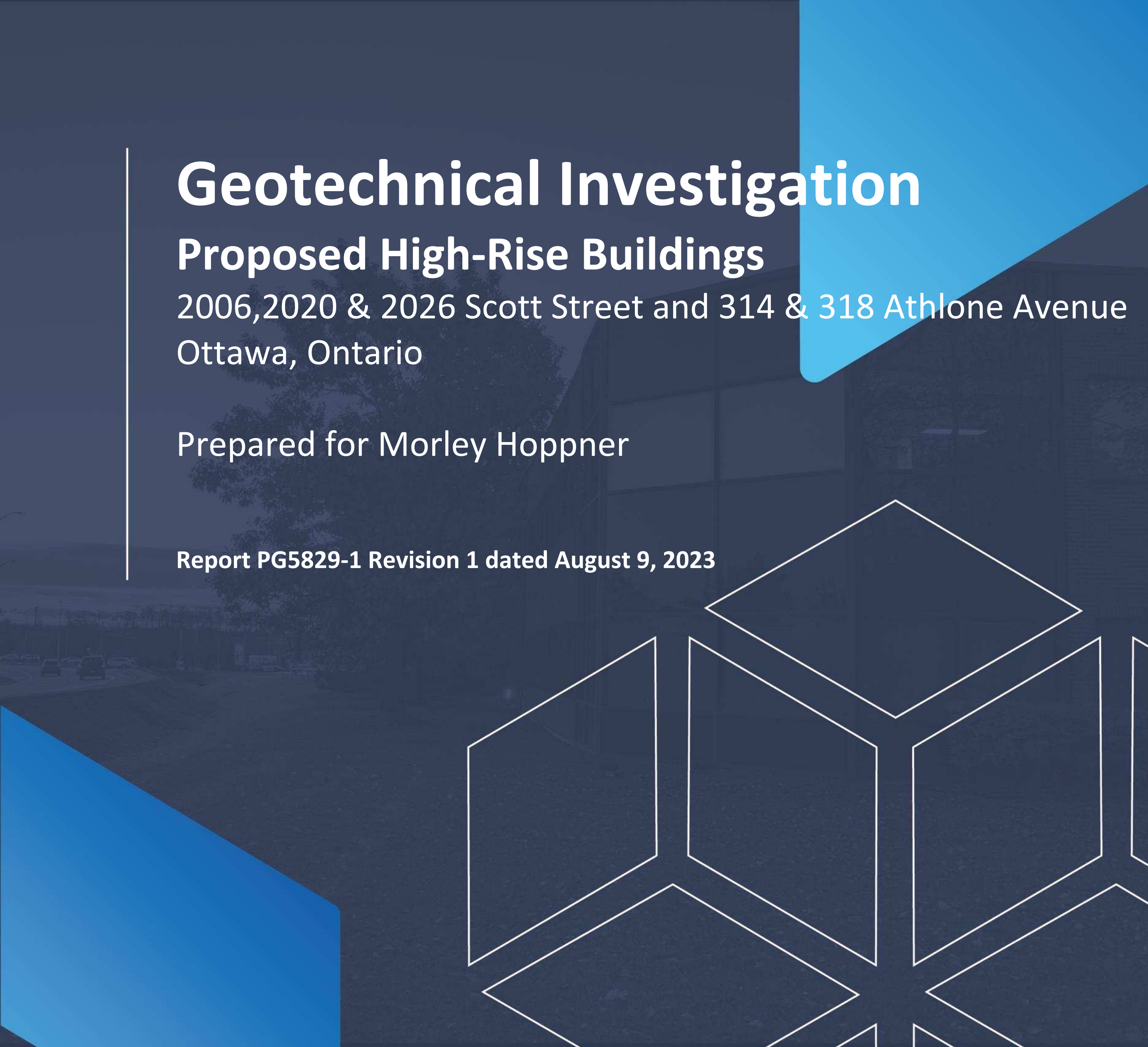

Geotechnical Investigation

Proposed High-Rise Buildings

2006,2020 & 2026 Scott Street and 314 & 318 Athlone Avenue
Ottawa, Ontario

Prepared for Morley Hoppner

Report PG5829-1 Revision 1 dated August 9, 2023

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Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG5829 -1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Morley Hoppner to conduct a geotechnical investigation for the proposed high-rise buildings to be located at 2006, 2020, & 2026 Scott Street and 314 & 318 Athlone Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

- ☐ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings and correspondence with the client, it is understood that the proposed development will consist of two (2) high-rise buildings, Buildings A and B, which will have 3 and 4 levels of underground parking, respectively. Associated access lanes, parking areas, walkways, and landscaped areas are also anticipated as part of the development. It is expected that the proposed buildings will be municipally serviced.

It is further anticipated that the proposed development will involve demolition of the existing structures at the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between May 25 and 26, 2020 and consisted of 11 boreholes advanced to a maximum depth of 6.2 m below the existing ground surface. A previous environmental investigation was completed at the subject site by Paterson on April 29, 2005, and included 9 boreholes which were advanced to a maximum depth of 4.9 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5829-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split- spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Bedrock samples were recovered using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in 5 boreholes (BH 1-21 through BH 4-21, and BH 8-21) and flexible polytube piezometers were installed in the remaining 6 boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program. One monitoring well was also installed as part of the 2005 investigation. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole locations are presented on Drawing PG5829-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil and bedrock samples will be stored for a period of one month after this report is completed, unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site, which consists of 5 contiguous properties, is occupied by a single-storey recreational building (2026 Scott Street), single-storey commercial buildings (2006 & 2020 Scott Street), and low-rise residential buildings (314 & 318 Athlone Avenue). The remainder of the site is generally occupied by associated asphalt-paved parking areas and landscaped areas.

The site is bordered to the north by Scott Street and the Westboro LRT Station, to the east by low-rise residential buildings and Athlone Avenue, to the south by Ashton Avenue and Lion’s Park, and to the west by low to mid-rise residential and commercial buildings. The existing ground surface across the site is relatively level at an approximate geodetic elevation of 62.5 to 63.4 m.

A 1200 mm diameter backbone watermain is located approximately 16 m to the north of the site, underlying Scott Street.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of an approximate 25 to 110 mm thick asphalt surface, 50 to 200 mm thick concrete surface, or topsoil underlain by fill extending to approximate depths of 0.2 to 2.6 m. The fill was generally observed to consist of silty sand and crushed stone with some cobbles and boulders, trace clay and trace wood fragments.

Bedrock

Practical refusal to augering was encountered at depths ranging from 0.2 to 2.6 m below the existing ground surface. Based on the recovered bedrock cores from boreholes BH 6, BH 1-21 through BH 4-21, and BH 8-21, the bedrock at the subject site consists of fair to excellent quality dolostone with interbedded limestone, generally increasing in quality with depth, and was cored to a maximum depth of 6.2 m below the existing ground surface.

Based on available geological mapping, the bedrock at the subject site consists of interbedded limestone and dolomite of the Gull River formation with a drift thickness of 1 to 3 m.

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

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on June 2, 2021. The observed groundwater levels are summarized in Table 1 below, and on the applicable Soil Profile and Test Data sheets presented in Appendix 1.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1-21	63.37	5.06	58.31	June 2, 2021
BH 2-21	63.37	5.96	57.41	June 2, 2021
BH 3-21	63.22	3.85	59.37	June 2, 2021
BH 4-21	62.63	3.79	58.54	June 2, 2021
BH 8-21	62.75	3.72	59.03	June 2, 2021
BH 6	-	3.00	-	May 10, 2005

Note: Ground surface at test hole locations were surveyed by Paterson and are referenced to a geodetic datum

It should be noted that the groundwater level readings in the monitoring wells can be influenced by surface water becoming trapped in the backfill materials. Based on the observed groundwater levels within the monitoring wells, the groundwater level is anticipated to occur within the bedrock at an approximate depth of 3.5 to 5 m below the existing ground surface.

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

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. The proposed high-rise buildings at the subject site are recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the excavation of the underground parking levels.

In addition, due to the proximity of the 1200 mm diameter backbone watermain, additional precautions should be taken during excavation activities to ensure that the existing service is not affected.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

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.

