continued next page		75.78 1.68							
2											
3											
4											
5											

BOREHOLE 021-2402.GPJ GLDR_CAN.GDT 4 7 02

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S

CHECKED: AH

PROJECT: 021-2402

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: 02-1

DRILLING DATE: June 17, 2002

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		borehole continued from previous page	75.78				
	R Drill / NW Casing	Weathered grey LIMESTONE BEDROCK	75.68				
2	Rotary Drill / NQ Core	Fresh to slightly weathered (at areas of fractures), medium grey to medium grained possibly slightly calcarenitic occasional argillaceous limestone and shale lamella. Upper 0.40 m of bedrock relatively fractured. - poorly developed near vertical fracture present at bedrock surface and at 3.35 m depth.	75.48 / 1.98	4	0	FR, PL, R FR, PL, R FR, U, VR	Granular Filter
3				5	10	FR, PL-U, VR	32mm PVC #10 Slot Screen
4		END OF BOREHOLE	73.10 / 4.36				W.L. in Screen at Elev. 75.72 m June 21, 2002
5							
6							

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S.

CHECKED: [illegible]

PROJECT: 021-2402

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 02-2

BORING DATE: June 17, 2002

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	77.71				
0	Power Auger, 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	0.00				
		Grey crushed stone (FILL)	77.62 / 0.09				
		Brown sandy silt, some gravel, trace brick (FILL)	77.50 / 0.21	1	50 DO	8	Bentonite Seal
				2	50 DO	7	Native Backfill
1		Brown SANDY SILT	76.67 / 1.04				
		Brown sandy silt, some gravel and clay, occasional boulder (GLACIAL TILL)	76.34 / 1.37	3	50 DO	18	Bentonite Seal
2				4	50 DO	>100	Granular Filter
	Rotary Drill, NW Casing			5	NQ RC	DD	
			74.84 / 2.87				
3		borehole continued next page					
4							
5							

PID ppm: 20 40 60 80 ⊕ / ppm: 100 200 300 400 □

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp — W — Wl: 10 20 30 40

ADDITIONAL LAB. TESTING

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S.

CHECKED: AH

PROJECT: 021-2402

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 02-2

DRILLING DATE: June 17, 2002

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		borehole continued from previous page	74.84												
3	Rotary Drill, NQ Core	Fresh to slightly weathered (at fractures), medium grey fine to medium grained DOLOSTONE with occasional undulatory very thin shale lamella between 2.87 m and 2.97 m. Dolostone contains carbonate infilled trace fossils (burroes).	74.84	6		100						F, U, VR			32mm PVC #10 Slot Screen
		- calcite / pyrite healed fault / joint approx 50 mm wide at 3.38 m depth - bedding at 90 degrees to core axis. - bedrock rubble between 2.44 m and 2.90 m depth. - fracture from 3.63 m and 4.11 m, approx. 22 mm wide and previously carbonate / quartz filled. Carbonates are now degraded predominantly to clays. Fracture is highly weathered		7		100						FR, U, R-VR FR, U-PL, R FR, PL-U, VR FR, PL, VR			
4												FR, PL, VR			
		END OF BOREHOLE	73.38 4.33												W.L. in Screen at Elev. 75.68 m June 21, 2002
5															
6															
7															

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S.

CHECKED:

RECORD OF BOREHOLE: 03-1

PROJECT: 021-2402

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

BORING DATE: 28 March 2003

SHEET 1 OF 1

DATUM: Local

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: POWER AUGER, 200 mm Diam. (Hollow Stem)

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	77.56
ASPHALT CONCRETE	0.05
Crushed stone (FILL)	77.26 / 0.30
Sand and gravel (FILL)	76.65 / 0.91
Brown SANDY SILT, traces of gravel (GLACIAL TILL)	76.39 / 1.17
Slightly weathered to sound grey LIMESTONE BEDROCK, with occasional black shale interbed	72.61 / 4.95
End of Borehole	

SAMPLES NUMBER	TYPE	BLOWS/0.3m	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)
1	50 DO	14			
2	NQ RC	-	96.3	76.7	43.3
3	NQ RC	-	94.9	90.8	70.8
4	NQ RC	-	93.8	93.8	93.8

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q- ● ; rem V. ⊕ U- O ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl ; 10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:

- Bentonite Seal
- Silica Sand
- 32 mm Diam. PVC #10 Slot Screen
- W.L. in Screen at Elev. 75.87 m April 13, 203

DEPTH SCALE METRES: 0 1 2 3 4 5 6

DEPTH SCALE

1 : 30

Golder Associates

LOGGED:

