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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Archaeological Studies

Geotechnical Investigation
Proposed Multi-Storey Building
2070 Scott Street
Ottawa, Ontario

Prepared For

Westboro Point Developments Ltd.

July 26, 2019

Report PG4935-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Westboro Point Developments Ltd. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 2070 Scott Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

☐ determine the subsurface soil and groundwater conditions at this site based on available subsoil information from current and previous investigations.

☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

Based on the preliminary concept drawings, it is our understanding that the proposed project is to consist of a multi-storey building with 2 to 3 levels of underground parking. Associated access lanes, parking areas and landscaped margins are also anticipated as part of the proposed development.

It is further understood that the proposed building will be serviced with municipal water and sewer.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on May 15, 2019. At that time, a total of three (3) boreholes (BH 4-19 through BH 6-19) were advanced to a maximum depth of 8.3 m below the existing ground surface. The boreholes were distributed in a manner to provide general coverage of the proposed development taking into consideration existing site features and underground utilities.

Previous geotechnical investigations conducted at the subject site by Paterson included three (3) boreholes (BH 1 through BH 3) completed on April 2, 2013, one (1) test pit (TP 1) completed on April 3, 2013, five (5) test pits (TP 1 through TP 5) completed on October 21, 2002, five (5) boreholes (BH 1 through BH 5) completed on October 15, 2001, and four (4) boreholes (BH 1-1 through BH 4-1) completed on November 18, 1996. The locations of the test holes are shown on Drawing PG4935-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted auger drill rig operated by a two person crew. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden. The test pit procedure consisted of excavating to the required depths at the selected locations and sampling the overburden. The test pits were backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

A field investigation program was also completed at the subject site by others during the period of April 2 through 5, 2013, consisting of a total of 12 boreholes (BH/MW 1 through BH 12) advanced to a maximum depth of 13.5 m below the existing ground surface. The borehole logs prepared by others are provided in Appendix 1.

Sampling and In Situ Testing

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

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

Diamond drilling was completed at select locations to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in boreholes BH 1, BH 3, BH 4-19, BH 5-19 and BH 6-19 to permit the monitoring of water levels subsequent to the completion of the sampling program.

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 we are otherwise directed.

3.2 Field Survey

The test holes completed during the most recent geotechnical investigations on May 15, 2019 and April 2 and 3, 2013 were selected and determined in the field by Paterson personnel to provide general coverage of the subject site. The location and ground surface elevation at these borehole locations were surveyed by Paterson personnel. The test holes were surveyed with respect to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located along the west property boundary near Churchill Avenue. A geodetic elevation of 66.18 m was provided for the TBM.

The location and ground surface elevation at each test hole location is presented on Drawing PG4935-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples recovered from the subject site were visually examined in our laboratory to review the field logs. Two bedrock samples were submitted for uniaxial compressive strength, the results of which are provided in Appendix 1.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the sulphate potential against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The results are provided in Appendix 1, and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant, with a mixture of asphaltic pavement structure, granular crushed stone and some concrete and construction debris located at the existing ground surface. The site is bordered by Scott Street to the north, Winona Avenue to the east, residential properties to the south, and Churchill Avenue to the west. The existing ground surface across the site slopes downward gradually from west to east, from approximate geodetic elevation 65.5 m at the west property line to approximate geodetic elevation 63 m at the east property line.

It is understood that the site was formerly occupied by two commercial buildings, a 3 storey building with a basement on the west portion of the site and a single storey, slab on grade building on the east portion of the site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of fill underlying the existing crushed stone surface or 80 to 150 mm thick asphalt surface. The fill material extended to approximate depths of 1.4 to 3.8 m below the existing ground surface and generally consisted of loose to dense, brown silty sand to silty clay with trace to some gravel, cobbles, boulders, and construction debris such as glass, wood chips, brick, and concrete.

Bedrock

Practical refusal to augering or excavation was encountered at the test holes at depths of 1.4 to 3.8 m below the existing ground surface. Bedrock was cored at boreholes BH 1, BH 3, BH 4-19, BH 5-19, and BH 6-19 to depths of 7.7 to 13.5 m, and consisted of a poor to excellent quality limestone to limestone with interbedded dolostone and shale.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and dolostone of the Gull River Formation with drift thicknesses of 1 to 2 m.

4.3 Groundwater

Groundwater levels were measured in the groundwater monitoring wells BH 1 and BH 3 on April 4, 2013, and in the groundwater monitoring wells BH 4-19 through BH 6-19 on May 15, 2019. The measured groundwater level (GWL) readings are presented in Table 1 below and on the Soil Profile and Test Data sheets in Appendix 1.

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

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface Elevation (m)	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 1	65.09	6.93	58.16	April 4, 2013
BH 3	63.07	5.27	57.80	April 4, 2013
BH 4-19	63.71	7.10	56.61	May 22, 2019
BH 5-19	63.34	6.14	57.20	May 22, 2019
BH 6-19	62.99	5.82	57.17	May 22, 2019

Note: - The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located along the west property boundary near Churchill Avenue with a geodetic elevation of 66.18 m.

5.0 Discussion

5.1 Geotechnical Assessment

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

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

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow depth of the bedrock 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 underground levels.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

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 to the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

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

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

Excavation side slopes in sound bedrock 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 for potential sloughing or to provide a stable base for the overburden shoring system.

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 equipment 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 would utilize such equipment. 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. Therefore, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

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.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum of 300 mm thick loose lifts and compacted using suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Delta Drain 6000.

5.3 Foundation Design

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

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

A factored bearing resistance value at ULS of **4,500 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 within 1.5 m below the founding level. This should be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the all the footing footprints. A minimum of one probe hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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

A site specific shear wave velocity test was completed to accurately determine the applicable seismic site classification for foundation design of the proposed building as presented in Table 4.1.8.4.A of the Ontario Building Code 2012. A seismic shear wave velocity test was completed by Paterson at the subject site. Two shear wave velocity profiles are presented in Appendix 2.

Field Program

The shear wave test location is presented in Drawing PG4935-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel installed 24 horizontal geophones in a straight line oriented roughly in a north-south direction along the eastern site boundary. The 4.5 Hz horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was connected to a computer and a trigger switch attached to a 12 pound dead blow hammer. The hammer trigger sends a signal to the seismograph to commence recording. The hammer strikes an I-Beam seated into the ground surface, which produces a polarized shear wave. The shots are repeated between four to eight times at each shot location to provide an accurate signal and reduce noise. The shot locations are completed in forward and reverse directions (i.e. striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were distributed at the centre of the geophone array and 1, 2 and 5 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson. The shear wave velocity measurement was calculated by the reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, immediately below the proposed building foundation of the upper 30 m profile. To compute the bedrock depth at each location, the layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave graphs. The bedrock velocity was interpreted by the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. As bedrock quality increases, the bedrock shear wave velocity increases.

The V_{s30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012, as presented below;

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

$$V_{s30} = \frac{30m}{\left(\frac{0m}{233m/s} + \frac{30m}{1{,}805m/s}\right)}$$

$$V_{s30} = 1{,}805m/s$$

Based on the seismic results, the average shear wave velocity, V_{s30}, for shallow foundations located at the subject site is 1,805 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building at the subject site, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

For the subject site development, all overburden soil should be removed from the subject site and the basement floor slab will be founded on a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 150 to 200 mm of sub-slab fill consists of a 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered at the time of the field investigation, an underfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

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

It is expected that a portion of the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 23.5 kN/m^3 (effective 15.5 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for the foundation wall which is poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Static Conditions

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

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

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

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

Seismic Conditions

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to the 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 the 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 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.

The centre to centre spacing between bond lengths should be a minimum of 1.2 m or four times the anchor hole diameter to ensure the group influence effects are minimized. 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.

Regardless of whether an anchor is a passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the base, which will provide the 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

The unconfined compressive strength of limestone at the subject site ranges between 65 and 125 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 1.

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	65 MPa
Effective unit weight - Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
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 3. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1	3.6	600
	3.2	1.2	4.4	750
	4.5	2	6.5	1500
125	1.6	0.6	2.2	600
	2	1	3	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other Considerations

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom to top of the anchor holes.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on test procedures 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

Where paved areas are considered for the project, the recommended pavement structures shown in Tables 4 through 6 would be applicable.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

Table 6 - Recommended Rigid Pavement Structure - Lowest Parking Level

Thickness (mm)	Material Description
150	**32 MPa - C4 - Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD with suitable vibratory equipment, noting that excessive vibration could lead to subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System

Where the proposed building is to have more than 2 underground parking levels, the following water suppression system is recommended to manage and control groundwater water infiltration over the long term. The water suppression system would be installed for the exterior foundation walls and underfloor drainage and would consist of the following (refer to Figure 4 - Water Suppression System in Appendix 2 for an illustration of this system cross-section):

❏ A concrete mud slab creating a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation. The thickness of the concrete mud slab will be determined during the excavation program when realistic groundwater infiltration can be properly assessed. However for preliminary design purposes, it is recommended that the concrete mud slab be designed at a minimum thickness of 150 mm.

❏ A waterproofing membrane to lessen the effect of water infiltration for the lower underground parking level(s) starting at 6 m below finished grade. The waterproofing membrane will consist of a bentonite waterproofing such as Tremco Paraseal or equivalent securely fastened to the temporary shoring system or the vertical bedrock surface. The membrane should extend to the bottom of the excavation at the founding level and extend horizontally over the concrete mud slab a minimum of 300 mm prior to the placement of the footings. Consideration can be given to doubling the bentonite waterproofing panels within the lower portion of the underground parking levels where hydrostatic pressure will be greater.

Water infiltration will result from two sources. The first will be water infiltration from the upper 6 m which is above the vertical waterproofed area. The second source will be water breaching the waterproofing membrane.

Foundation Drainage

A composite drainage layer should be placed from finished grade to the bottom of the foundation wall. Where the proposed building is to have more than 2 underground parking levels and the water suppression system is employed, the composite drainage layer should be placed between the waterproofing membrane and the foundation wall.

It is recommended that the composite drainage system consist of DeltaDrain 6000, MiraDrain G100N or an approved equivalent. 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 will be required to control water infiltration below the lowest underground parking level slab. For design purposes, it’s recommended that 150 mm diameter perforated pipes be placed at approximate 6 m spacing underlying the lowest level floor slab. 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.

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, as recommended above, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

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 and Temporary Shoring

Side Slopes

The excavation side slopes in the overburden, above the groundwater level, extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

As noted above, excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

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.

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

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements 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 shoring system could consist of soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure the 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 the preferred method.

The earth pressures acting on the shoring system may be calculated with 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 calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be 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 test pit completed at one of the adjacent building foundations and the relatively shallow depth of the bedrock at the subject site, it is expected that the buildings along the southern boundary of the site are most likely founded on the bedrock surface. Therefore, underpinning is not expected to be required for this project.

However, Paterson should review the condition of the bedrock underlying the adjacent building foundations at the time of construction to evaluate if bedrock stabilization is required.

6.4 Pipe Bedding and Backfill

A minimum of 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 SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Section 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Due to the limited capacity of the existing sewers, it is anticipated that pumped groundwater can be temporarily contained within a cistern/holding tank to permit reduced discharge volumes, if required. Provided the proposed groundwater infiltration control system is properly implemented where more than 3 underground parking levels are built, it is expected that groundwater flow will be low (i.e.- less than 3,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that where the underground levels extend below the groundwater level (3 or more underground levels), the lower portion of the foundation will have a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place against the bedrock face in this scenario, long-term groundwater lowering is anticipated to be negligible for the area.

Further, based on our observations, the groundwater level is anticipated at a 5 to 7 m depth and located within the bedrock. Therefore, local groundwater lowering is not anticipated under short-term conditions due to construction of the proposed building.

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

6.6 Winter Construction

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

Where excavations are completed in proximity to existing structures, they may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

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

❑ Review the bedrock stabilization and excavation 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.

❑ 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. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Westboro Point Developments Ltd. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Richard Groniger, C. Tech.

Scott Dennis

Scott S. Dennis, P. Eng.