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed buildings, all existing overburden material should be excavated from within the proposed building footprints.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

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

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Vibration Monitoring Plan for 1200 mm Diameter Backbone Watermain

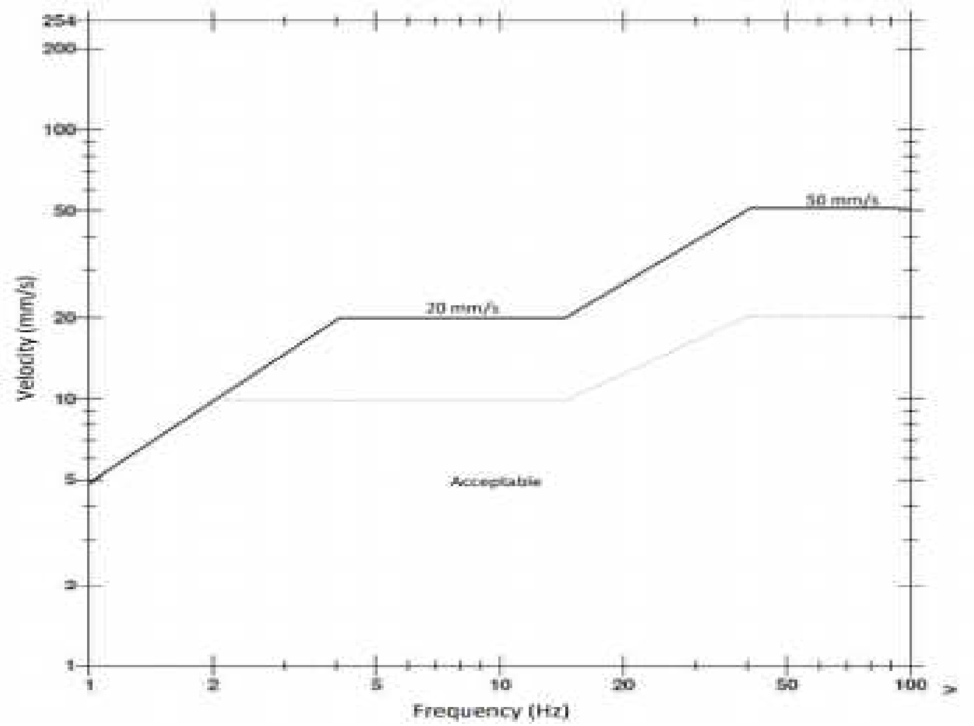
The following vibration monitoring program is recommended to ensure that excessive vibrations do not occur at the nearby watermain location:

- ☐ Vibration levels will be continuously monitored using 2 vibration monitors which are installed at the northern site boundary, which is nearest to the watermain.

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel.

The vibration limits, provided below, are recommended for the construction operation to be completed in the vicinity of the 1200 mm diameter watermain.

Vibration Criteria Figure - Proposed Vibration Limits at the Watermain



The monitoring protocol should include the following information:

Warning Level Event

- ❑ Paterson will review all vibrations over the established warning level, illustrated by the blue line in the above figure, and;
- ❑ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event

- ❑ Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in the above figure.
- ❑ Ensure monitors are functioning
- ❑ Issue the vibration exceedance result

The data collected should include the following:

☐ Measured vibration levels
☐ Distance from the construction activity to monitoring location
☐ Vibration type

Monitoring should be compliant with all related regulations.

Incidence & Exceedance Reporting

In case a vibration incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

☐ The location of vibration exceedance
☐ The date, time and nature of the exceedance/incident
☐ Purpose of the exceeded monitor and current vibration criteria
☐ The likely cause of the exceedance/incident
☐ The response action that has been completed to date
☐ The proposed measures to address the exceedance/incident.

The contractor should implement mitigation measures for future excavation or any construction activities as necessary and provide updates on the effectiveness of the improvement. Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any vibrations to identify issues and react in a quick manner in the event of an exceedance.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed on a sound bedrock bearing surface at depth can also be designed using a higher factored bearing resistance value at ultimate limit states (ULS) of **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5, if founded on bedrock which is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint of the footings. At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant. Also, the above probing program can be omitted if the bedrock side profile in the excavation demonstrates and confirms that the bedrock is sound.

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 wit a rock hammer.

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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or shallower).

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class C**, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012, for foundations considered at this site. A higher seismic class, such as Class A or B, may be achievable for this site, however, a site-specific shear wave velocity test would be required to confirm the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

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

5.5 Basement Slab Construction

For the proposed development, it is anticipated that the overburden soil will be removed from the building footprints, leaving the bedrock as the founding medium for the basement floor slabs. It is anticipated that the basement area for the proposed buildings will be mostly parking and the recommended rigid pavement structure noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrades should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the field investigation, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor slabs. This is discussed further in Subsection 6.1.

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/m3 (effective 15.5 kN/m3) 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 unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressure

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

K_O = At-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the basement 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 exercisedduring the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a \ \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earthforce component (P_o) under seismic conditions can be calculated using:

$P = 0.5\ K \cdot \gamma \cdot H^2$, where $K \ = 0.5$ 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 thewall, 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 shouldbe factored as live loads, as per OBC 2012.

If the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3).

Where soil is retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees anda drained unit weight of 20 kN/m^3.

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

5.7 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60-to-90-degree pullout of rock cone 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 load capacity of each individual anchor. A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone and dolostone ranges between about 50 and 80 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 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2:

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 Dolostone/Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone/Dolostone edrock	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3 below. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

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	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

5.8 Pavement Design

Flexible Pavement Structures

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5 below:

Table 4 - Recommended Flexible 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 Flexible Pavement Structure – Access Lanes and Heavy Truck Parking Areas

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

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

The pavement granular (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Rigid Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 6 below. The flexible pavement structure presented in Tables 4 and 5 above should be used for at grade access lanes and heavy loading parking areas.

Table 6 – Recommended Pavement Structure – Lower Parking Level

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

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level.

The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is anticipated that the portion of the proposed building foundation walls located below the long-term groundwater table (approximate depth of 4 m) will be blind poured and placed against a groundwater infiltration control system. Also, a perimeter foundation drainage system will be required as a secondary system to manage any groundwater which may breach the waterproofing membrane.

The groundwater infiltration control system for the foundation walls is recommended to consist of the following:

- ☐ Line drill the excavation perimeter (usually a 150 to 200 mm spacing).
- ☐ Mechanical bedrock removal along the foundation walls can be undertaken up to 150 mm from the finished vertical excavation face.
- ☐ Grind the bedrock surface up to the outer face of the line drill holes to ensure a satisfactory surface for the ground infiltration control system.
- ☐ In bedrock overbreaks, shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
- ☐ Place a suitable waterproofing membrane, such as Tremco Paraseal or approved equivalent, against the prepared bedrock surface. The waterproofing membrane should extend from 4 m below existing grade down to footing level.
- ☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- ☐ Pour foundation wall against the composite drainage system.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underslab Drainage System

Underslab drainage will be required to control water infiltration underlying the lowest level floor slab. For preliminary design purposes, we recommend that 150 mm in diameter perforated pipes be placed within at approximate 6 m spacing.

The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings, such as canopy footings.

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes in the overburden and 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.

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.

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 should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

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 design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

The temporary shoring system may consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability.

The toe of the shoring is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre- augered holes if a soldier pile and lagging system is used.

The earth pressures acting on the shoring system may be calculated 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 calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

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 a aggressive to very aggressive corrosive environment.

7.0 Recommendations

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

☐ Grading plan review from a geotechnical perspective, once the final grading plan is available.

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

☐ Sampling and testing of the concrete and fill materials.

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

☐ Observation of all subgrades prior to backfilling.

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

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Morley Hoppner or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Puneet Bandi M.Eng

LICENSED PROFESSIONAL ENGINEER
Aug 9-2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis
Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Morley Hoppner (Digital copy)
- ☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5829

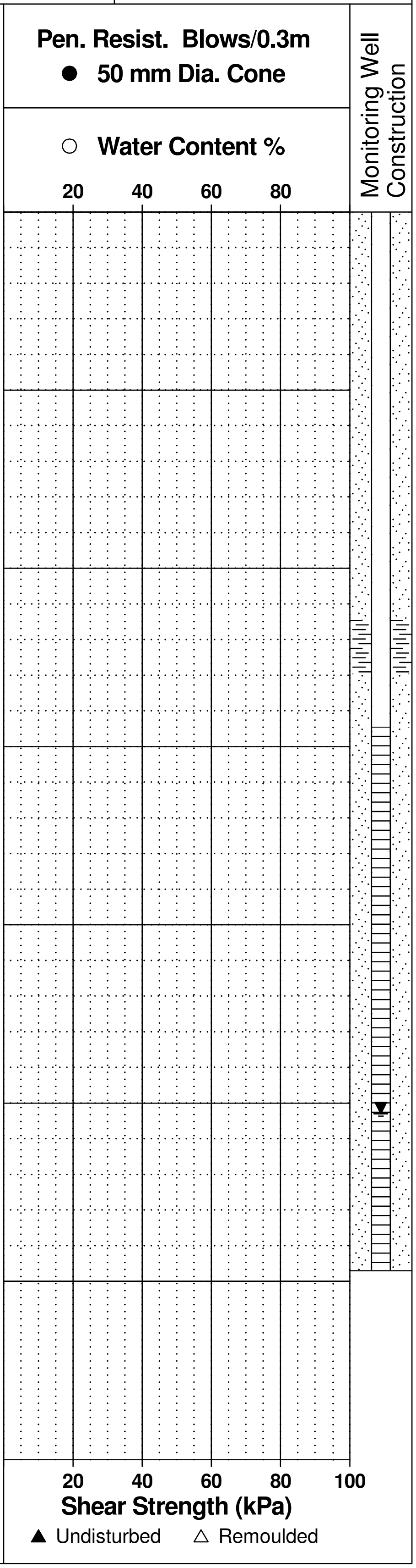
REMARKS

HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill

DATE May 25, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.37
Asphaltic concrete 0.04							
FILL: Brown silty sand with crushed stone		AU	1				
		SS	2	25	25	1	62.37
1.22							
FILL: Brown silty sand, some crushed stone and gravel, trace clay		SS	3	50	50+		
2.03						2	61.37
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with limestone		RC	1	100	71		
						3	60.37
		RC	2	100	100		
						4	59.37
						5	58.37
		RC	3	100	97		
6.00						6	57.37
End of Borehole							
(GWL @ 5.06m - June 2, 2021)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 25, 2021