CHECKED:

BOREHOLE 021-2402-8000.GPJ HYDROGEO.GDT 1/5/03

RECORD OF BOREHOLE: 03-2

PROJECT: 021-2402
LOCATION: See Site Plan
SAMPLER HAMMER, 64kg; DROP, 760mm

BORING DATE: 28 March 2003

SHEET 1 OF 1
DATUM: Local
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - O: 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp —O W— Wl: 10 20 30 40

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		77.51								
0	POWER AUGER 200 mm Diam. (Hollow Stem)	(From TP 03-2) Light brown sand and gravel with building debris (FILL)		0.00								Bentonite Seal
3		Slightly weathered grey LIMESTONE BEDROCK, with occasional black shale interbed		74.61 2.90	1	50 DO		98.2	80.7	71.9		Silica Sand 32 mm Diam. PVC #10 Slot Screen
					2	NQ RC		100	100	95.8		
5		End of Borehole		72.56 4.95								W.L. in Screen at Elev. 75.98 m April 13, 2003

DEPTH SCALE
1 : 30

Golder Associates

LOGGED:
CHECKED:

BOREHOLE 021-2402-6000.GPJ HYDROGEO.GDT 1/5/03

RECORD OF TEST PITS

Test Pit Number	Depth (metres)	Description
TP03-1 (trench)	0.00 – 0.10	ASPHALT
	0.10 – 0.25	Crushed stone gravel (FILL)
	0.25 – 2.60	Light brown sand (FILL)
	2.60	End of Test Pit on LIMESTONE bedrock
		Note: Black staining and hydrocarbon odour at bedrock adjacent to basement foundation.
TP03-2 (trench)	0.00 – 1.00	Light brown Sand and gravel (FILL)
	1.00-2.60	Building debris and sand (FILL)
	2.60	End of Test Pit on LIMESTONE bedrock
		Note: base of rubble heavily oil stained (2.20-2.50 m).
TP03-3	0.00 – 2.50	Light brown sand (FILL)
	2.50	End of Test Pit on LIMESTONE bedrock
		Note: no evidence of staining or hydrocarbon odours.
TP03-4	0.00 – 2.00	Light brown sand, trace silt (FILL)
	2.00 – 2.20	Grey brown sandy silt and gravel (GLACIAL TILL)
	2.20	End of Test Pit on LIMESTONE bedrock
		Note: no evidence of staining or hydrocarbon odours
TP03-5	0.00 – 0.10	ASHPHALT
	0.10 – 0.25	Crushed stone gravel (FILL)
	0.25 – 2.70	Grey brown silty sand and gravel with building debris (FILL)
	2.70	End of Test Pit on LIMESTONE bedrock
		Note: traces of hydrocarbon product at 2.50 m near the base of basement foundation
TP03-6	0.00 – 0.15	Crushed stone gravel (FILL)
	0.15 – 1.50	Grey brown silty sand and gravel with building debris (FILL)
	1.50 – 2.00	Building rubble (FILL)
	2.00	End of Test Pit on LIMESTONE bedrock
		Note: no evidence of staining or hydrocarbon odours

Logged by: PAH
Checked by: KPH

RECORD OF TEST PITS - continued

Test Pit Number	Depth (metres)	Description
TP03-7	0.00 – 0.10	Crushed stone gravel (FILL)
	0.10 – 1.70	Brown silty sand, some gravel and building debris (FILL)
	1.70	End of Test Pit on LIMESTONE bedrock
		Note: no evidence of staining or hydrocarbon odours