LICENSED PROFESSIONAL ENGINEER
July 26, 2019
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Report Distribution:

❑ Westboro Point Developments Ltd. (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

SYMBOLS AND TERMS

UNIAXIAL COMPRESSIVE STRENGTH TESTING RESULTS

ANALYTICAL RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 2070 and 2090 Scott St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

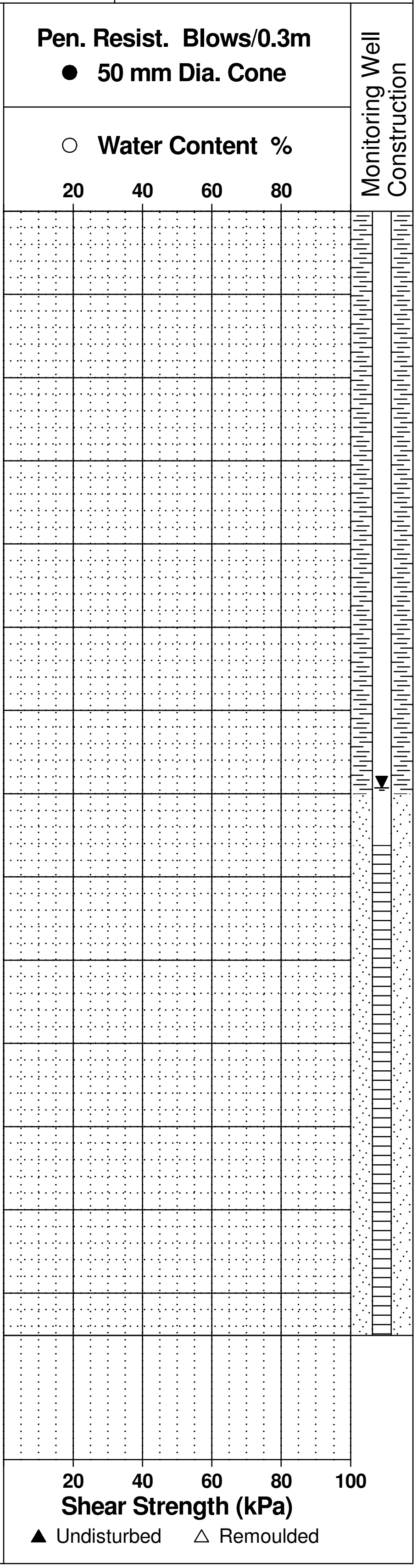
BORINGS BY CME 55 Power Auger

DATE April 2, 2013

FILE NO. PG2936

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.09
100mm Asphaltic concrete over crushed stone 0.25		AU	1				
FILL: Brown silty sand with gravel 1.37		SS	2	8	4	1	64.09
Fractured **BEDROCK:** 1.65							
		RC	1	100	75	2	63.09
BEDROCK: Grey limestone						3	62.09
		RC	2	100	72		
4.40						4	61.09
BEDROCK: Grey limestone interbedded with dolostone		RC	3	100	100	5	60.09
6.02						6	59.09
		RC	4	100	96	7	58.09
		RC	5	100	84	8	57.09
						9	56.09
BEDROCK: Grey limetone with intermittent dolostone and shale		RC	6	100	71	10	55.09
						11	54.09
		RC	7	100	85	12	53.09
		RC	8	100	100	13	52.09
13.51							
End of Borehole							
(GWL @ 6.93m-April 4, 2013)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 2070 and 2090 Scott St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 2, 2013

FILE NO. PG2936

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE						0	63.77		
150mm Asphaltic concrete over crushed stone 0.46		AU	1						
FILL: Brown silty clay with sand, gravel, trace glass 1.68		SS	2	21	7	1	62.77		
FILL: Brown silty sand iwth gravel, cobbles, trace boulders 2.29		SS	3	4	48	2	61.77		
FILL: Topsoil, trace wood chips 2.59		SS	4	42	12				
GLACIAL TILL: Brown silty sand with gravel, trace cobbles and boulders 3.18		SS	5	33	50+	3	60.77		

End of Borehole

Practical refusal to augering at 3.18m depth

(BH dry upon completion)

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 2070 and 2090 Scott St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY CME 55 Power Auger

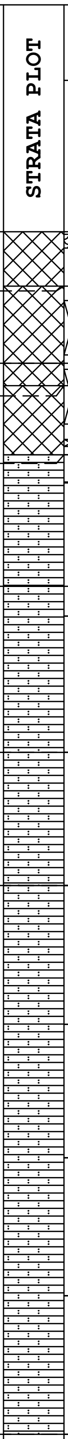
DATE April 2, 2013

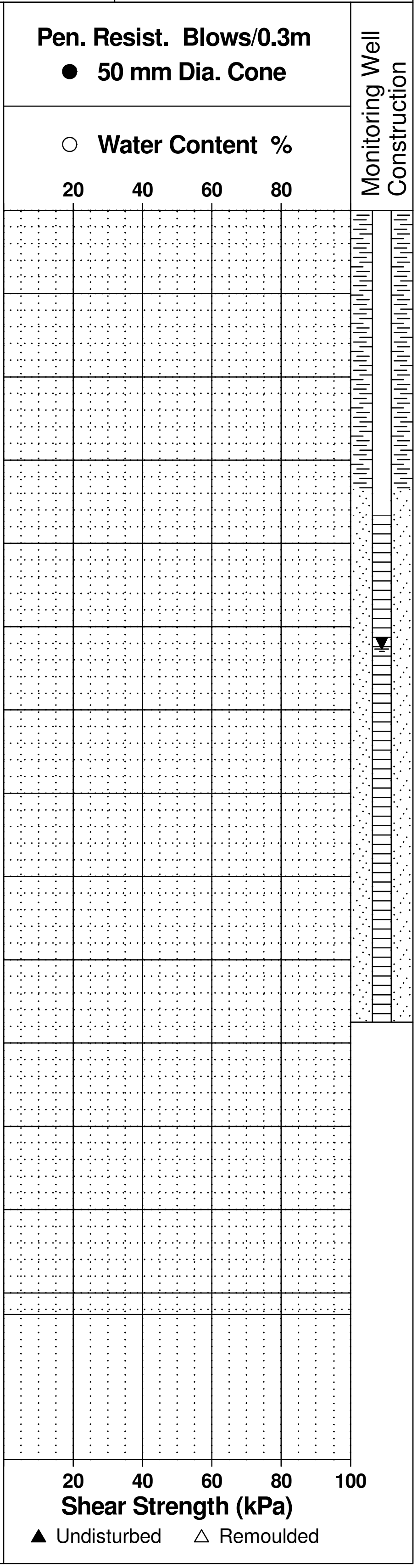
FILE NO. PG2936

HOLE NO. BH 3

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
25mm Asphaltic concrete over crushed stone	0.60	AU	1		
FILL: Red-brown silty sand	1.45	SS	2	46	11
FILL: Topsoil	1.70	SS	3	71	6
FILL: Brown silty sand with gravel, cobbles, trace boulders	2.46	SS	4	100	50+
BEDROCK: Grye limestone	3.91	RC RC	1 2	100 100	75 67
BEDROCK: Grey to black dolostone interbedded with limestone	5.74	RC	3	71	29
BEDROCK: Grey limestone interbedded with dolostone	7.21	RC	4	100	83
BEDROCK: Black dolostone interbedded with limestone	13.26	RC RC RC RC	5 6 7 8	90 86 100 100	60 57 83 80
End of Borehole					
(GWL @ 5.27m-April 4, 2013)					

DEPTH (m)	ELEV. (m)
0	63.07
1	62.07
2	61.07
3	60.07
4	59.07
5	58.07
6	57.07
7	56.07
8	55.07
9	54.07
10	53.07
11	52.07
12	51.07
13	50.07





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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 May 15

FILE NO. PG4935

HOLE NO. BH 4-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.71
FILL: Brown silty sand and gravel	AU	1				
	SS	2	21	6	1	62.71
	SS	3	54	53		
2.13					2	61.71
FILL: Brown silty sand with crushed stone	SS	4	56	50+		
2.84					3	60.71
BEDROCK: Grey limestone	RC	1	88	45	4	59.71
	RC	2	100	75	5	58.71
	RC	3	100	26	6	57.71
					7	56.71
	RC	4	100	58	8	55.71
8.31						
End of Borehole						
(GWL @ 7.10m - May 22, 2019)						

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY CME 55 Power Auger

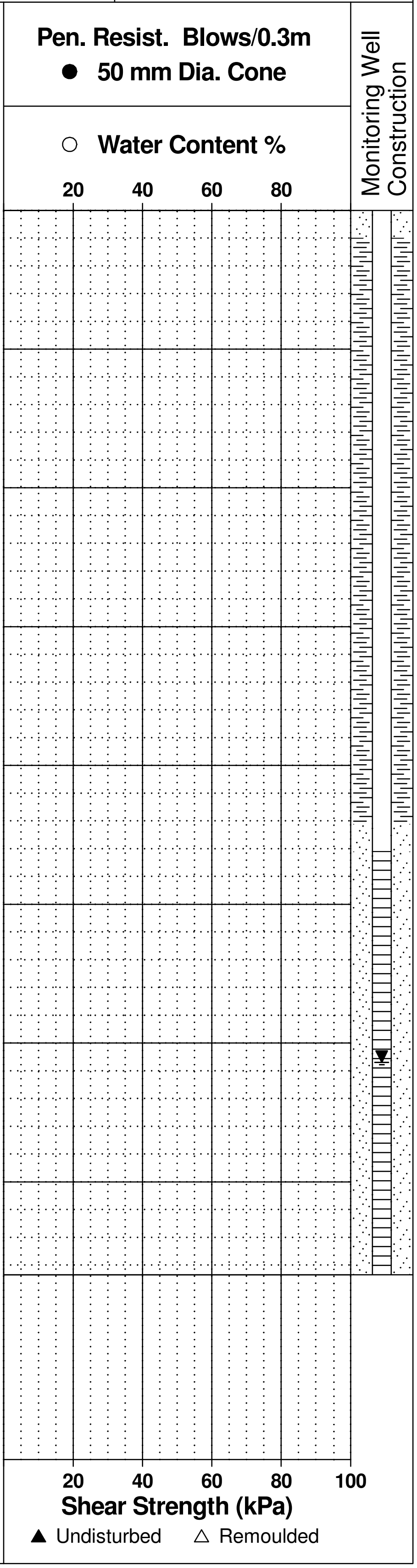
DATE 2019 May 15

FILE NO. PG4935

HOLE NO. BH 5-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.13				
FILL: Brown silty sand	AU	1		
	SS	2	46	11
1.37				
FILL: Brown silty sand, some clay, trace brick	SS	3	24	3
2.21				
BEDROCK: Grey limestone	RC	1	100	39
	RC	2	100	52
	RC	3	100	56
	RC	4	100	73
	RC	5	100	38
7.67				
End of Borehole				
(GWL @ 6.14m - May 22, 2019)				

DEPTH (m)	ELEV. (m)
0	63.34
1	62.34
2	61.34
3	60.34
4	59.34
5	58.34
6	57.34
7	56.34



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY CME 55 Power Auger

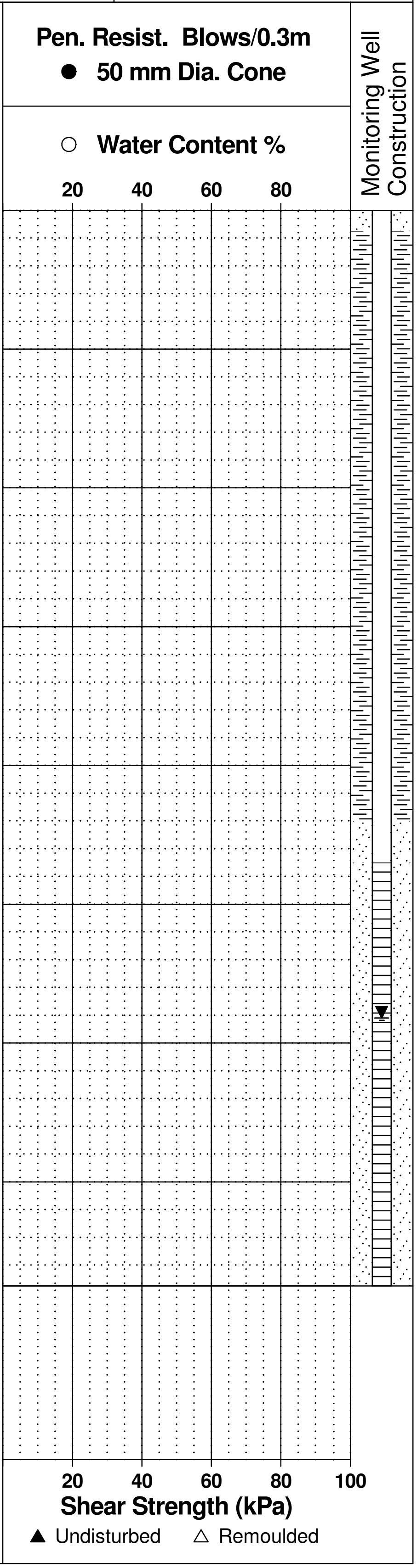
DATE 2019 May 15

FILE NO. PG4935

HOLE NO. BH 6-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand and crushed rock, some concrete	AU	1		
	SS	2	25	13
	SS	3	64	50+
2.13				
BEDROCK: Grey limestone	RC	1	100	20
	RC	2	100	64
	RC	3	98	60
	RC	4	100	85
	RC	5	100	74
7.75				
End of Borehole				
(GWL @ 5.82m - May 22, 2019)				

DEPTH (m)	ELEV. (m)
0	62.99
1	61.99
2	60.99
3	59.99
4	58.99
5	57.99
6	56.99
7	55.99



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 2070 and 2090 Scott St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the west side of Churchill Avenue, along the west property line. Geodetic elevation = 66.18m.