FILE NO. PG5829

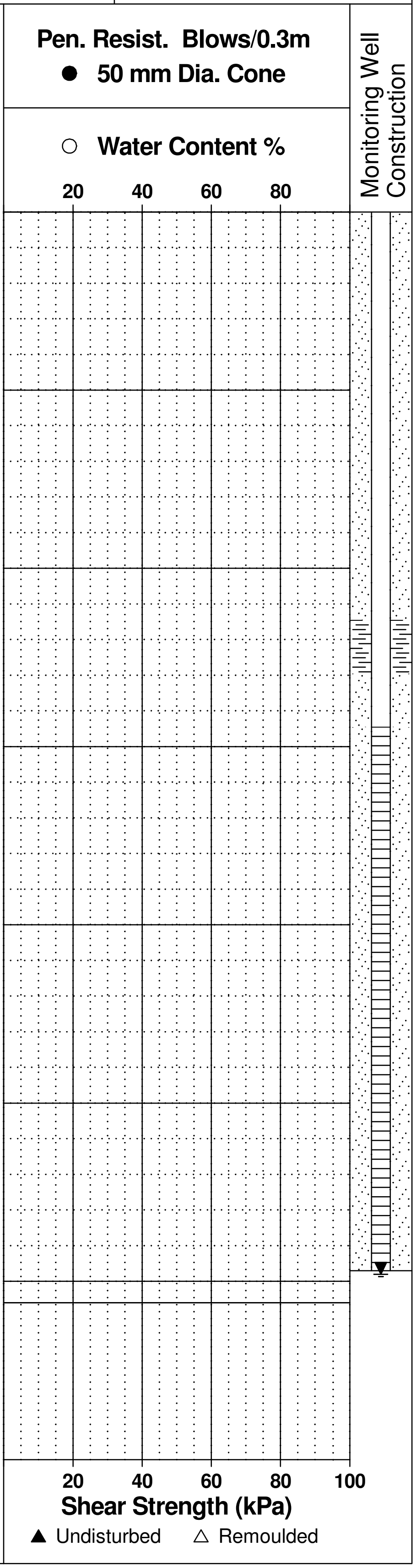
HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Brown silty sand with crushed stone	0.76
FILL: Brown silty sand with crushed stone and gravel, occasional cobbles, boulders and rock fragments	2.57
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with limestone	6.12
End of Borehole	

(GWL @ 5.96m - June 2, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	50+
RC	1	100	85
RC	2	100	93
RC	3	100	100

DEPTH (m)	ELEV. (m)
0	63.37
1	62.37
2	61.37
3	60.37
4	59.37
5	58.37
6	57.37



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

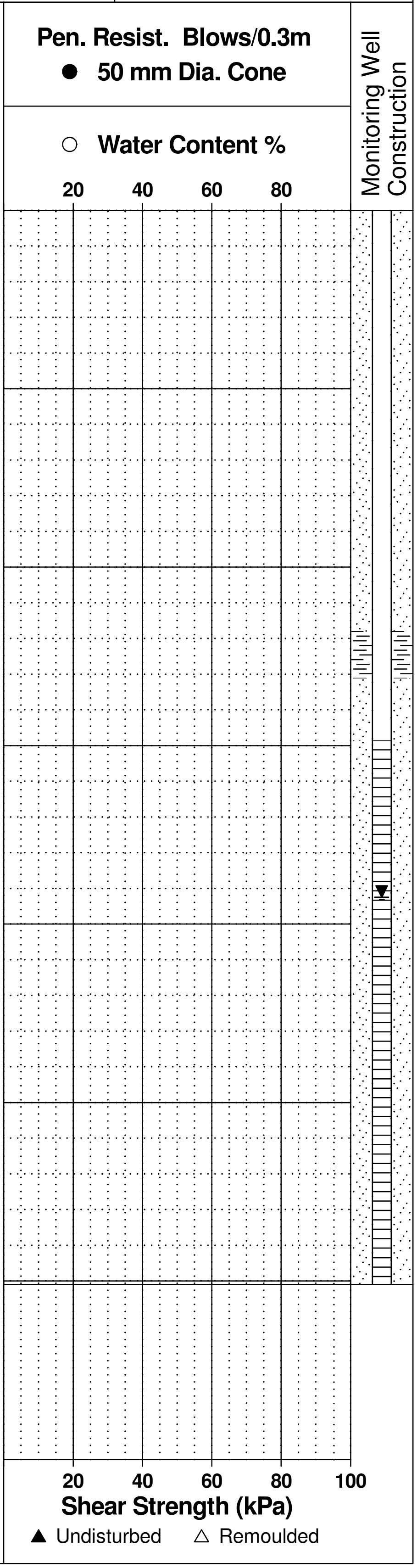
DATE May 25, 2021

FILE NO. PG5829

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with brown silty sand	0.20	AU	1		
FILL: Brown sity sand, some crushed stone and gravel, occasional cobbles	2.08	SS	2	17	18
		SS	3	18	9
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with limestone	6.02	RC	1	100	83
		RC	2	100	92
		RC	3	100	100
End of Borehole (GWL @ 3.86m - June 2, 2021)					

DEPTH (m)	ELEV. (m)
0	63.22
1	62.22
2	61.22
3	60.22
4	59.22
5	58.22
6	57.22



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 25, 2021

FILE NO. PG5829

HOLE NO. BH 4-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: 25mm Asphaltic concrete over crushed stone with silty sand 0.25	AU	1		
	AU	2		
FILL: Brown silty sand with crushed stone and gravel, occasional cobbles	SS	3	58	11
2.13	SS	4	55	17
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with limestone	RC	1	100	84
	RC	2	100	88
6.00	RC	3	100	100
End of Borehole (GWL @ 3.79m - June 2, 2021)				

DEPTH (m)	ELEV. (m)
0	62.63
1	61.63
2	60.63
3	59.63
4	58.63
5	57.63
6	56.63

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

○ **Water Content %**
20 40 60 80

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 25, 2021

FILE NO. PG5829

HOLE NO. BH 5-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	62.96
Asphaltic concrete 0.08						
FILL: Crushed stone with brown silty sand 0.76	AU AU	1 2				
FILL: Brown silty sand with gravel, some clay, occasional cobbles and boulders 1.80	SS SS	3 4	58 20	20 50+	1	61.96

End of Borehole

Practical refusal to augering at 1.80m depth

(Piezometer blocked - June 2, 2021)

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5829

REMARKS

HOLE NO. BH 6-21

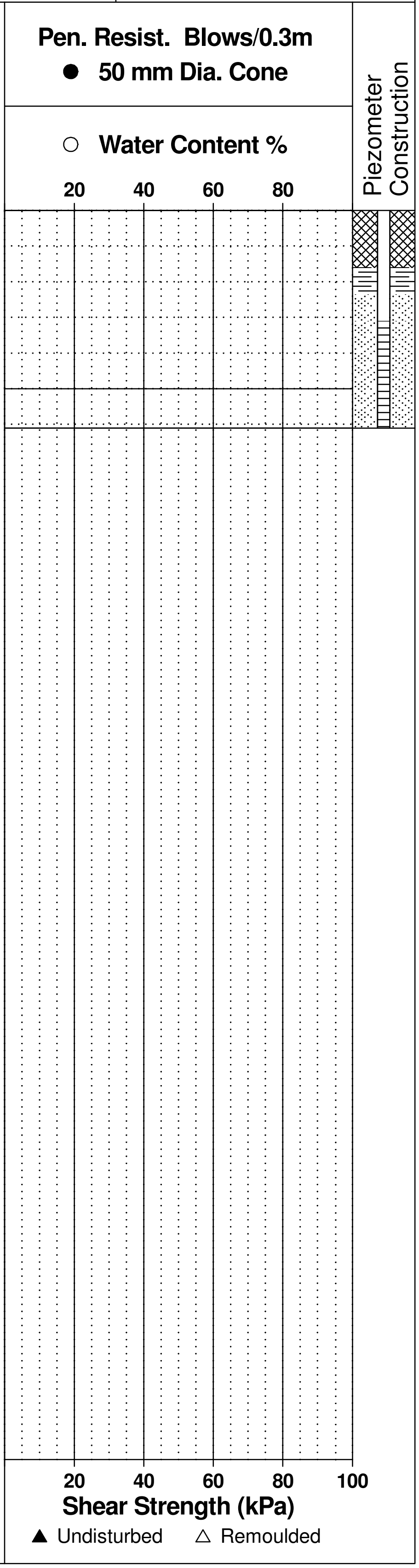
BORINGS BY CME-55 Low Clearance Drill **DATE** May 26, 2021