Logged by: PAH
Checked by: KPH

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - WATER SUPPRESSION SYSTEM

DRAWING PG5118-1 - TEST HOLE LOCATION PLAN

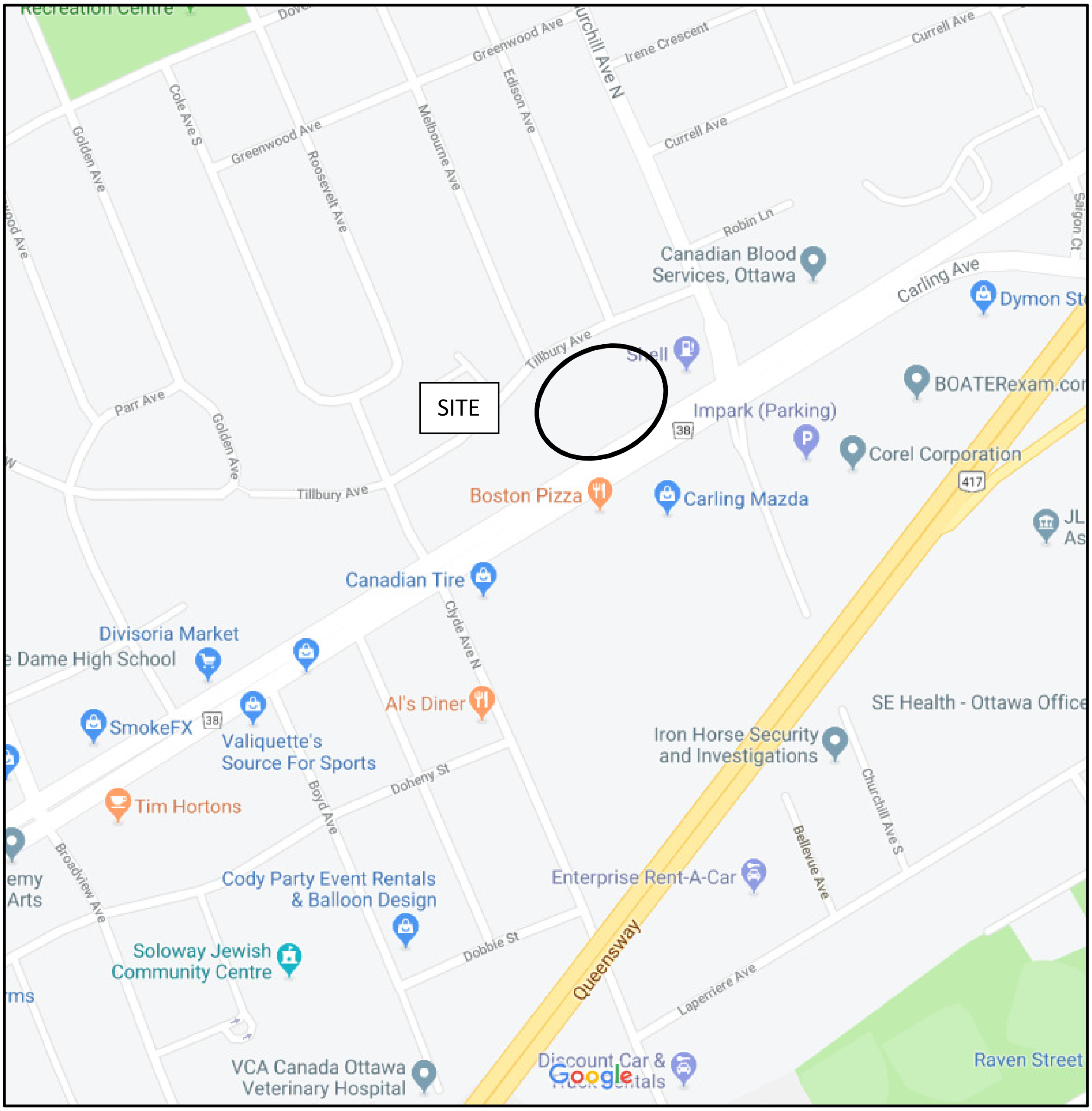
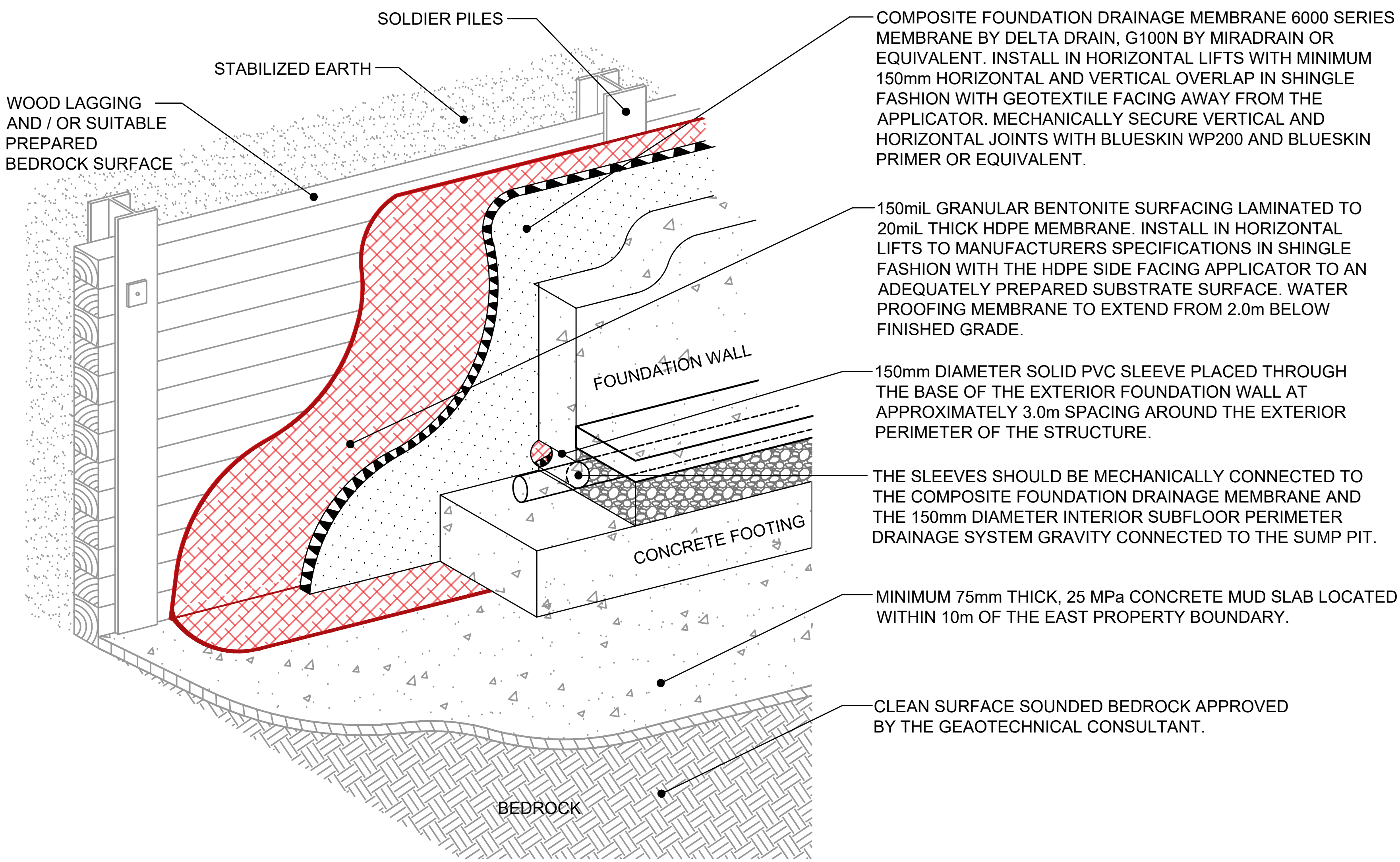