REMARKS

BORINGS BY Hydraulic Shovel

DATE April 3, 2013

FILE NO. PG2936

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	65.19		
Asphaltic concrete 0.08									
FILL: Crushed stone 0.76									
FILL: Brown silty sand with gravel, cobbles, trace boulders and topsoil 1.62		G	1			1	64.19		
End of Test Pit Practical refusal to excavation at 1.62m depth Concrete foundation wall founded directly over bedrock surface at 1.62m depth. No footing encountered. (TP dry upon completion)									

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

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Engineers
28 Concourse Gate, Unit 1, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2074 Scott Street
Ottawa, Ontario

DATUM

REMARKS

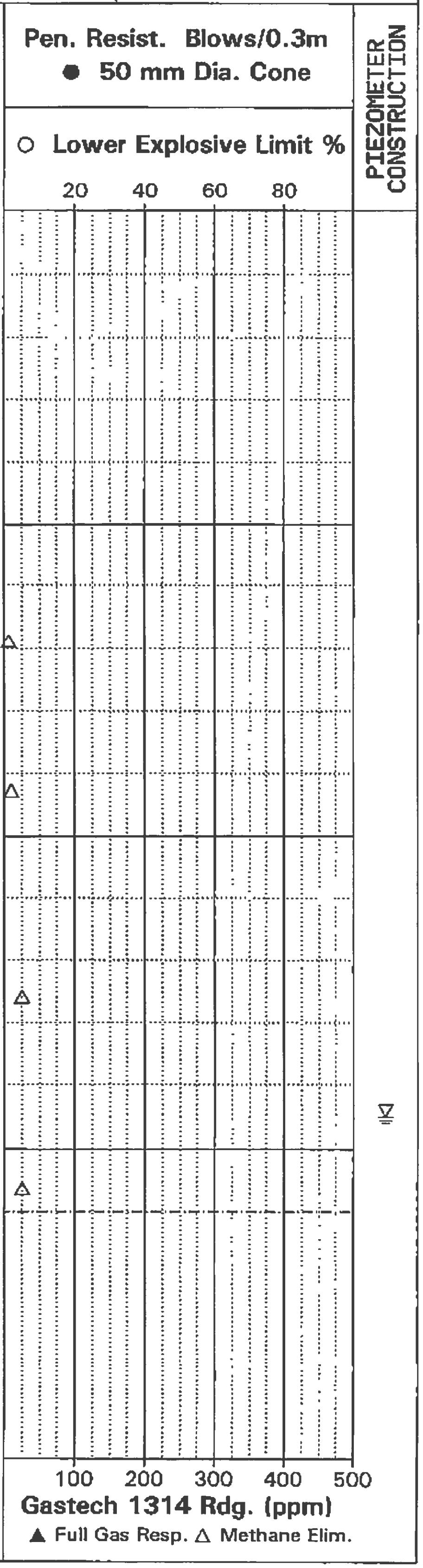
BORINGS BY Backhoe

DATE 21 OCT 02

FILE NO. E2283

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
						1	
FILL: Brown sand	▨	G	1				
	▨	G	2			2	
2.44							
Brown SANDY SILT		G	3				
3.05						3	
Grey SAND		G	4				
3.20							
End of Test Pit (Soil saturated below 2.9m depth)							



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2074 Scott Street
Ottawa, Ontario

FILE NO. **E2283**

HOLE NO. **TP 2**

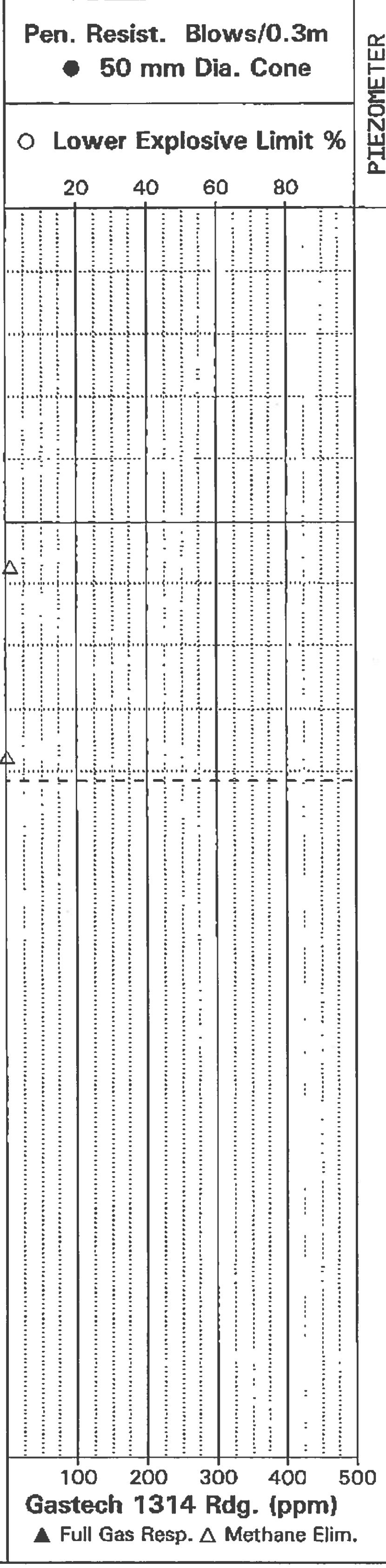
DATUM

REMARKS

BORINGS BY Backhoe

DATE 21 OCT 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Brown sand and gravel, some cobbles and plastic pieces	▨					1	
	▨	G	1				
1.83	▨	G	2				
End of Test Pit TP terminated on bedrock surface @ 1.83m depth (TP dry upon completion)							



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2074 Scott Street
Ottawa, Ontario

DATUM

REMARKS

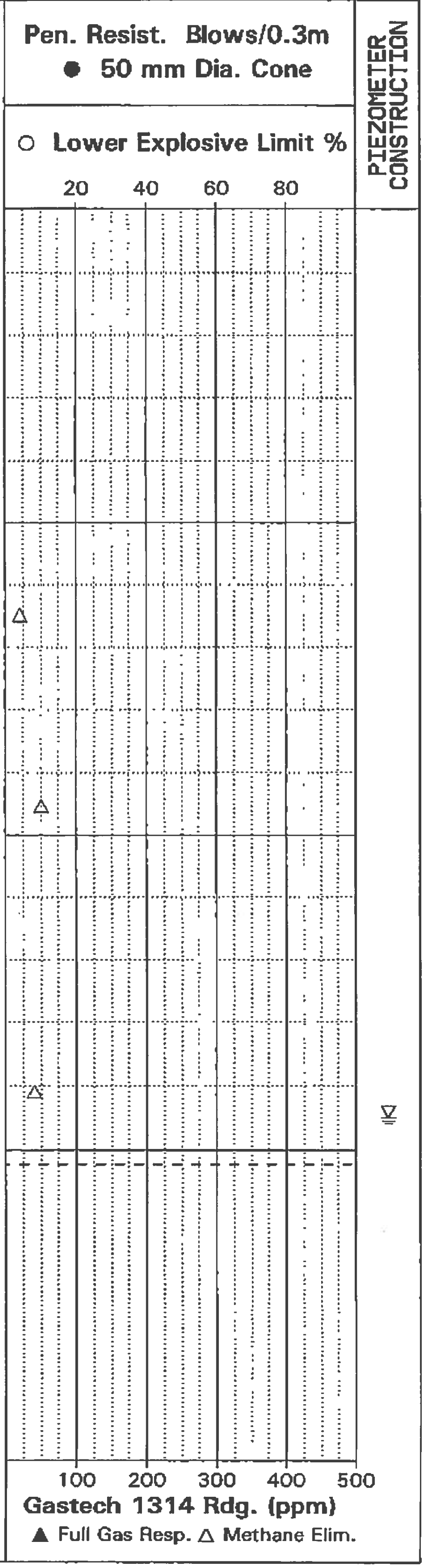
BORINGS BY Backhoe

DATE 21 OCT 02

FILE NO. E2283

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	
	▨					1	
FILL: Dark brown sand and gravel with plastic, asphalt and steel pieces	▨	G	1				
	▨	G	2				
	▨					2	
	▨	G	3				
3.05	▨					3	
End of Test Pit (Soil saturated below 2.9m depth)							



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2074 Scott Street
Ottawa, Ontario

DATUM

REMARKS

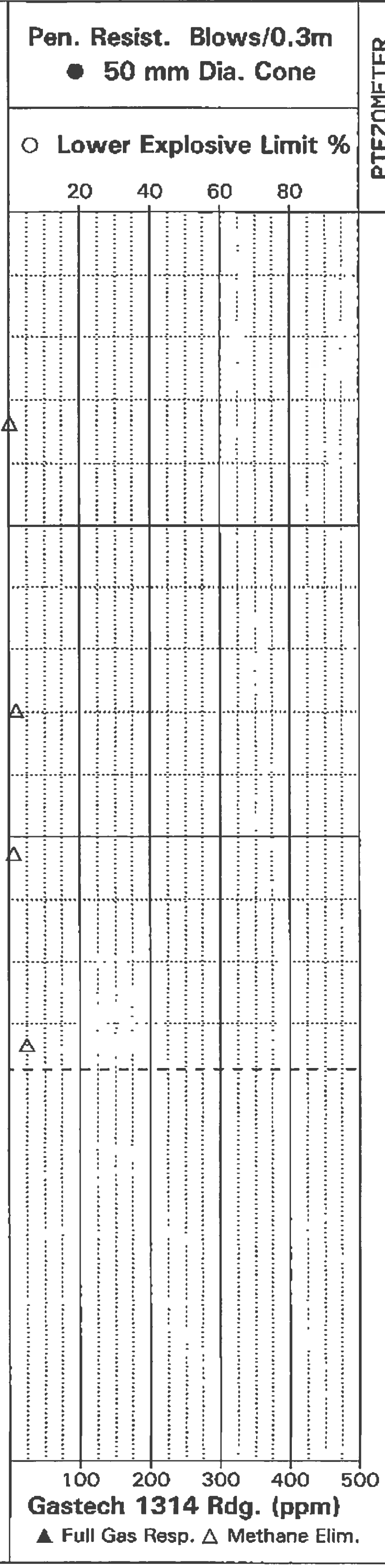
BORINGS BY Backhoe

DATE 21 OCT 02

FILE NO. E2283

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
GROUND SURFACE						0		
		G	1					
FILL: Gravel with miscellaneous debris						1		
		G	2					
						2		
2.13		G	3					
GLACIAL TILL: Dense, brown silty sand and gravel								
2.74		G	4					
End of Test Pit (TP dry upon completion)								



JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Engineers
28 Concourse Gate, Unit 1, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2074 Scott Street
Ottawa, Ontario

DATUM

REMARKS

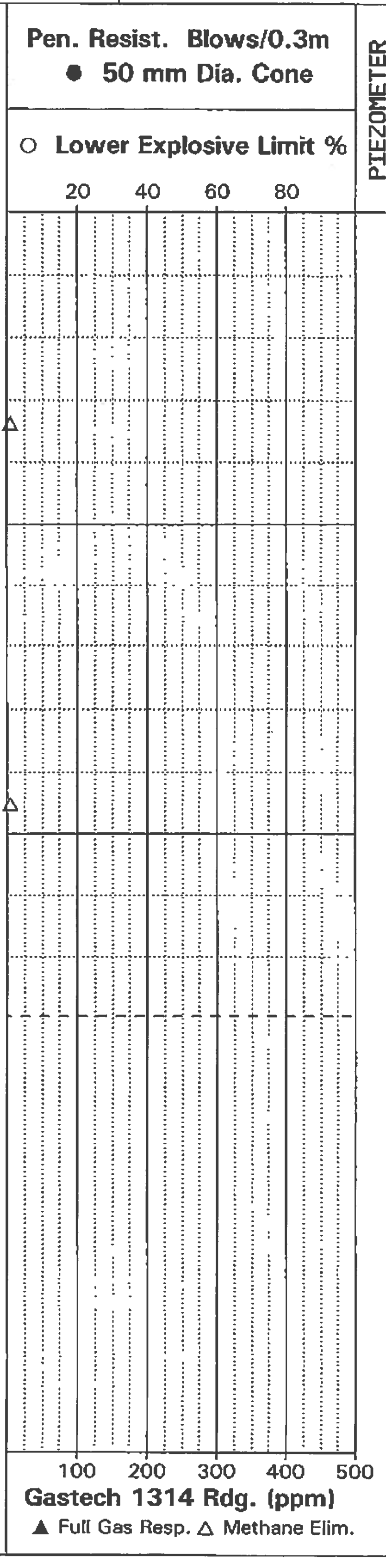
BORINGS BY Backhoe

DATE 21 OCT 02

FILE NO. **E2283**

HOLE NO. **TP 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Mixture of sand, gravel, boulders, brick and wood pieces	▨	G	1				
						1	
		G	2				
2.13						2	
GLACIAL TILL: Dense silty sand and gravel	⁘						
2.59							
End of Test Pit (TP dry upon completion)							



PIEZOMETER CONSTRUCTION

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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located at the southeast corner of Winona Ave. and Scott St. Assumed elevation = 100.00m.