SOIL DESCRIPTION	Layer bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.55
Asphaltic concrete	0.04						
FILL: Crushed stone, trace sand	0.30	AU	1				
FILL: Brown silty sand with crushed stone, gravel and rock fragments	1.22	SS	2	11	50+	1	61.55

End of Borehole

Practical refusal to augering at 1.22m depth

(Piezometer blocked - June 2, 2021)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5829**

REMARKS

HOLE NO. **BH 7-21**

BORINGS BY CME-55 Low Clearance Drill

DATE May 26, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.85
Asphaltic concrete 0.11		AU	1				
FILL: Crushed stone with silty sand 0.46		AU	2				
FILL: Brown silty sand with crushed stone, gravel, trace asphalt and topsoil		SS	3	67	50+		
		AU	4			1	61.85
1.42							
End of Borehole Practical refusal to augering at 1.42m depth (Piezometer blocked - June 2, 2021)							

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

○ **Water Content %**
20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 26, 2021

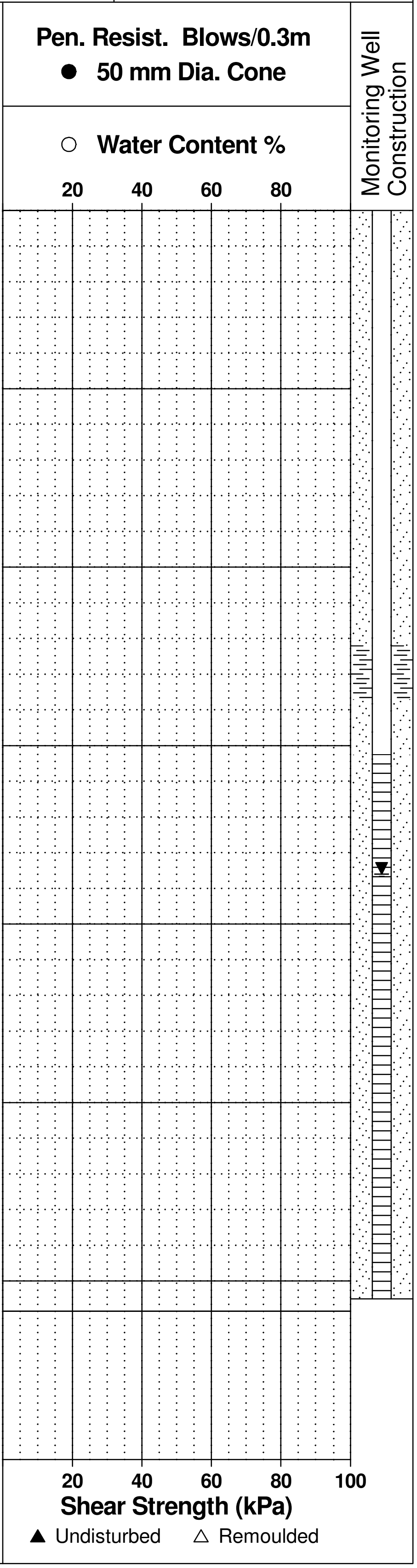
FILE NO. PG5829

HOLE NO. BH 8-21

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.04				
FILL: Crushed stone with silty sand	0.25	AU AU	1 2		
FILL: Brown silty sand with gravel, occasional cobbles, boulders, trace clay	1.04	SS	3	18	50+
BEDROCK: Fair to excellent quality, grey dolostone interbedded with limestone	6.17	RC RC RC RC	1 2 3 4	59 88 100 100	54 87 97 100
End of Borehole					

(GWL @ 3.72m - June 2, 2021)

DEPTH (m)	ELEV. (m)
0	62.75
1	61.75
2	60.75
3	59.75
4	58.75
5	57.75
6	56.75



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 26, 2021

FILE NO. PG5829

HOLE NO. BH 9-21

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
Asphaltic concrete 0.04						0	62.62		
FILL: Crushed stone with silty sand 0.30		AU	1						
FILL: Brown to grey silty sand, some wood		SS	2	33	22	1	61.62		
1.73		SS	3	50	50+				
End of Borehole Practical refusal to augering at 1.73m depth									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5829

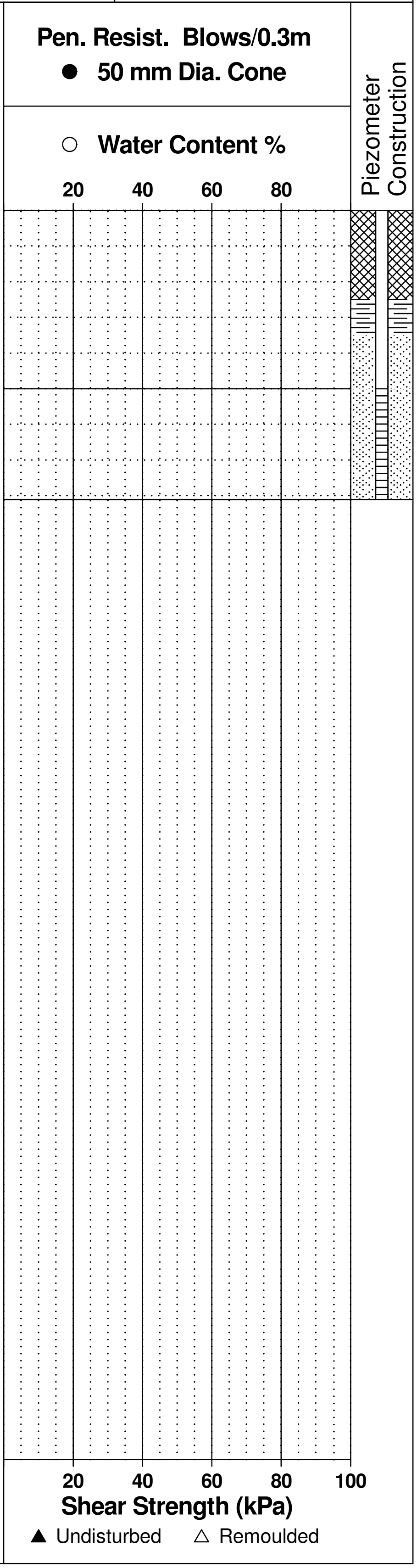
REMARKS

HOLE NO. BH10-21

BORINGS BY CME-55 Low Clearance Drill

DATE May 26, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.45
Asphaltic concrete 0.04							
FILL: Brown silty sand, some crushed stone 0.30		AU	1				
FILL: Brown silty sand, some gravel, occasional cobbles, trace wood		SS	2	67	33	1	61.45
1.62		SS	3	100	50+		
End of Borehole Practical refusal to augering at 1.62m depth (Piezometer blocked - June 2, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026, 2020 & 2006 Scott St. and 314 & 318 Athlone Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 26, 2021

FILE NO. PG5829

HOLE NO. BH11-21

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)	Piezometer Construction
GROUND SURFACE						0	62.82		
Asphaltic concrete 0.04									
FILL: Crushed stone with silty sand 0.25		AU	1						
FILL: Brown silty sand, some crushed stone and gravel 0.63		AU	2						
End of Borehole Practical refusal to augering at 0.63m depth (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

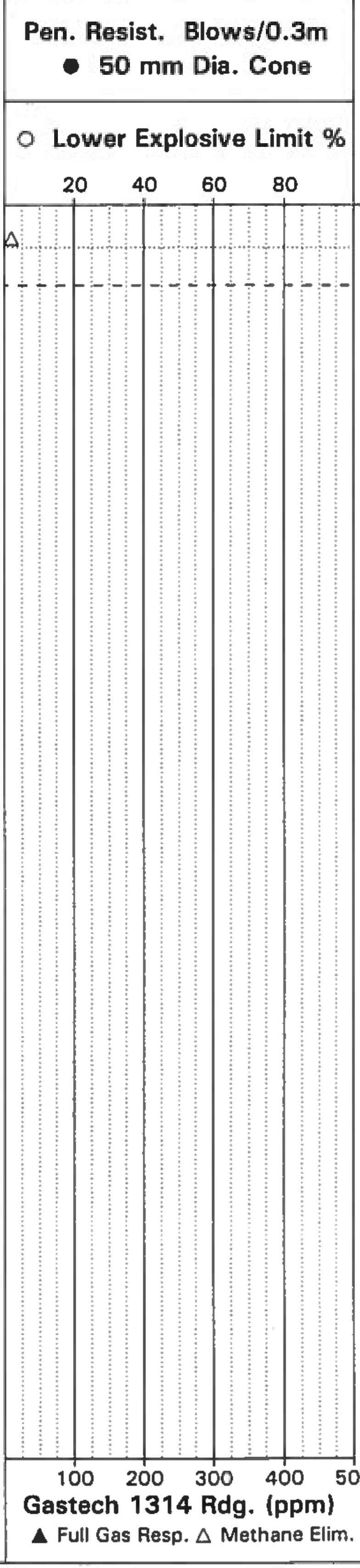
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 1