FIGURE 1

KEY PLAN

patersongroup



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SURFACE DEVELOPMENTS

PROPOSED MULTI-STOREY BUILDING
1619 AND 1655 CARLING AVENUE

OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM

Scale: NTS	Date: 11/2020
Drawn by: RCG	Report No.: PG5118-1
Checked by: RG	Drawing No.: FIG.2
Approved by: DG	Revision No.:

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LEGEND:

- BOREHOLE WITH MONITOING WELL LOCATION, CURRENT INVESTIGATION (PE4987, 2020)
- BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PE1472, 2008)
- BOREHOLE WITH MONITOING WELL LOCATION, PREVIOUS INVESTIGATION (PE1472, 2008)
- TEST PIT LOCATION, PREVIOUS INVESTIGATION (PE1472, 2008)
- BOREHOLE LOCATION, PREVIOUS INVESTIGATION (E1116, 1994)
- BOREHOLE WITH MONITORING WELL LOCATION BY OTHERS (GOLDER ASSC. 2001-2003)
- TEST PIT LOCATION BY OTHERS (GOLDER ASSC. 2001-2003)
- TEST PIT LOCATION BY OTHERS (TEST PIT LOG NOT AVAILABLE FOR CURRENT INVESTIGATION)

77.71 GROUND SURFACE ELEVATION (m)

[74.84] BEDROCK SURFACE ELEVATION (m)

TBM #1 (PE4987, 2020) - TOP OF GRATE OF CATCH BASIN MANHOLE LOCATED NEAR THE NORTHEAST CORNER OF SUBJECT SITE. GEODETIC ELEVATION = 77.35m