REMARKS

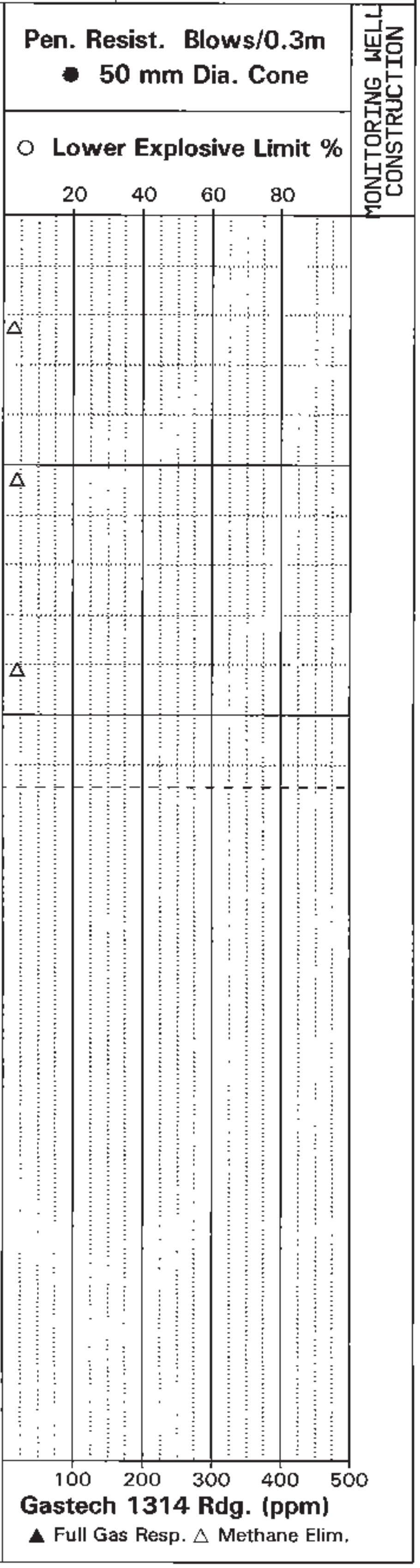
BORINGS BY CME 55 Power Auger

DATE 15 OCT 01

FILE NO. E2283

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.71
Asphaltic concrete 0.08							
FILL: Crushed stone 0.25							
FILL: Black silty sand with gravel - light brown by 0.45m depth		AU	1				
		SS	2	33	6	1	97.71
		SS	3	21	4		
						2	96.71
2.29 End of Borehole Auger refusal on inferred bedrock surface @ 2.29m depth (BH dry upon completion)							



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located at the southeast corner of Winona Ave. and Scott St. Assumed elevation = 100.00m.

REMARKS

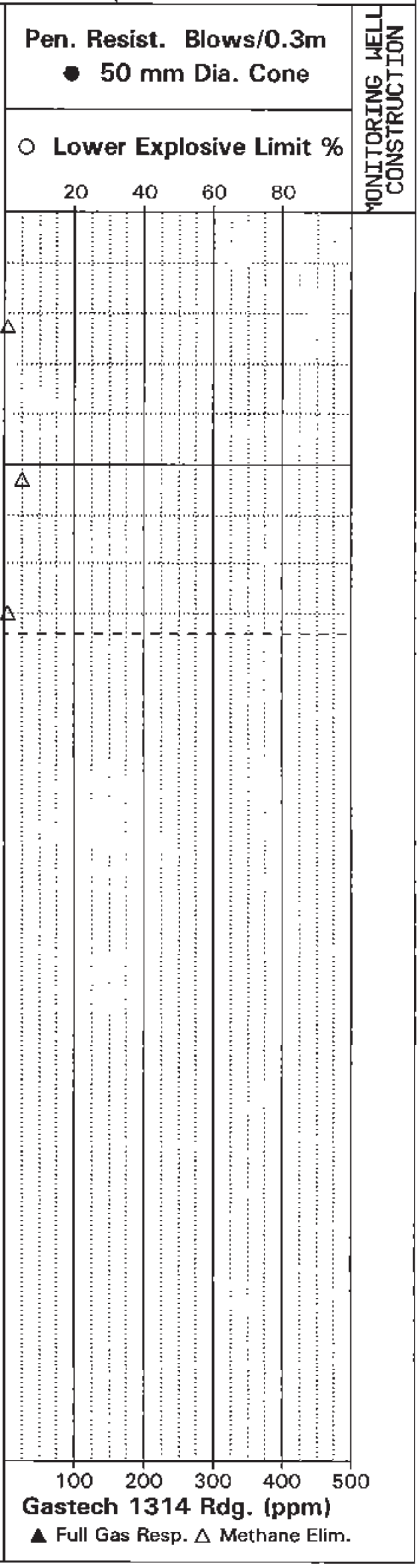
BORINGS BY CME 55 Power Auger

DATE 15 OCT 01

FILE NO. E2283

HOLE NO. BH 2

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	98.54
Asphaltic concrete	0.08							
FILL: Crushed stone	0.23	▨						
FILL: Brown silty sand and gravel		▨	AU	4				
		▨	SS	5	33	12	1	97.54
	1.68	▨	SS	6	100	25		
End of Borehole Auger refusal on inferred bedrock surface @ 1.68m depth (BH dry upon completion)								



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located at the southeast corner of Winona Ave. and Scott St. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 15 OCT 01

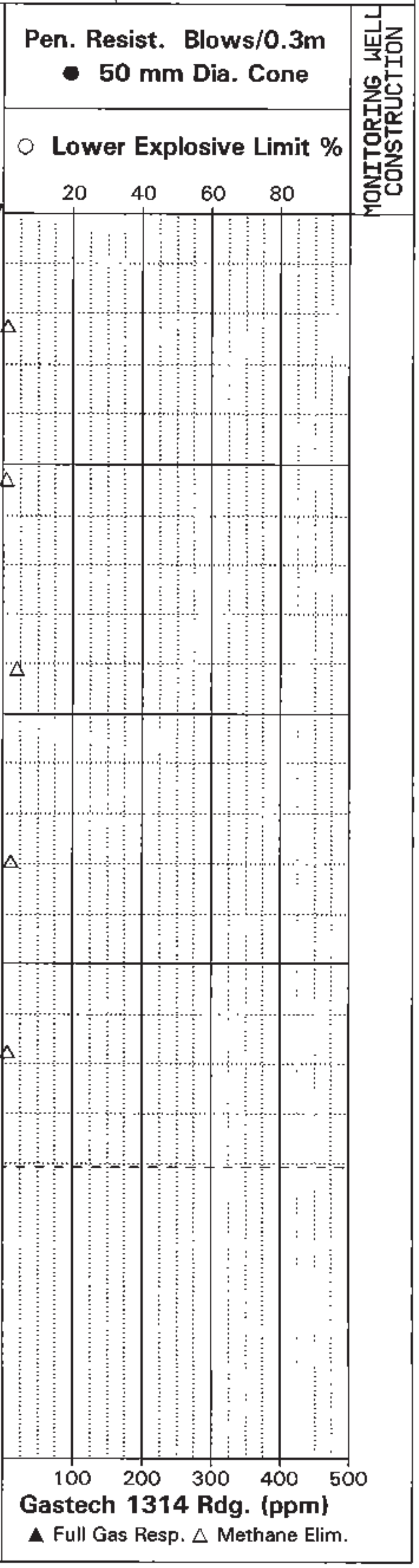
FILE NO. **E2283**

HOLE NO. **BH 3**

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.37
Asphaltic concrete	0.08						
FILL: Brown silty sand and gravel, some cobbles and brick pieces		AU	7				
		SS	8	42	17	1	99.37
		SS	9	25	10		
						2	98.37
		SS	10	17	7		
						3	97.37
		SS	11	17	22		
End of Borehole	3.81						

Auger refusal on inferred bedrock surface @ 3.81m depth

(BH dry upon completion)



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located at the southeast corner of Winona Ave. and Scott St. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 15 OCT 01

FILE NO. E2283

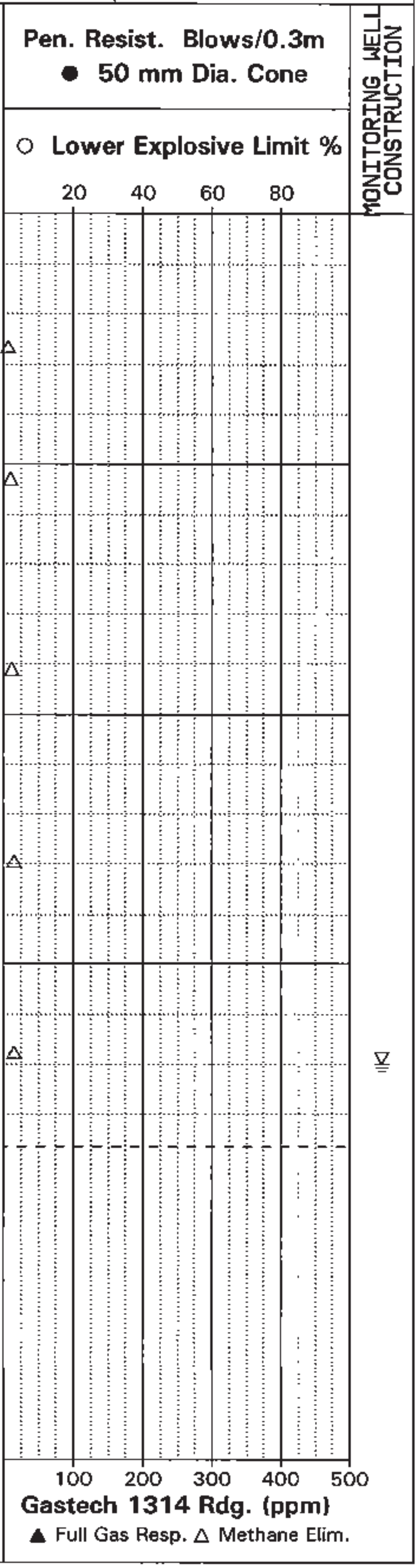
HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.51
Asphaltic concrete 0.08						
FILL: Brown sand and gravel 0.60	AU	12				
FILL: Brown silty sand	SS	13	38	7	1	99.51
	SS	14	42	3	2	98.51
	SS	15	38	5		
					3	97.51
	SS	16	42	5		
3.73 End of Borehole						

End of Borehole

Auger refusal on inferred bedrock surface @ 3.73m depth

(Open hole GWL @ 3.4m depth)



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
2070 Scott Street
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located at the southeast corner of Winona Ave. and Scott St. Assumed elevation = 100.00m.

REMARKS

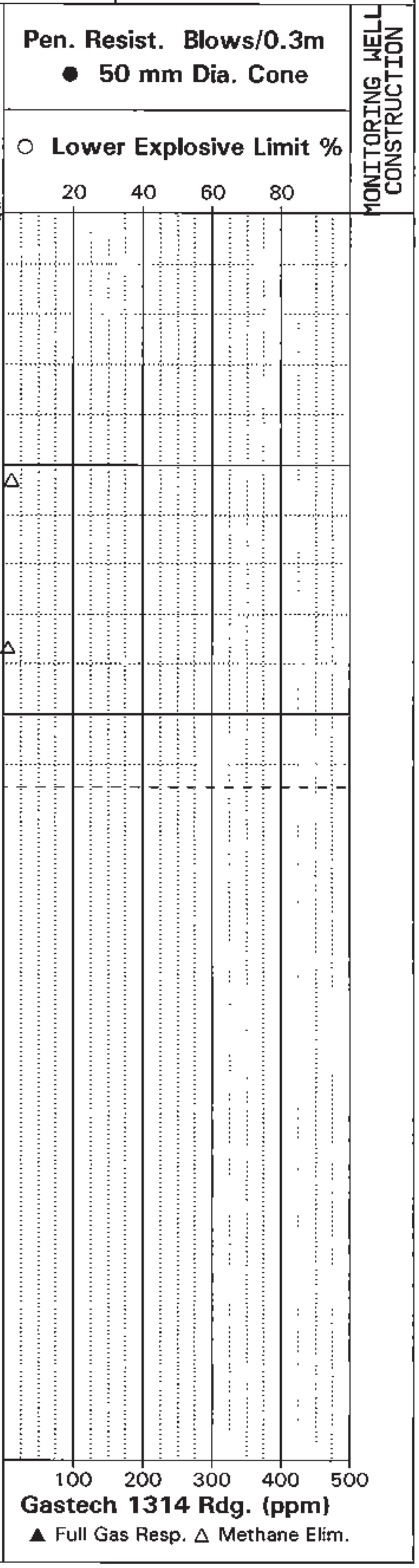
BORINGS BY CME 55 Power Auger

DATE 15 OCT 01

FILE NO. **E2283**

HOLE NO. **BH 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.74
Asphaltic concrete 0.08							
FILL: Brown silty sand and gravel		SS	17	42	29	1	99.74
		SS	18	71	17		
						2	98.74
2.29 End of Borehole Auger refusal on inferred bedrock surface @ 2.29m depth (BH dry upon completion)							



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SOIL PROFILE & TEST DATA

Environmental Site Characterization
Scott Street @ Churchill Avenue North
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located @ the southeast corner of Winona Avenue and Scott Street. Assumed elevation = 100.00m.