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE							0		
TOPSOIL	0.05								
FILL: Brown silty sand with gravel	0.38	▨	SS	1	53	4+			
End of Borehole Practical refusal to augering @ 0.38m depth									



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

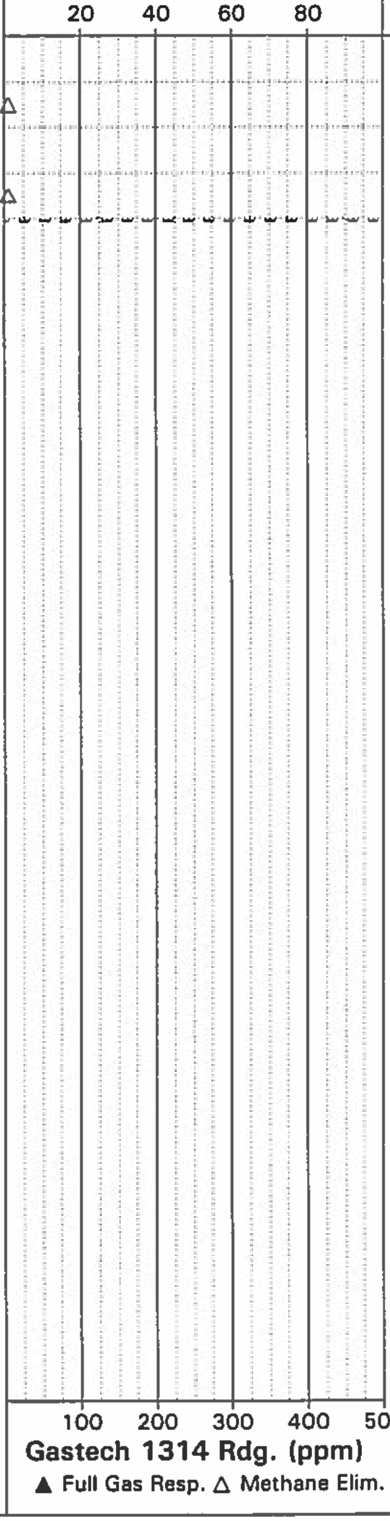
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 1A

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 / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0			
TOPSOIL 0.05									
FILL: Brown silty sand with gravel and clay	▨	SS	1	50	5				
0.81	▨	SS	2	38	17+				
End of Borehole Practical refusal to augering @ 0.81m depth									



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

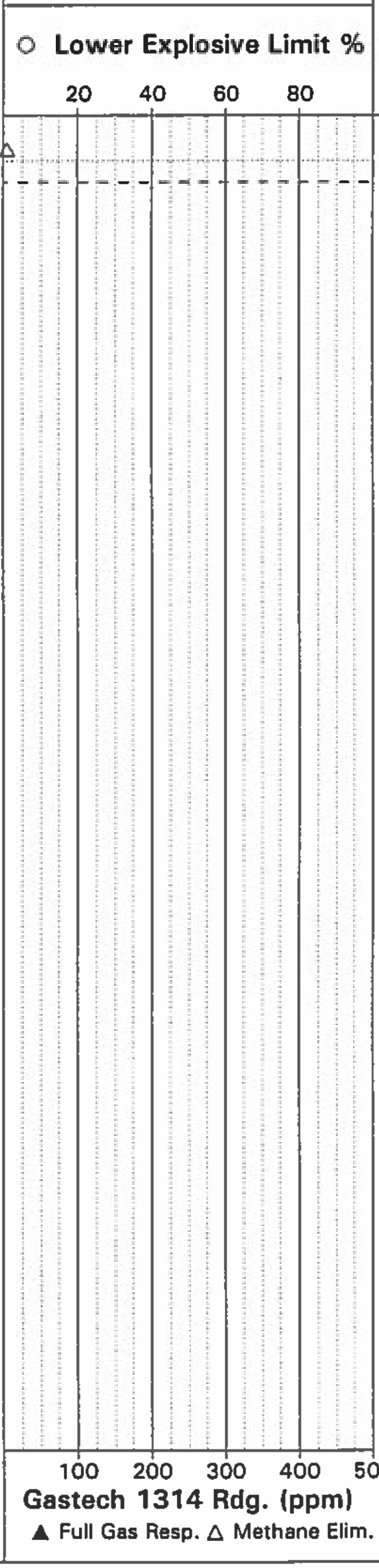
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 1B

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 / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0			
TOPSOIL 0.05									
FILL: Brown silty sand with black organic matter 0.30	▨	SS	1	58	17+				
End of Borehole Practical refusal to augering @ 0.30m depth									



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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
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DATUM

REMARKS

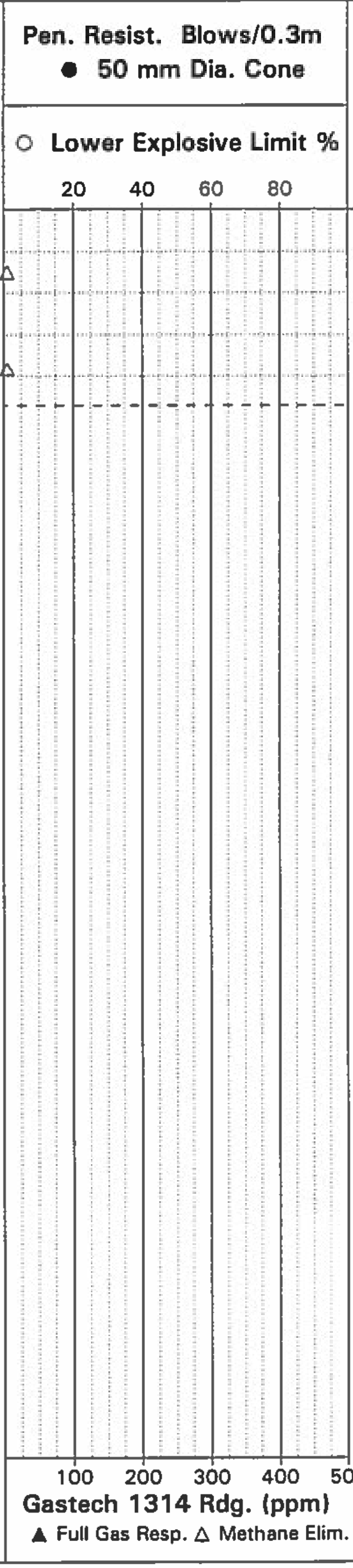
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

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	
25mm Topsoil	▨						
FILL: Dark brown silty sand with gravel	▨	SS	1	50	4		
	▨	SS	2	62	17+		
0.94							
End of Borehole Practical refusal to augering @ 0.94m depth							



Monitoring Well Construction

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SOIL PROFILE & TEST DATA

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314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

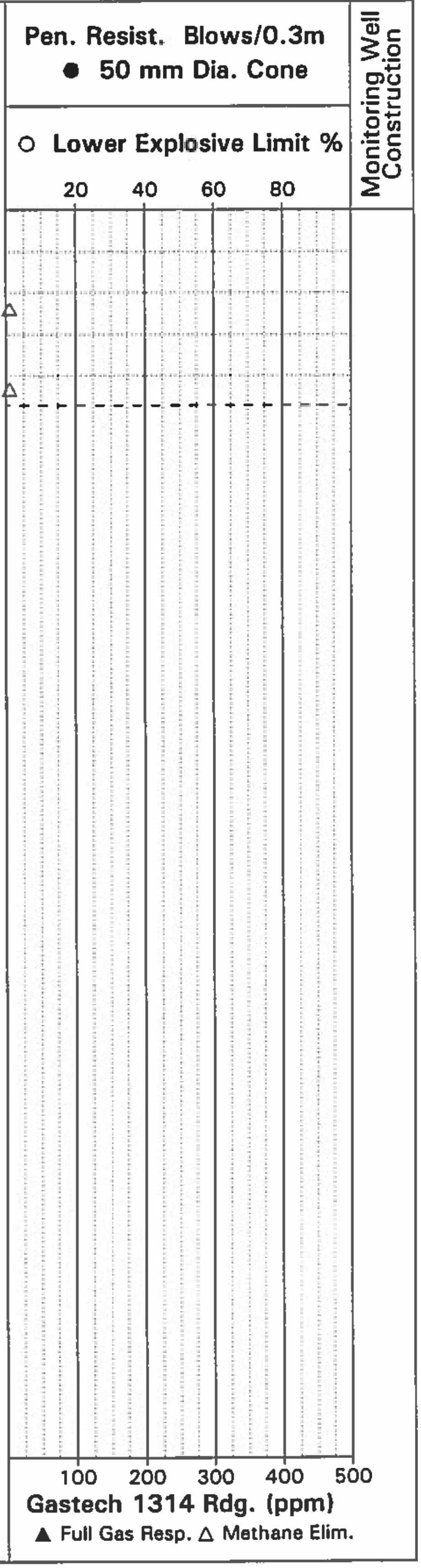
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 3