TBM #2 (PE1472, 2008) - TOP OF MANHOLE COVER LOCATION ALONG CARLING AVENUE. GEODETIC ELEVATION = 77.51m

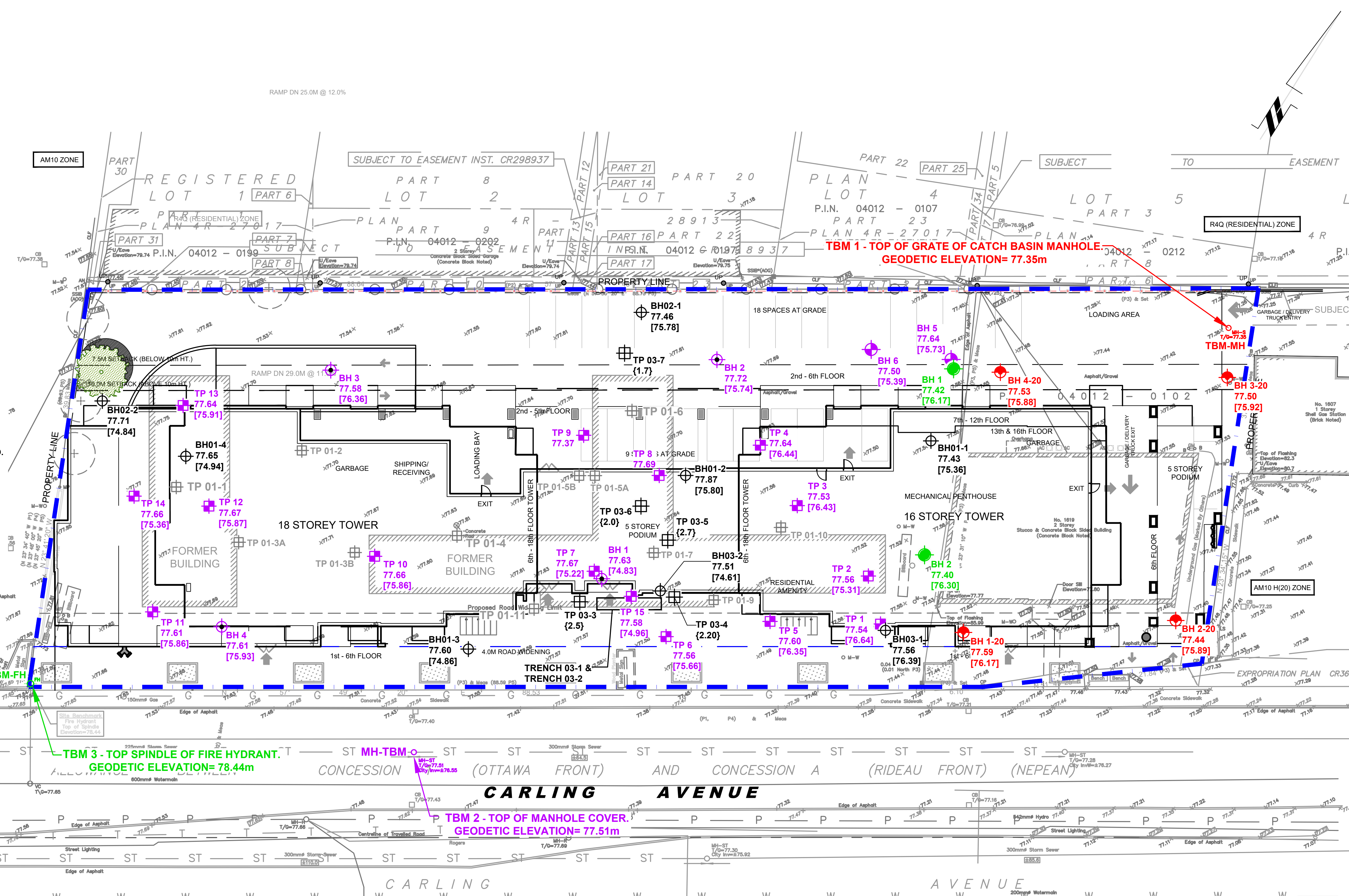
TBM #3 (E1116, 1994) - TOP OF SPINDLE OF FIRE HYDRANT AT SOUTHWEST CORNER OF PROPERTY. GEODETIC ELEVATION = 78.44m

BASEPLAN PROVIDED BY RODERICK LAHEY ARCHITECT INC.

SURVEY BASEPLAN PREPARED BY FARLEY, SMITH & DENNIS LTD.

SCALE: 1:400

0 5 10 15 20 25m



1	UPDATED SITE BOUNDARIES NEW 2020 BOREHOLES ADDED	30/11/2020	MS
NO.	REVISIONS	DATE	INITIAL

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SURFACE DEVELPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
1619 AND 1655 CARLING AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 10/2019
Drawn by: RCG	Report No.: PG5118-1
Checked by: RG	Dwg. No.: PG5118-1
Approved by: DJG	Revision No.: 1

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