REMARKS

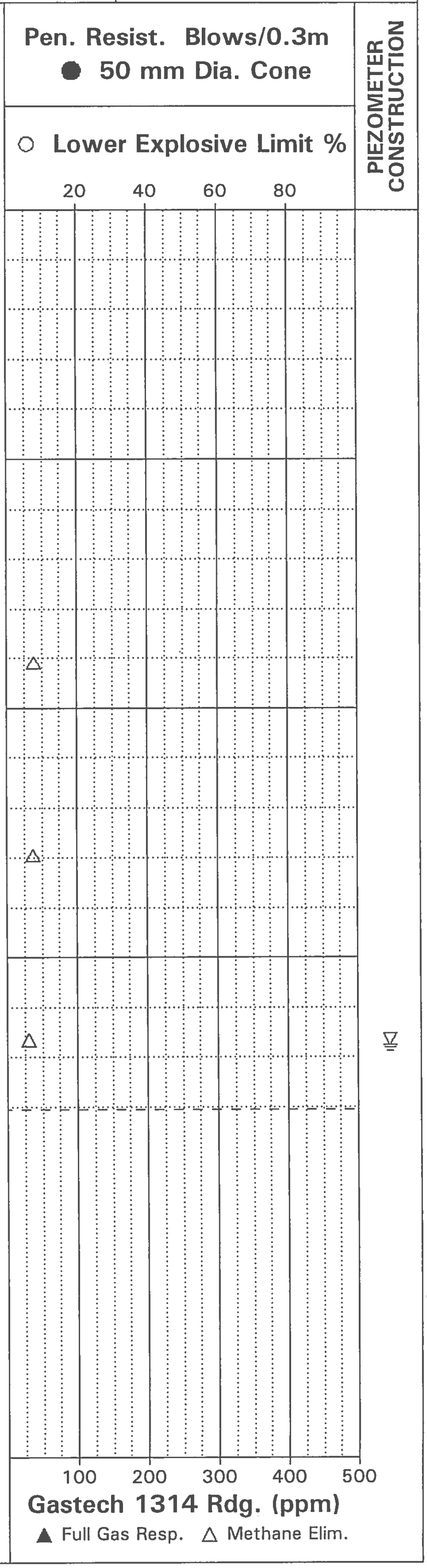
BORINGS BY Power Auger

DATE 18 November 1996

FILE NO. **E1381**

HOLE NO. **BH 1-1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.48
15mm Asphaltic concrete over grey crushed stone 0.19							
						1	99.48
FILL: Brown, mixture of silt, sand, gravel, occ. pcs. of asphaltic concrete		SS	1	62	14		
						2	98.48
		SS	2	50	6		
						3	97.48
3.61		SS	3	54	2		
End of Borehole Auger refusal on inferred bedrock @ 3.61m depth. (Open hole WL @ 3.35m depth.)							



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SOIL PROFILE & TEST DATA

Environmental Site Characterization
Scott Street @ Churchill Avenue North
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located @ the southeast corner of Winona Avenue and Scott Street. Assumed elevation = 100.00m.

REMARKS

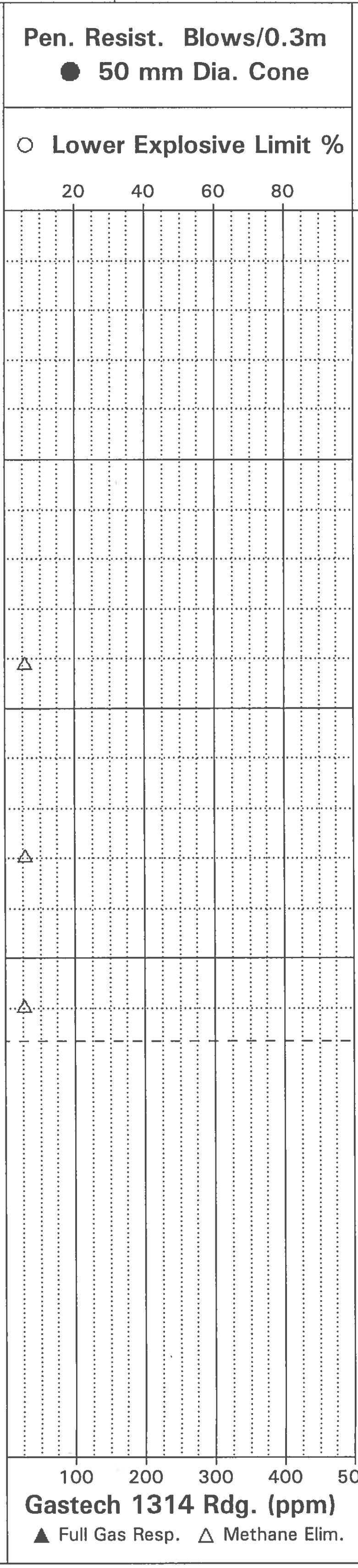
BORINGS BY Power Auger

DATE 18 November 1996

FILE NO. **E1381**

HOLE NO. **BH 2-1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
GROUND SURFACE						0	100.70	
Asphaltic concrete 0.05								
Grey crushed stone 0.25								
						1	99.70	
FILL: Brown, mixture of silt, sand, gravel, occ. pcs. of asphaltic concrete		SS	4	67	6			
						2	98.70	
		SS	5	54	6			
						3	97.70	
		SS	6	45	25+			
3.33								
End of Borehole Auger refusal on inferred bedrock @ 3.33m depth. (BH dry upon completion)								



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SOIL PROFILE & TEST DATA

Environmental Site Characterization
Scott Street @ Churchill Avenue North
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located @ the southeast corner of Winona Avenue and Scott Street. Assumed elevation = 100.00m.

REMARKS

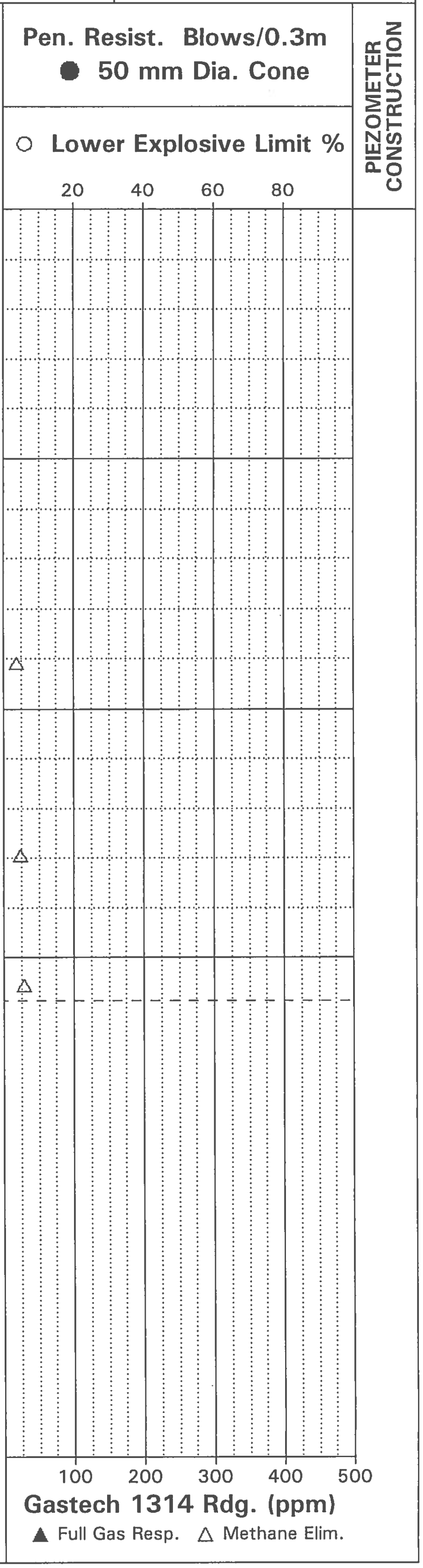
BORINGS BY Power Auger

DATE 18 November 1996

FILE NO. E1381

HOLE NO. BH 3-1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.37
Asphaltic concrete 0.05							
Grey crushed stone 0.25							
						1	99.37
FILL: Brown, mixture of silt, sand, gravel, some organics, occ. pcs. of asphaltic concrete		SS	7	38	11		
						2	98.37
		SS	8	33	10		
						3	97.37
3.17		SS	9	100			
End of Borehole Auger refusal on inferred bedrock @ 3.17m depth. (BH dry upon completion)							



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SOIL PROFILE & TEST DATA

Environmental Site Characterization
Scott Street @ Churchill Avenue North
Ottawa, Ontario

DATUM TBM - Top nut of fire hydrant located @ the southeast corner of Winona Avenue and Scott Street. Assumed elevation = 100.00m.

REMARKS

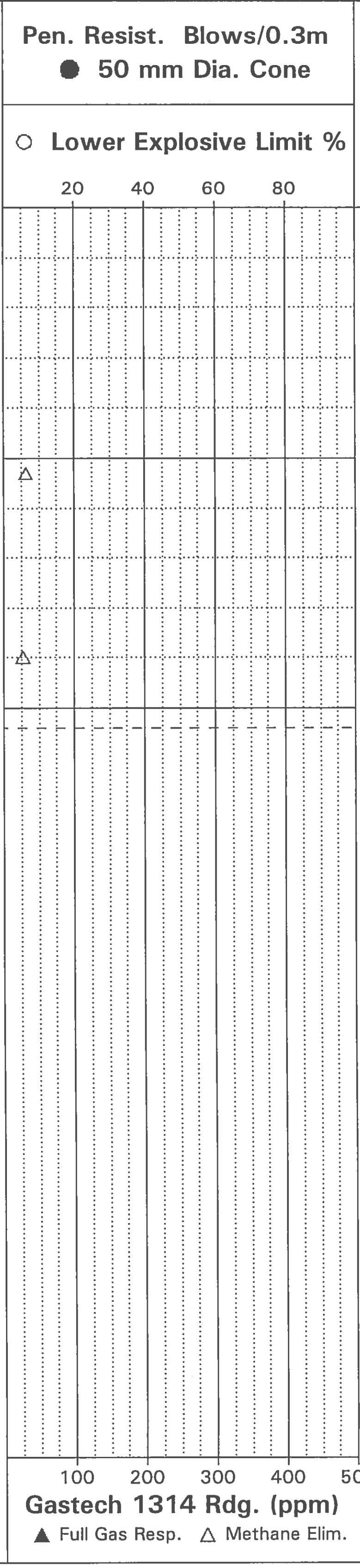
BORINGS BY Power Auger

DATE 18 November 1996

FILE NO. E1381

HOLE NO. BH 4-1

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.73
Asphaltic concrete	0.05						
Grey crushed stone	0.13						
FILL: Dark brown, mixture of silt, sand, gravel, some organics, occ. pcs. of brick							
						1	97.73
		SS	10	58	16		
		SS	11	36	25+		
						2	96.73
	2.08						
End of Borehole Auger refusal on inferred bedrock @ 2.08m depth. (BH dry upon completion)							



PIEZOMETER CONSTRUCTION

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

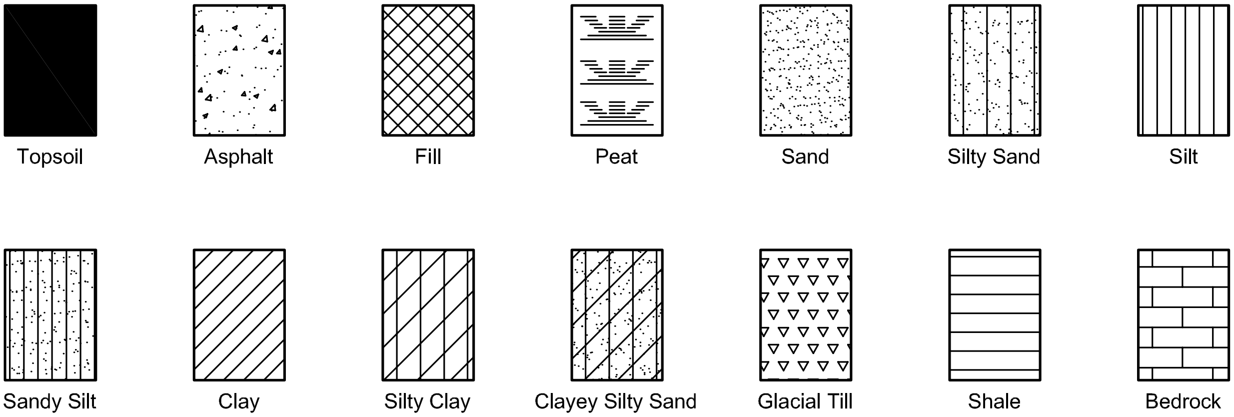
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

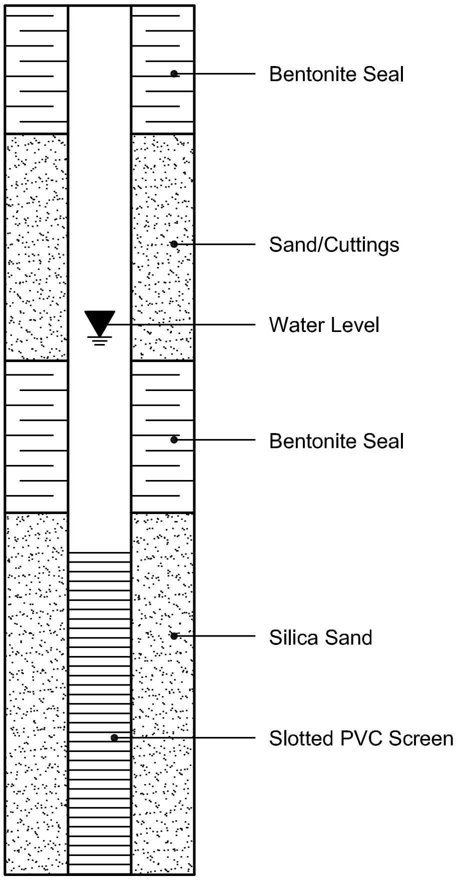
SYMBOLS AND TERMS (continued)