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	
Concrete	0.10							
FILL: Gravel	0.18							
FILL: Brown to dark brown sandy clay with gravel			SS	1	42	3		
	0.94		SS	2	67	17+		
End of Borehole Practical refusal to augering @ 0.94m depth								



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314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

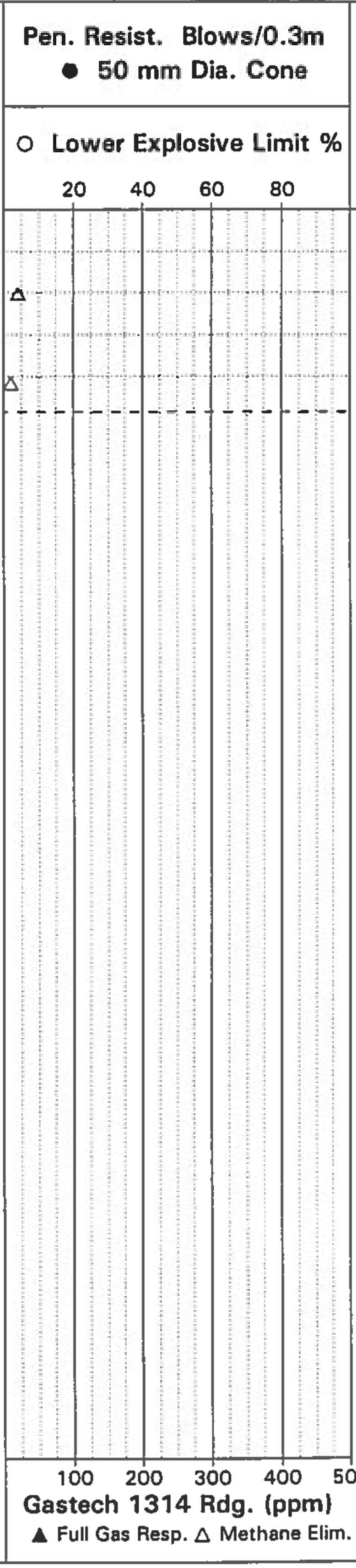
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete 0.10							
FILL: Dark brown silty sand with gravel, organic matter, tile, glass and plaster pieces	▨	SS	1	75	6		
0.97	▨	SS	2	20	17+		
End of Borehole Practical refusal to augering @ 0.97m depth							



Monitoring Well Construction

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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

FILE NO. PE0497

HOLE NO. BH 5

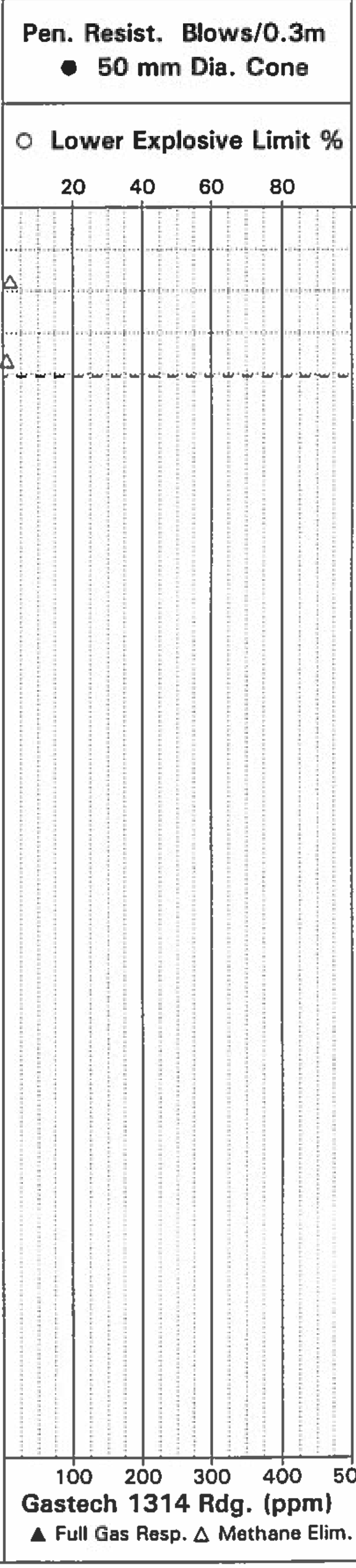
DATUM

REMARKS

BORINGS BY Portable Drill

DATE 29 APR 05

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete 0.05							
FILL: Brown silty sand with gravel	▨	SS	1	62	3		
0.81	▨	SS	2	50	17+		
End of Borehole Practical refusal to augering @ 0.81m depth							



Monitoring Well Construction

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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY Portable Drill

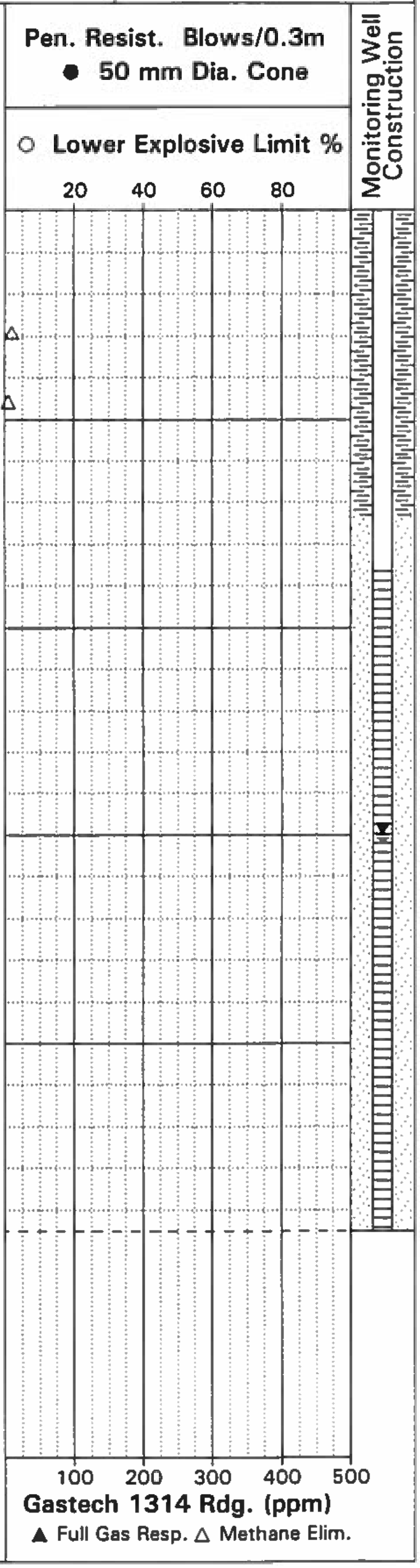
DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 6

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete	0.10						
FILL: Gravel	0.28						
FILL: Brown sandy silty clay with gravel		SS	1	54	7		
	0.94	SS	2	100	17+	1	
BEDROCK		RC	1	43	0		
		RC	2	87	82	2	
		RC	3	96	82	3	
		RC	4	91	91	4	
End of Borehole	4.90						

(GWL @ 3.00m-May 10/05)



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Ottawa, Ontario

DATUM

REMARKS

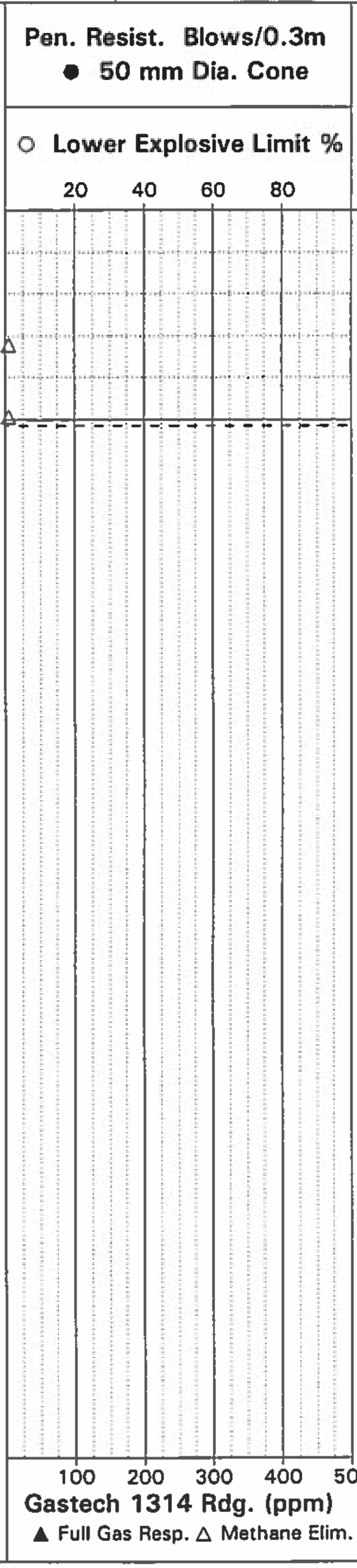
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0		
Concrete 0.10								
FILL: Gravel 0.34								
FILL: Dark brown sandy clay with gravel and topsoil		SS	1	58	18			
1.03		SS	2	100	17+	1		
End of Borehole Practical refusal to augering @ 1.03m depth								