STRATA PLOT

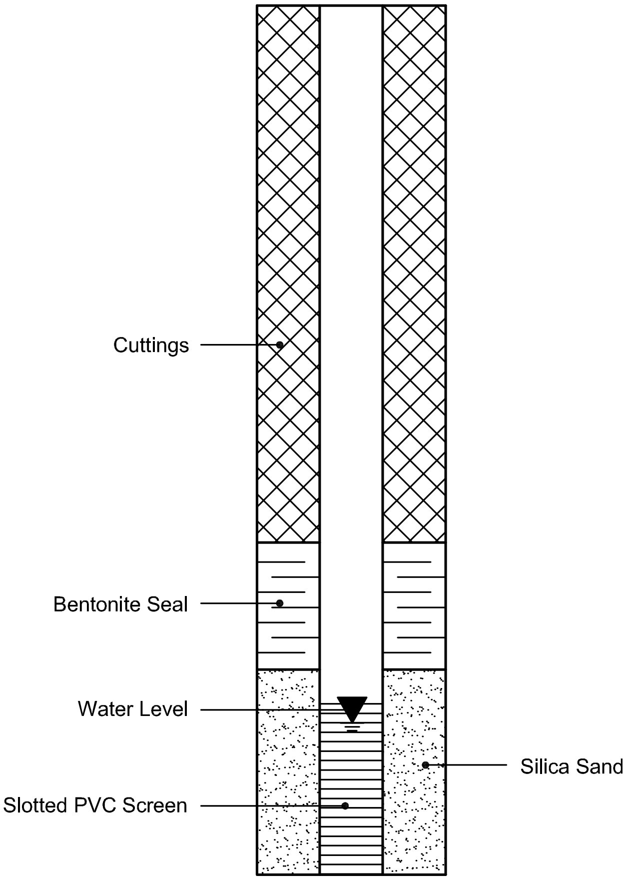


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



BOREHOLE/MONITORING WELL #: BH/MW1

BOREHOLE LOG

Project No: 2471-1301

Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON

Client: EJSpa Corporation

Borehole Location: N5027060 E440905

TOC Elevation: 98.82 m *

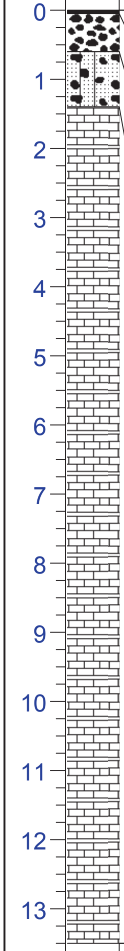
Water Level: 7.18 m btoc (April 11, 2013)

Water Level Elevation: 91.64 m * (April 11, 2013)

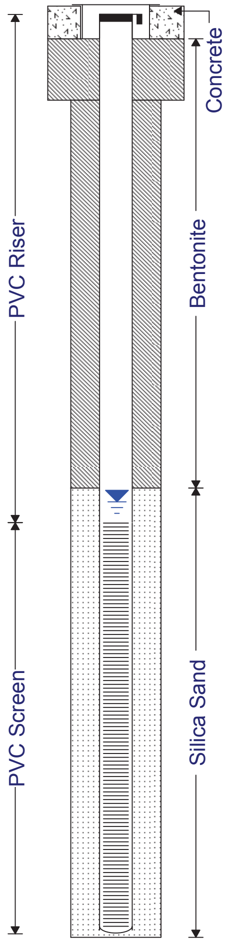
Bottom of Well Depth: 13.15 m btoc

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses
		Ground Surface	98.887 0.000					
		Asphalt						
		Gravel (FILL) grey, dry		N/S	N/S	N/S	N/S	N/S
		Silty Sand and Gravel (FILL) brown, trace organics, dry	97.487 1.400					
		Bedrock limestone						
		End of Borehole	85.387 13.500					



Well Completion Details



Notes:
SS = split spoon sample
ppm = parts per million
m btoc = meters below top of casing
m bgs = meters below ground surface
N/S = no environmental soil sampling performed

*Elevation data based on Franz survey conducted relative to a temporary benchmark (top of yellow and blue fire hydrant on east side of Churchill Avenue North) that was assigned a relative elevation of 100.00 m

MOE Well Cluster Tag No. A140444

Drilled By: George Downing Estate Drilling Ltd.

Drill Method: CME 75 (hollow-stem augers/NQ coring)

Drill Date: April 2, 2013

Logged by: David Kiar

Well Pipe Diameter: 0.03 m

Borehole Diameter: 0.20 m / 0.08 m

Checked by: Mike Grinnell

Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH/MW2

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N5027060 E440905

TOC Elevation: 98.73 m *
Water Level: 7.07 m btoc (April 11, 2013)
Water Level Elevation: 91.67 m * (April 11, 2013)
Bottom of Well Depth: 10.07 m btoc

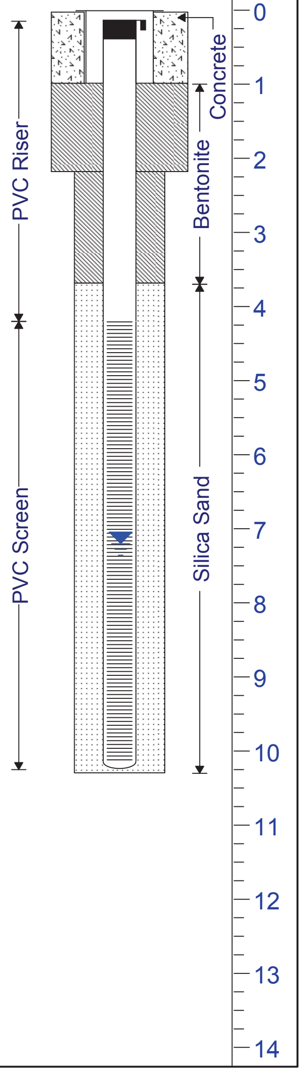
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m) * / Depth (m bgs)
	Ground Surface	98.879
0	***Asphalt***	0.000
	Sand and Gravel (FILL) grey/brown, dry to damp some rock/cobble fragments starting at approximately 1.8 m	
2.2	***Bedrock*** limestone	96.679 2.200
10.3	End of Borehole	88.579 10.300

SAMPLE

Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses
2-1	SS	50	0	PHCs, BTEX, Metals
2-2	SS	60	0	---
2-3	SS	30	0	---
2-4	SS	30	0	---

Well Completion Details



Notes:
SS = split spoon sample
ppm = parts per million
m btoc = meters below top of casing
m bgs = meters below ground surface

*Elevation data based on Franz survey conducted relative to a temporary benchmark (top of yellow and blue fire hydrant on east side of Churchill Avenue North) that was assigned a relative elevation of 100.00 m

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: CME 75 (hollow-stem/NQ coring)
Drill Date: April 4, 2013
Logged by: David Kiar

Well Pipe Diameter: 0.03 m
Borehole Diameter: 0.20 m / 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH/MW3

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N5027094 E440940

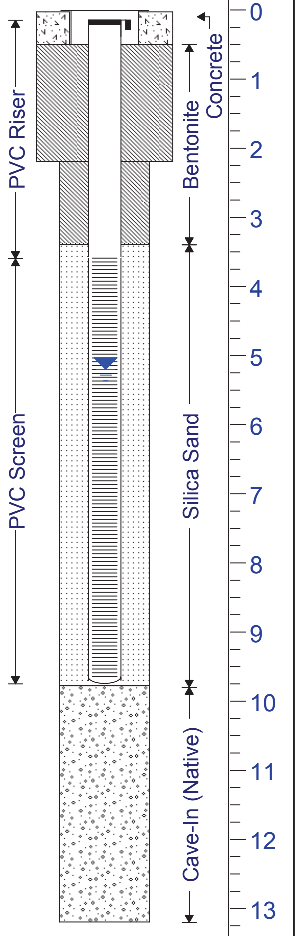
TOC Elevation: 96.85 m *
Water Level: 5.20 m btoc (April 11, 2013)
Water Level Elevation: 91.65 m * (April 11, 2013)
Bottom of Well Depth: 9.65 m btoc

SUBSURFACE PROFILE				SAMPLE				
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses
		Ground Surface	96.904					
0		**Asphalt**	0.000					
1		**Silty Sand and Gravel (FILL)** brown, trace organics, dry to damp		N/S	N/S	N/S	N/S	N/S
2		**Bedrock** limestone	94.704 2.200					
13		End of Borehole	83.704 13.200					

Notes:
SS = split spoon sample
ppm = parts per million
m btoc = meters below top of casing
m bgs = meters below ground surface
N/S = no environmental soil sampling performed

*Elevation data based on Franz survey conducted relative to a temporary benchmark (top of yellow and blue fire hydrant on east side of Churchill Avenue North) that was assigned a relative elevation of 100.00 m

Well Completion Details



Drilled By: George Downing Estate Drilling Ltd.
Drill Method: CME 75 (hollow-stem/NQ coring)
Drill Date: April 4, 2013
Logged by: David Kiar

Well Pipe Diameter: 0.03 m
Borehole Diameter: 0.20 m / 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH4

BOREHOLE LOG

Project No: 2471-1301

Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON

Client: EJSpa Corporation

Borehole Location: N/A

TOC Elevation: N/A (borehole only)

Water Level: N/A (borehole only)

Water Level Elevation: N/A (borehole only)

Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface							No Monitoring Well Installed	
0		***Asphalt***	0.000							0
		Sand and Gravel (FILL) brown/grey, trace silt, clay and cobble, damp to moist.		4-1	SS	30	0	PHCs, BTEX, Metals		
1				4-2	SS	50	0	---		1
				4-3	SS	0	---	---		
		Bedrock Limestone	1.400							
		End of Borehole								
2										2
3										3

Notes:
SS = split spoon sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.

Drill Method: CME 75 (hollow-stem)

Drill Date: April 4, 2013

Logged by: David Kiar

Well Pipe Diameter: N/A

Borehole Diameter: 0.20 m

Checked by: Mike Grinnell

Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH5

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		**Asphalt**	0.000						No Monitoring Well Installed	0
		Silty Sand and Gravel (FILL) brown/grey, some cobble, dry to damp.		5-1	SS	40	0	PHCs, BTEX, Metals		
1				5-2	SS	30	0	---		1
				5-3	SS	0	---	---		
		Bedrock limestone	1.400							
		End of Borehole								
2										2
3										3

Notes:
SS = split spoon sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: CME 75 (hollow-stem)
Drill Date: April 4, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.20 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH6

BOREHOLE LOG

Project No: 2471-1301

Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON

Client: EJSpa Corporation

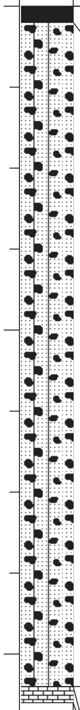
Borehole Location: N/A

TOC Elevation: N/A (borehole only)

Water Level: N/A (borehole only)

Water Level Elevation: N/A (borehole only)

Bottom of Well Depth: N/A (borehole only)



SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		**Asphalt** **Sand and Gravel (FILL)** brown, trace asphalt and brick debris, trace silt and clay, some cobble, damp.	0.000	6-1	SS	60	0	PHCs, BTEX, Metals	No Monitoring Well Installed	0
1				6-2	SS	20	0	---		1
				6-3	SS	0	---	---		
2				6-4	SS	0	---	---		2
		Bedrock limestone	2.100							
		End of Borehole								
3										3

Notes:
SS = split spoon sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.

Drill Method: CME 75 (hollow-stem)

Drill Date: April 4, 2013

Logged by: David Kiar

Well Pipe Diameter: N/A

Borehole Diameter: 0.20 m

Checked by: Mike Grinnell

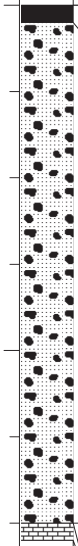
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH7

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		***Asphalt***	0.000						No Monitoring Well Installed	0
		Sand and Gravel (FILL) brown, trace silt and clay, some cobble, dry to damp.		7-1	CS	90	0	---		
1				7-2	CS	90	0	---		1
				7-3	CS	100	0	PHCs, BTEX, Metals		
		Bedrock limestone	1.500							
		End of Borehole								
2										2
3										3

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.09 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH8

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m) * / Depth (m bgs)
0	Ground Surface	
0	***Interlock Brick***	0.000
	Silty Sand and Gravel (FILL) brown, some cobble, damp.	
2.4	***Sandy Silt*** light brown, trace gravel, damp to moist.	2.400
4.6	***Bedrock*** limestone	4.600
	End of Borehole	

SAMPLE

Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses
8-1	CS	50	0	---
8-2	CS	50	0	PHCs, BTEX, Metals
8-3	CS	60	0	---
8-4	CS	50	0	---
8-5	CS	95	0	---

Well Completion Details

No Monitoring Well Installed

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH9

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE					Well Completion Details	Depth (m)
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses		
		Ground Surface								
0		**Grass**	0.000						No Monitoring Well Installed	0
		Topsoil dark brown, organic, damp.								
		Sand and Gravel brown, trace silt and cobble, damp.	0.600	9-1	CS	50	0	PHCs, BTEX, Metals		
1										1
				9-2	CS	100	0	---		
2		**Bedrock** limestone	2.000							2
		End of Borehole								
3										3

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH10

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		**Grass**	0.000						No Monitoring Well Installed	0
		Topsoil dark brown, organic, damp								
		Silty Sand and Gravel brown, some cobble, trace brick debris, damp to moist.	0.600	10-1	CS	70	0	PHCs, BTEX, Metals		
1										1
				10-2	CS	100	0	---		
2		**Bedrock** limestone	2.000							2
		End of Borehole								
3										3

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: NA
Borehole Diameter: 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH11

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		**Asphalt**	0.000						No Monitoring Well Installed	0
		Sand and Gravel (FILL) brown, some silt, some cobble, trace debris, damp.		11-1	CS	50	0	PHCs, BTEX, PAHs, Metals		
1										1
				11-2	CS	25	0	---		
2		**Bedrock** limestone	2.000							2
		End of Borehole								
3										3

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

BOREHOLE/MONITORING WELL #: BH12

BOREHOLE LOG

Project No: 2471-1301
Project: Phase II ESA, 2070-2074 & 2090 Scott Street, Ottawa, ON
Client: EJSpa Corporation
Borehole Location: N/A

TOC Elevation: N/A (borehole only)
Water Level: N/A (borehole only)
Water Level Elevation: N/A (borehole only)
Bottom of Well Depth: N/A (borehole only)

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m) * / Depth (m bgs)	Sample ID	Sample Type	Sample Recovery	Organic Vapour Measurements (ppm)	Lab Analyses	Well Completion Details	Depth (m)
		Ground Surface								
0		***Asphalt***	0.000						No Monitoring Well Installed	0
		Sand and Gravel (FILL) brown, some silt and cobble, trace debris, dry to damp.		12-1	CS	50	150	PHCs, BTEX, PAHs, Metals		
1										1
				12-2	CS	25	0	---		
		Bedrock limestone	1.700							
		End of Borehole								
2										2

Notes:
CS = geoprobe core sample
ppm = parts per million
m bgs = meters below ground surface

Drilled By: George Downing Estate Drilling Ltd.
Drill Method: Geoprobe 7822DT
Drill Date: April 5, 2013
Logged by: David Kiar

Well Pipe Diameter: N/A
Borehole Diameter: 0.08 m
Checked by: Mike Grinnell
Sheet: 1 of 1

patersongroup
Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7 J5
Phone: (613) 226-7381 Fax.: (613) 226-6344

ROCK CORE COMPRESSIVE STRENGTH

CLIENT:	**Slengora Limited c/o Cleland Jardine Engineering**	FILE No.:	**PG2936**
ADDRESS:	472 Tillbury Avenue	REPORT No.:	1
PROJECT:	**2070 & 2090 Scott Street**	DATE:	03-Apr-13

STRUCTURE TYPE & LOCATION: Rock Cores

CORE DATA AND TESTING RESULTS

Lab. No.	**63136M**		**63136M**			
Core No.	1		2			
Location	BH1		BH3			
	RC8		RC8			
Nominal MSA(mm)						
Date Cast						
Date Cored						
Date Tested	03-Apr-13		03-Apr-13			
(D) Ave. Diameter (mm)	46.00		46.00			
(H) Height (mm)	88.0		89.5			
(W) Weight (g)	415.8		417.3			
(A) Area = $\pi D^2/4$ (mm^2)	1661.9		1661.9			
(V) Volume = A X H /1000 (cm^3)	146.2		148.7			
Unit Weight = W /V x1000 (kg/r	**2843**		**2806**			
Capped Height(mm)	88.0		89.5			
H / D ratio	1.91		1.95			
Correction factor (k)	0.992		0.995			
(L) Load (lbs)	46900		23900			
Mpa = L x 4.448222 / A	125.5		64.0			
MPa (corrected)	**124.5**		**63.7**			
Direction of Loading						
Curing Conditions						

REMARKS

Core No.1 Depth: 39' 8" to 40' 0"

Core No.2 Depth: 40' 6" to 40' 10"

DISTRIBUTION

TECHNICAL PERSONNEL

TECHNICIAN: G. Brown

VERIFIED BY: S. Brown

APPROVED BY: Stephen J. Walker, P. Eng.

Certificate of Analysis

Report Date: 08-Apr-2013
Order Date:3-Apr-2013

Client: **Paterson Group Consulting Engineers**
Client PO: 13998
Project Description: PG2936

	Client ID:	BH3-SS2	-	-	-
	Sample Date:	02-Apr-13	-	-	-
	Sample ID:	1314147-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.77	-	-	-
Resistivity	0.10 Ohm.m	29.0	-	-	-
Anions					
Chloride	5 ug/g dry	27	-	-	-
Sulphate	5 ug/g dry	106	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 - GROUNDWATER SUPPRESSION SYSTEM

DRAWING PG4935-1 - TEST HOLE LOCATION PLAN

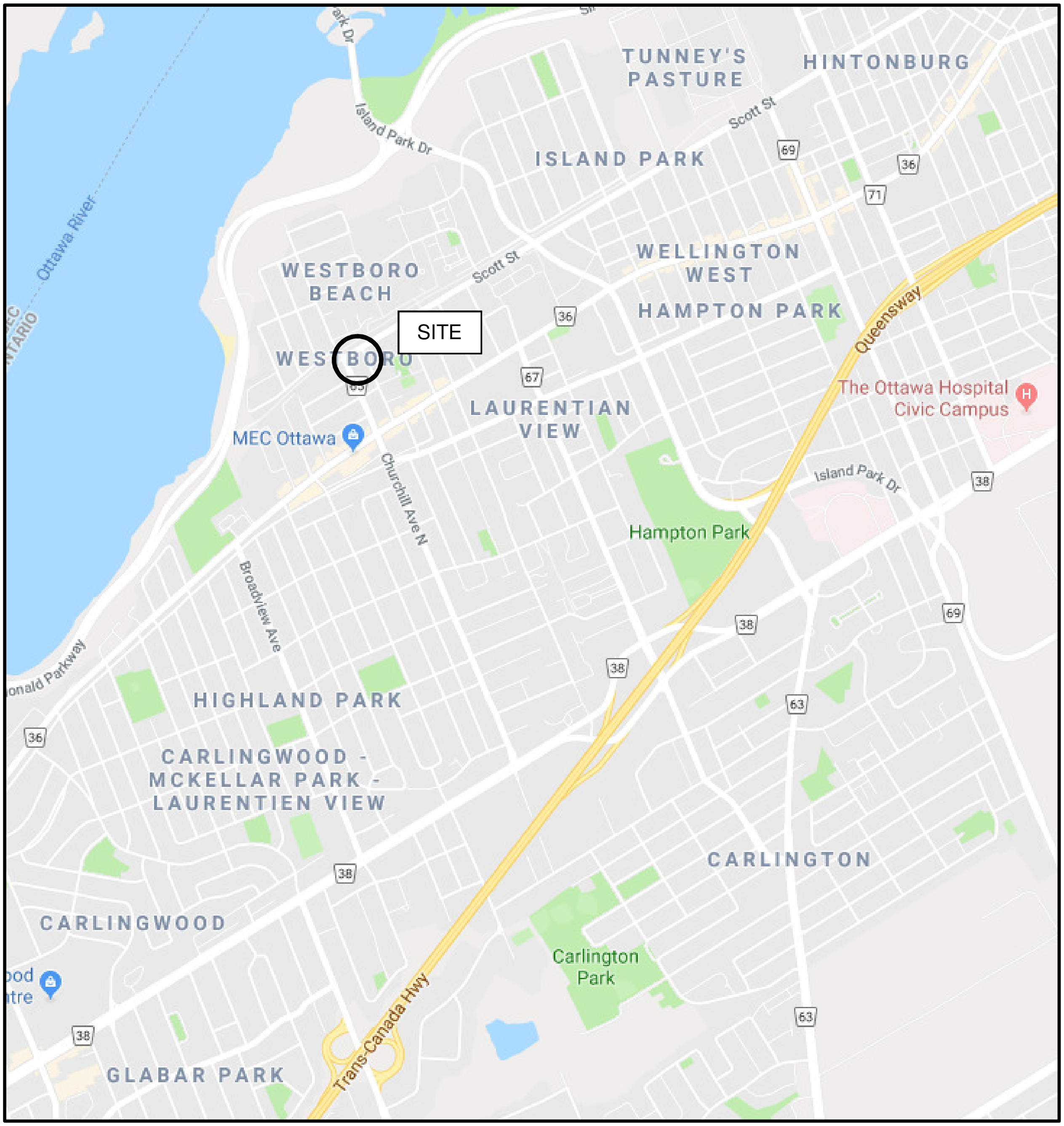


FIGURE 1

KEY PLAN

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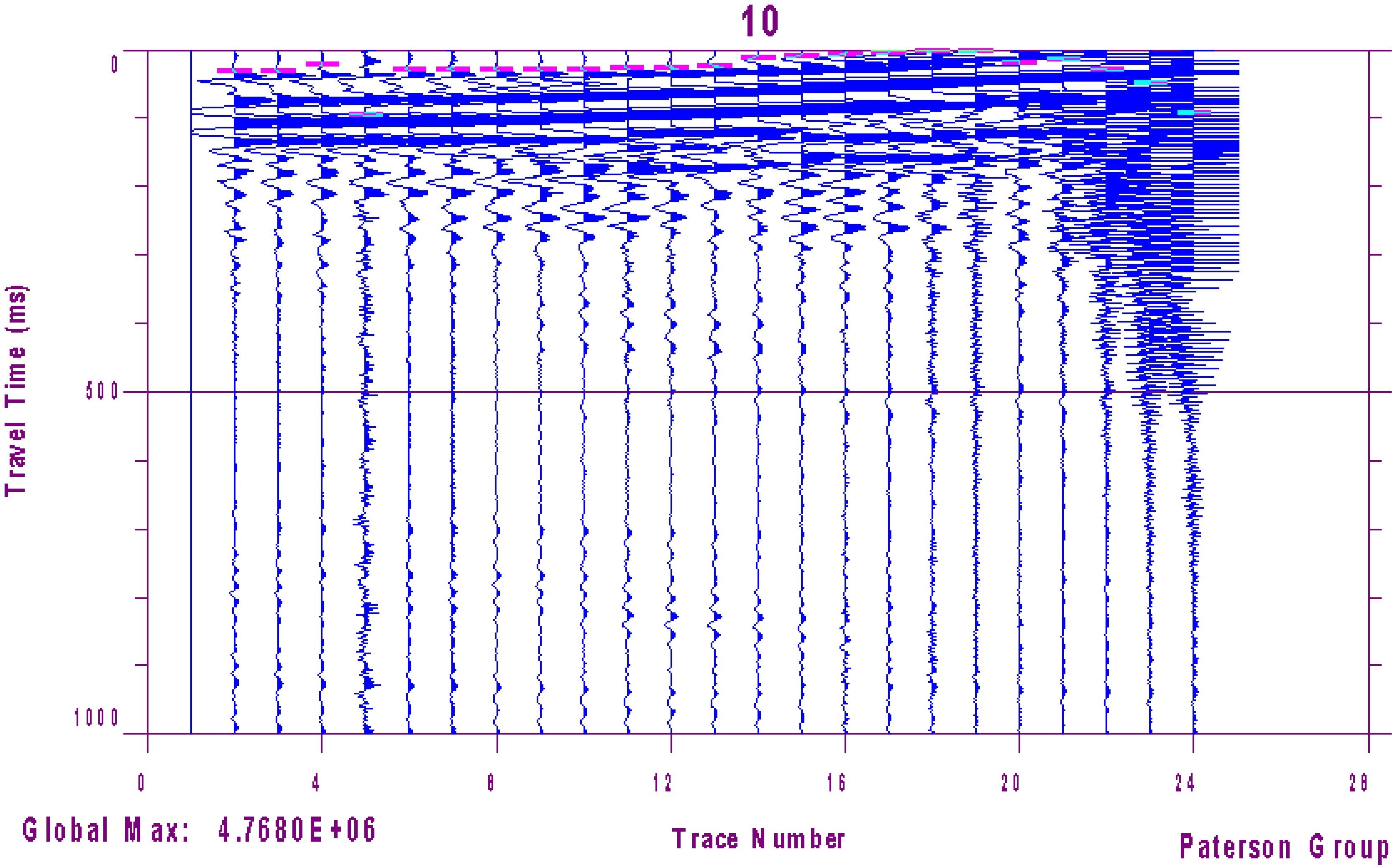