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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
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Ottawa, Ontario

DATUM

REMARKS

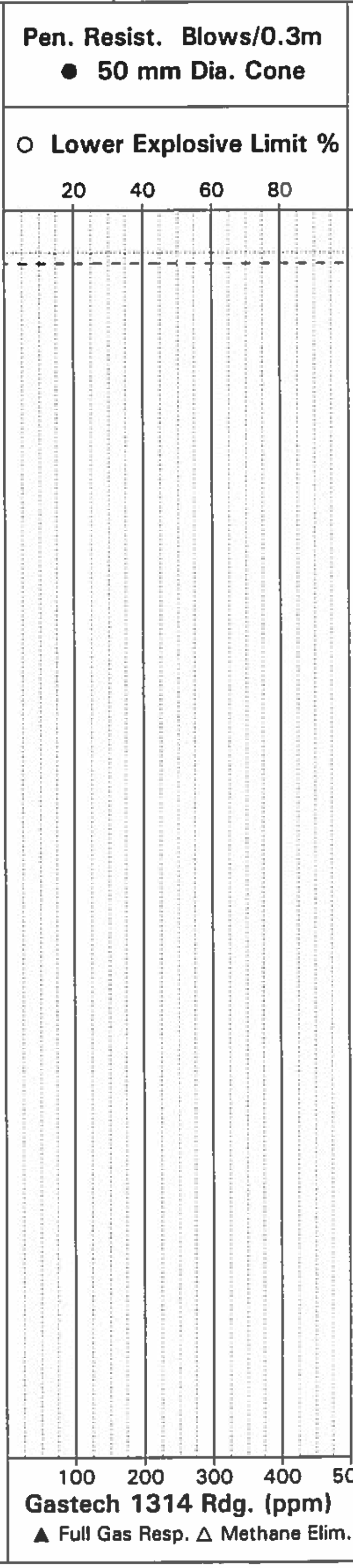
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 8

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Monitoring Well Construction
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE							0		
Concrete	0.20								
BEDROCK	0.25								
End of Borehole									



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SOIL PROFILE & TEST DATA

Phase I-II Environmental Site Assessment
314 Athlone Avenue
Ottawa, Ontario

DATUM

REMARKS

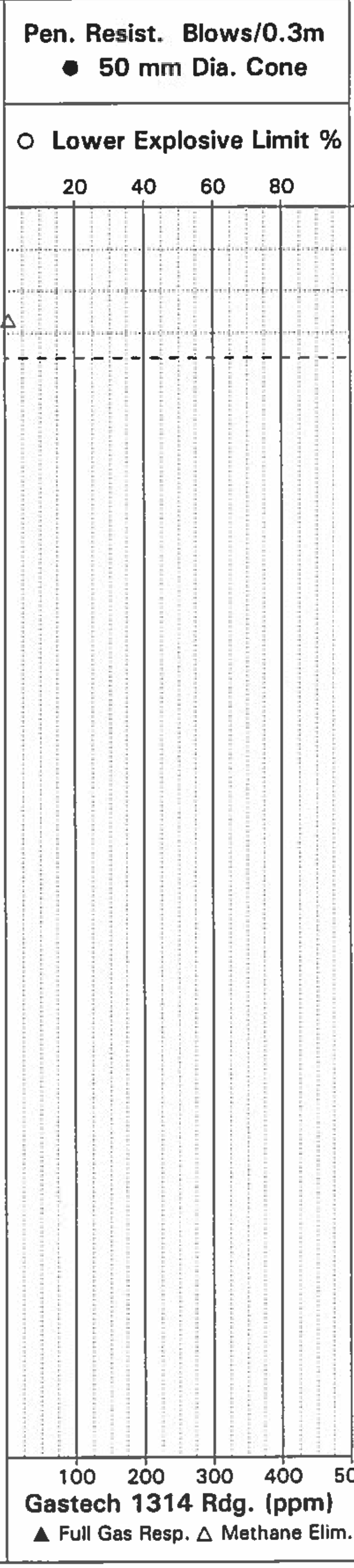
BORINGS BY Portable Drill

DATE 29 APR 05

FILE NO. PE0497

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0		
Concrete 0.10								
FILL: Gravel 0.36								
FILL: Brown silty sand with clay and gravel 0.63		SS	1	71	17+			
BEDROCK 0.72								
End of Borehole								



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

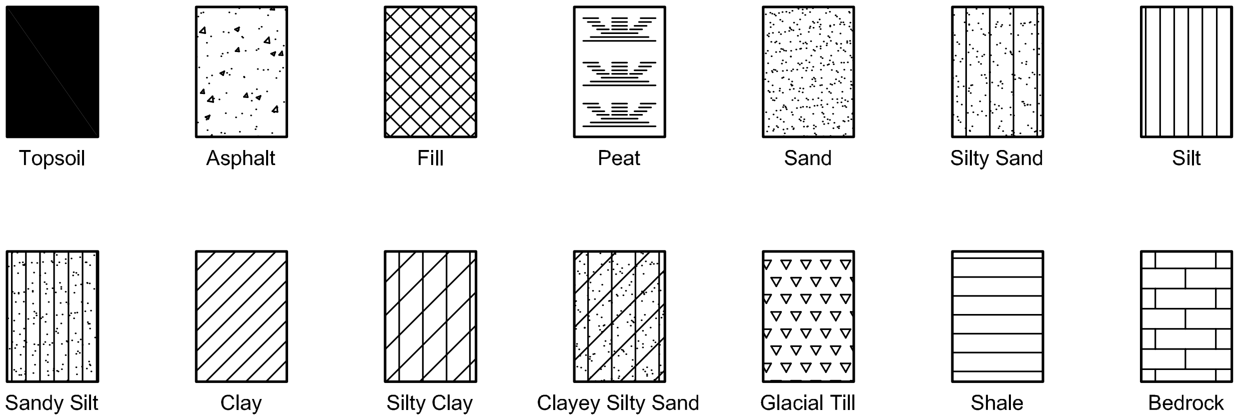
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

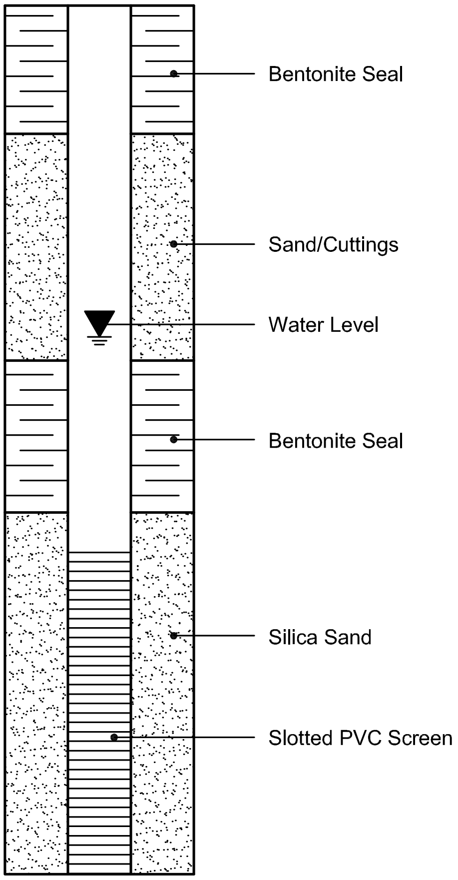
SYMBOLS AND TERMS (continued)

STRATA PLOT

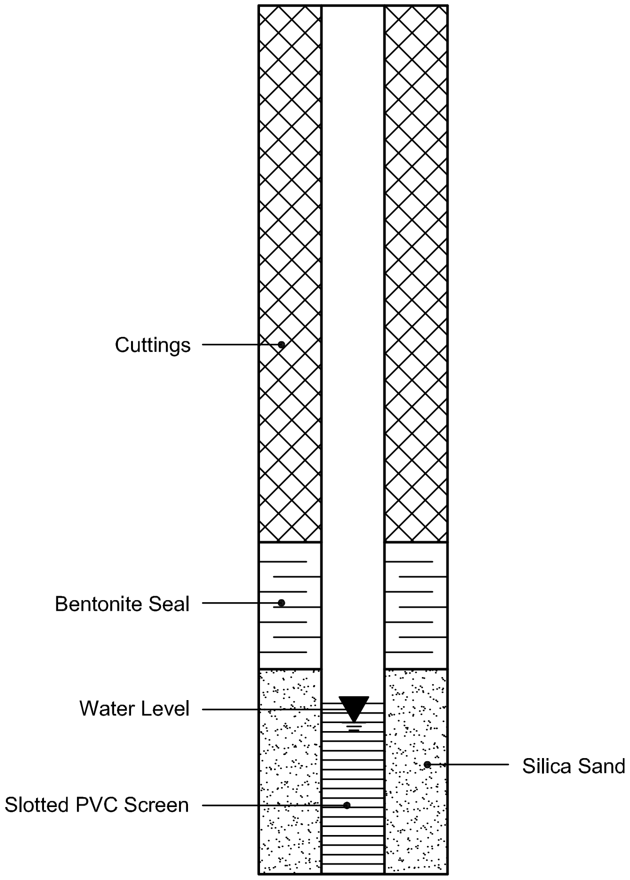


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29758

Report Date: 01-Jun-2021

Order Date: 27-May-2021

Project Description: PG5829

	Client ID:	BH1-21 SS3	-	-	-
	Sample Date:	25-May-21 09:00	-	-	-
	Sample ID:	2122375-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	87.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.68	-	-	-
Resistivity	0.10 Ohm.m	8.43	-	-	-
Anions					
Chloride	5 ug/g dry	321	-	-	-
Sulphate	5 ug/g dry	336	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5829-1 – TEST HOLE LOCATION PLAN