Figure 2 – Shear Wave Velocity Profile at Shot Location 24 m

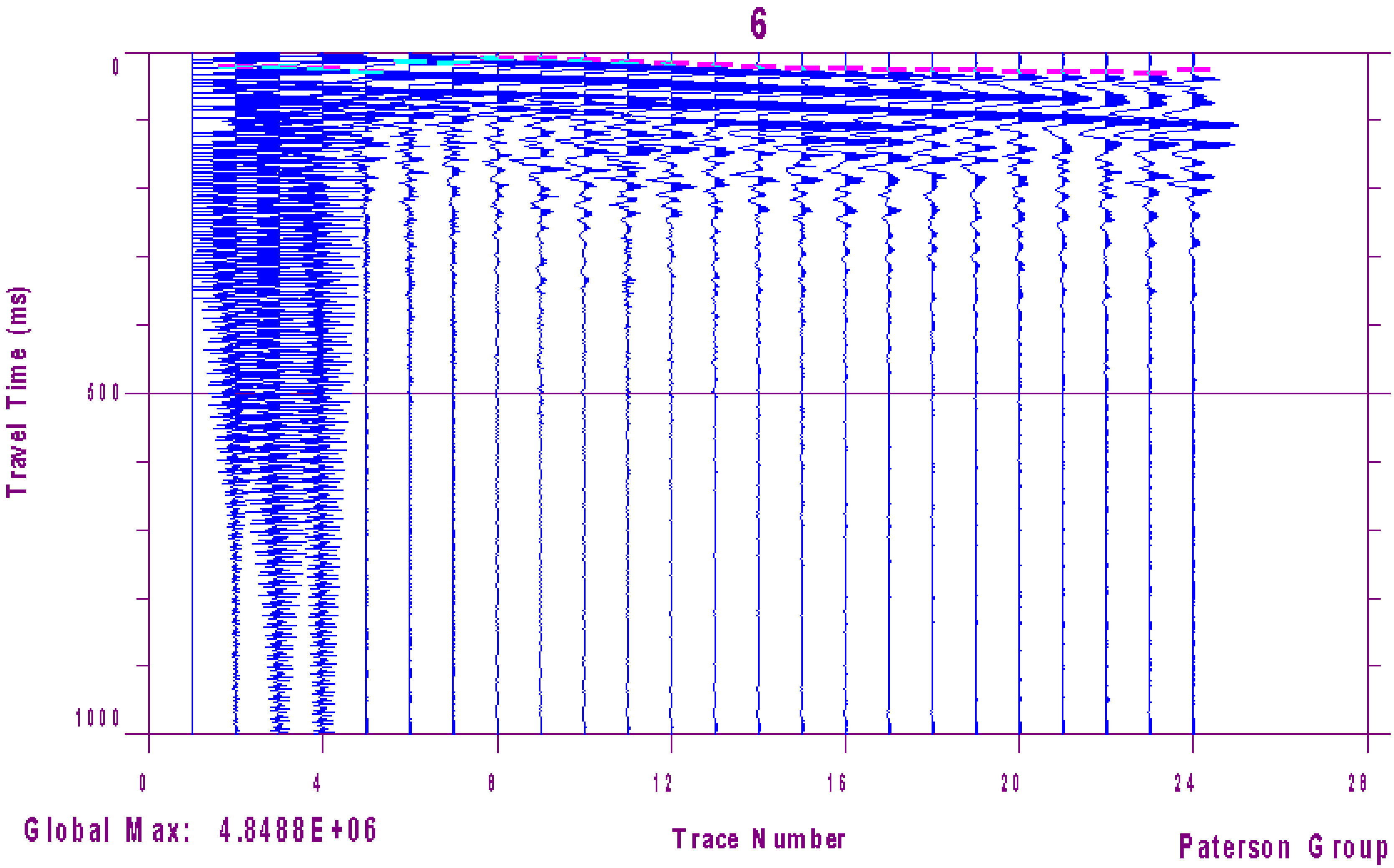
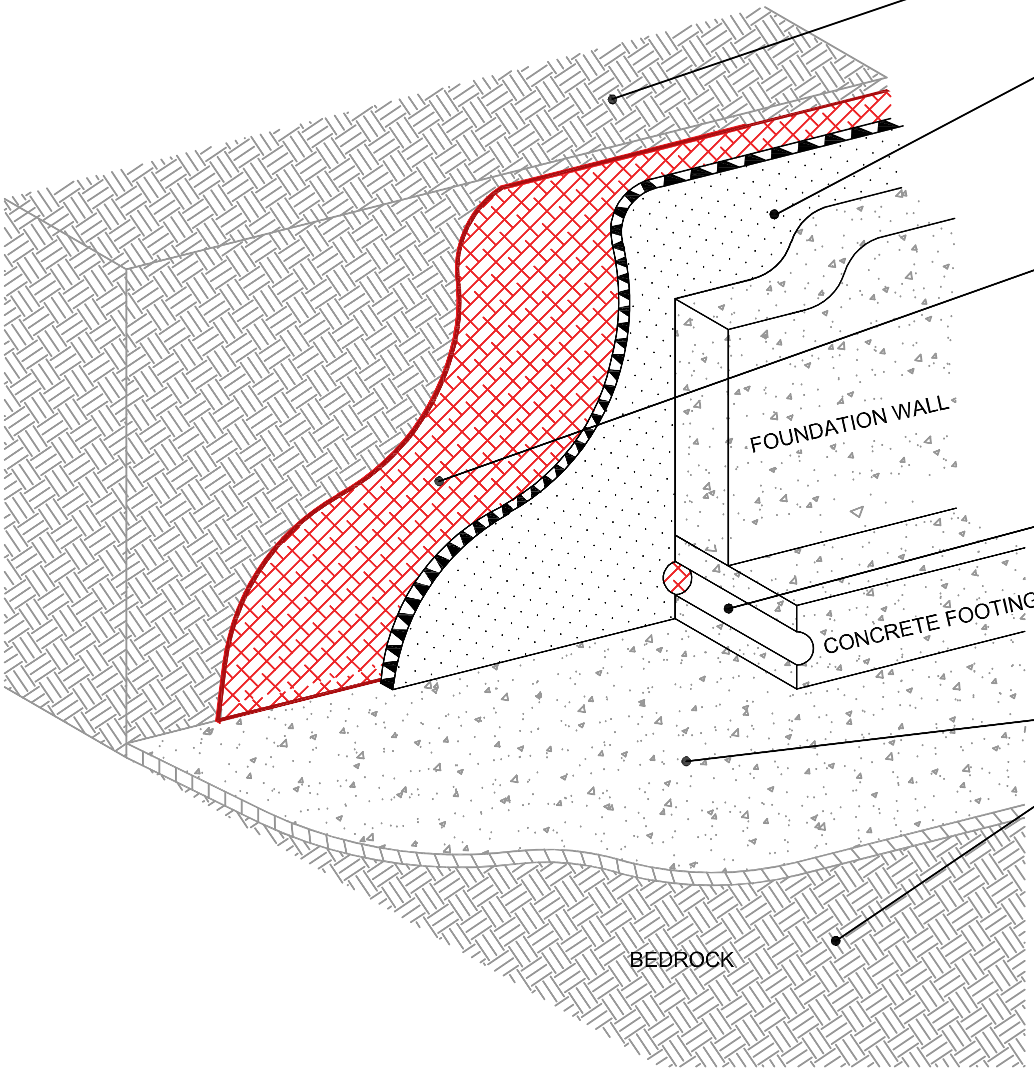


Figure 3 – Shear Wave Velocity Profile at Shot Location -1 m



SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

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

WATERPROOFING MEMBRANE WILL CONSIST OF 150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE SUCH AS TREMCO PARASEAL OR EQUIVALENT. TREMCO PARASEAL TO BE INSTALLED IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATERPROOFING MEMBRANE TO EXTEND FROM 3m BELOW GRADE TO UNDERSIDE OF FOOTING

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE FOOTING AT APPROXIMATELY 3m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE. THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

150mm THICK, 15 MPa CONCRETE MUD SLAB

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

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

patersongroup
consulting engineers

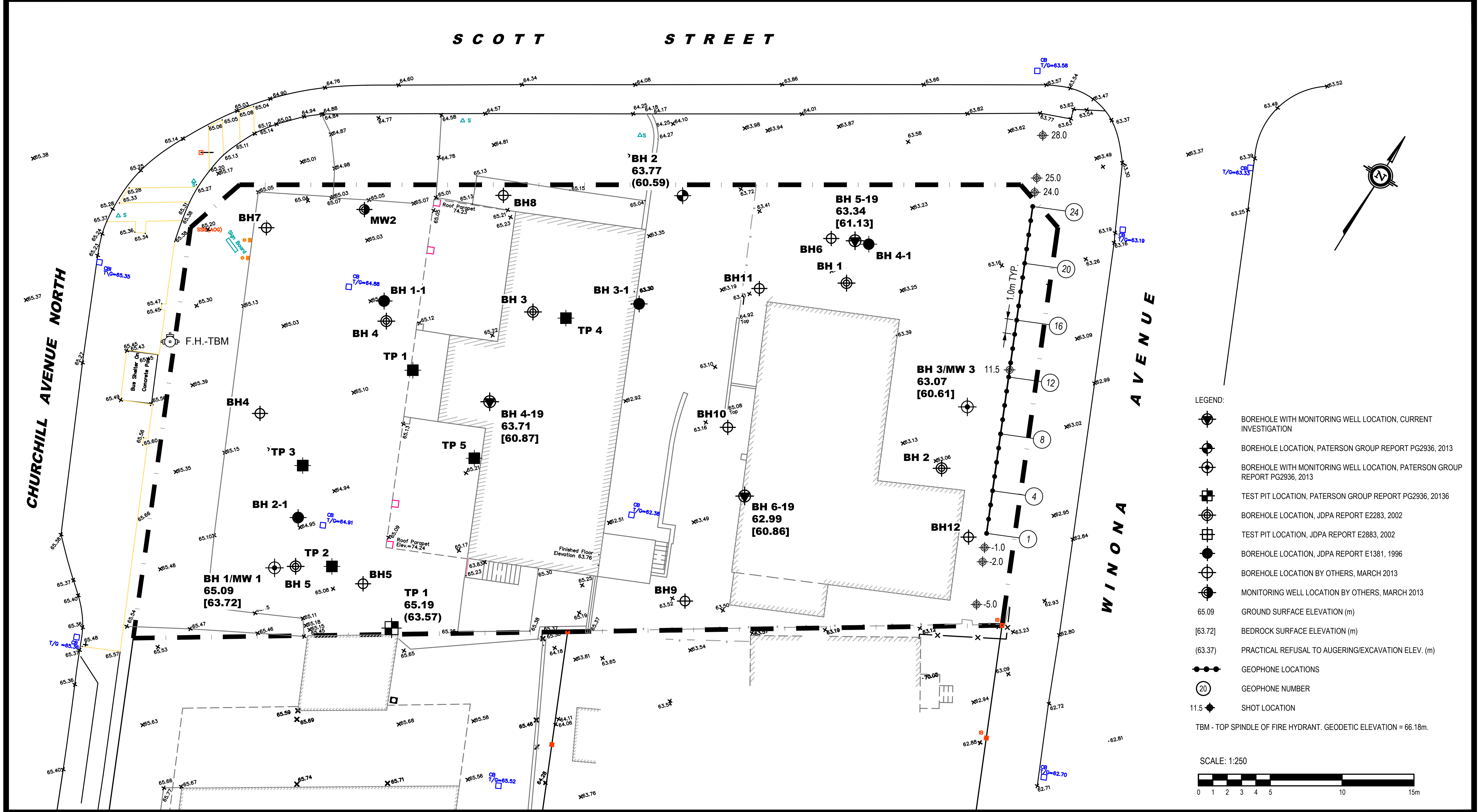
154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344
www.patersongroup.ca

WESTBORO POINT DEVELOPMENTS LTD.
PROPOSED MULTI-STOREY BUILDING
2070 SCOTT STREET
OTTAWA, ONTARIO

Title:

GROUNDWATER SUPPRESSION SYSTEM

Scale: N.T.S.	**Date:** 05/2019
Drawn by: RCG	**Report No.:** PG4935-1
Checked by: RG	**Drawing No.:** FIG. 4
Approved by: CDS	**Revision No.:**



LEGEND:

⊕	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PG2936, 2013
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PG2936, 2013
■	TEST PIT LOCATION, PATERSON GROUP REPORT PG2936, 20136
⊕	BOREHOLE LOCATION, JDPA REPORT E2283, 2002
⊞	TEST PIT LOCATION, JDPA REPORT E2883, 2002
●	BOREHOLE LOCATION, JDPA REPORT E1381, 1996
⊕	BOREHOLE LOCATION BY OTHERS, MARCH 2013
⊕	MONITORING WELL LOCATION BY OTHERS, MARCH 2013
65.09	GROUND SURFACE ELEVATION (m)
[63.72]	BEDROCK SURFACE ELEVATION (m)
(63.37)	PRACTICAL REFUSAL TO AUGERING/EXCAVATION ELEV. (m)
●●●	GEOPHONE LOCATIONS
⑳	GEOPHONE NUMBER
11.5 ⊕	SHOT LOCATION

TBM - TOP SPINDLE OF FIRE HYDRANT. GEODETIC ELEVATION = 66.18m.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

WESTBORO POINT DEVELOPMENTS LTD.

GEOTECHNICAL INVESTIGATION
2070 SCOTT STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 06/2019
Drawn by: MPG	Report No.: PG4935-1
Checked by: SD	Dwg. No.: PG4935-1
Approved by: DJG	Revision No.: 0

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