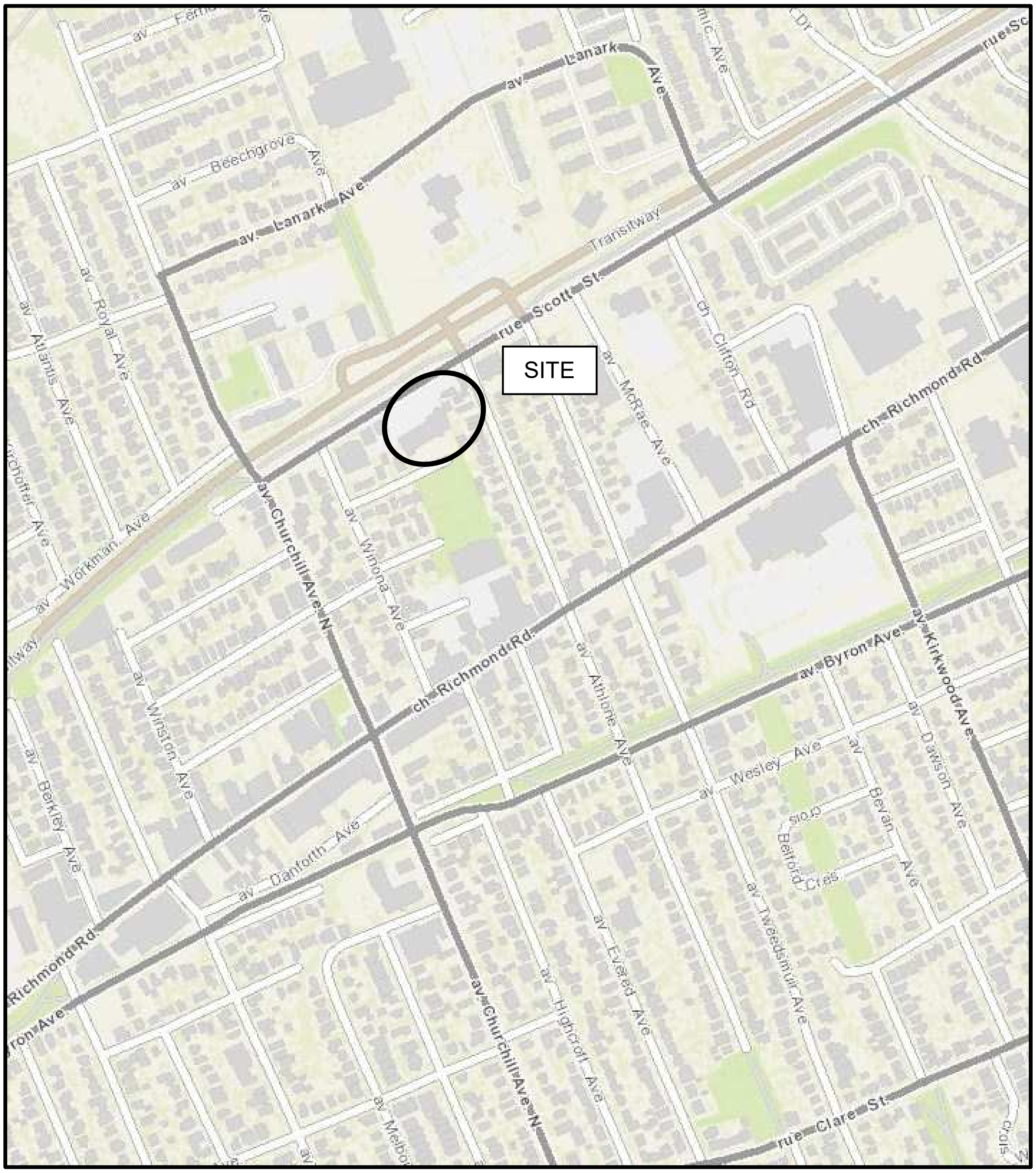
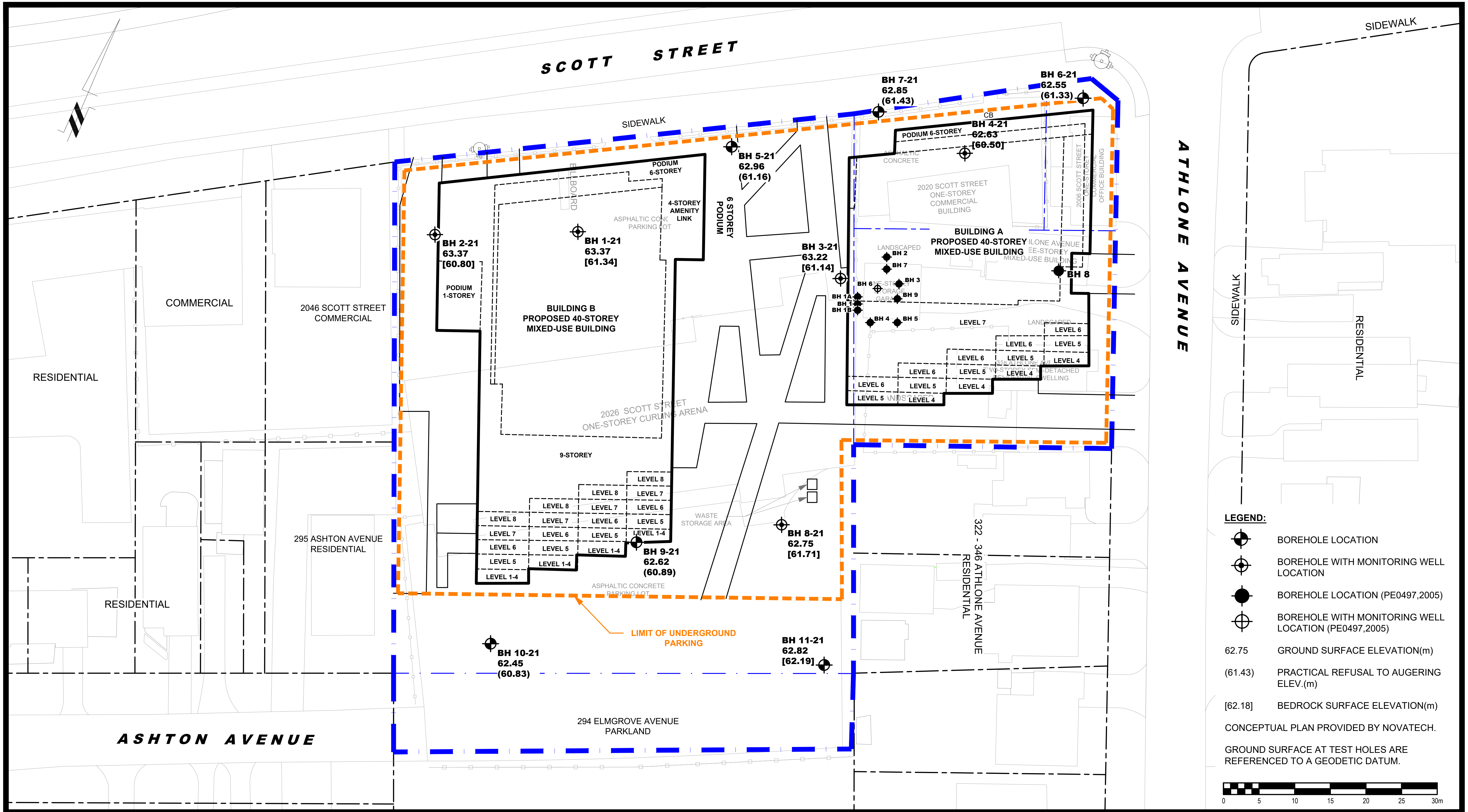


FIGURE 1

KEY PLAN

PATERSON
GROUP



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN	02/08/2023	PB

MORLEY HOPPNER GROUP

GEOTECHNICAL INVESTIGATION - PROPOSED HIGH RISE BUILDINGS
2006, 2020 & 2026 SCOTT STREET AND 314 & 318 ATHLONE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500

Date: 05/2021

Drawn by: RCG

Report No.: PG5829-1

Checked by: OC

Dwg. No.: PG5829-1

Approved by: SD

Revision No.: 1

p:\autocad drawings\geotechnical\pg58xx\pg5829\pg5829-1 test hole location plan (rev.01).dwg