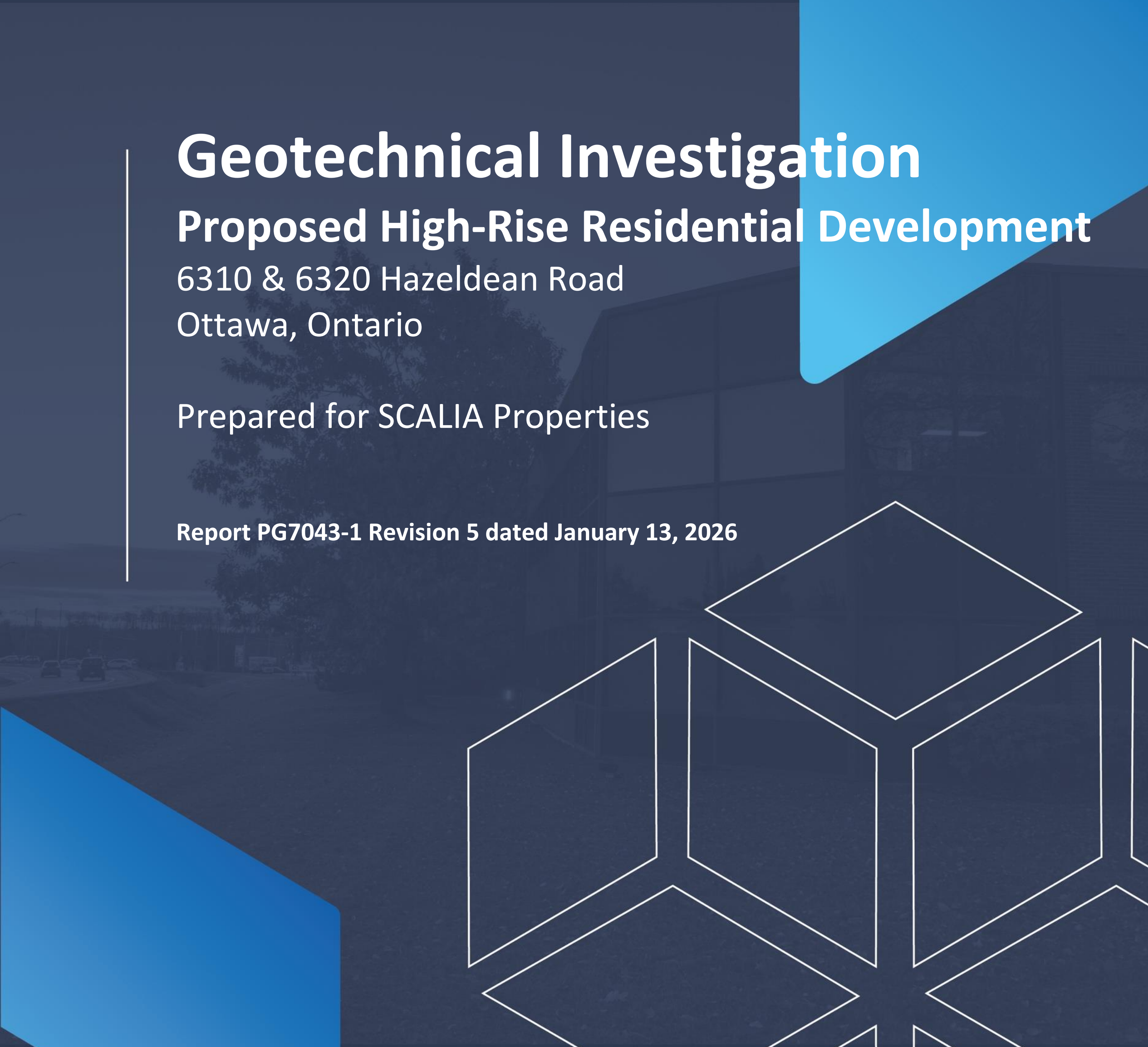
Geotechnical Investigation

Proposed High-Rise Residential Development

6310 & 6320 Hazeldean Road
Ottawa, Ontario

Prepared for SCALIA Properties

Report PG7043-1 Revision 5 dated January 13, 2026



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1.0 Introduction

Paterson Group (Paterson) was commissioned by SCALIA Properties to conduct a geotechnical investigation for the proposed development to be located at 6310 & 6320 Hazeldean Road in the City of Ottawa (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 a test hole program.
- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. 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 site 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 current conceptual drawings, it is our understanding that a 12-storey high-rise building with an associated 2-storey podium/parking level, and a 21-storey high-rise building along the eastern half of the subject site. The above-ground structures will overlie up to two basement levels of underground parking. The first basement level will a occupy the majority of the subject site and the second level will be located throughout the eastern half of the proposed development.

It is further expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was undertaken between March 18 to March 21, 2024. During that time a total of 10 boreholes and 20 probeholes were advanced to a maximum depth of 11.7 m below existing ground surface. Additionally, one (1) test pit was advanced on July 21, 2025, to a maximum depth of 4.3m below the ground surface.

Previous investigations were undertaken in November 2018 and December 2020. During that time, a total of 7 boreholes (BH 1-18 to BH 7-18) and 11 test pits (TP 1-20 to TP 11-20) were advanced to a maximum depth of 6.1 and 2.3 m below the existing ground surface, respectively.

The boreholes were drilled using a truck-mounted auger drill rig and portable drilling equipment 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 from our geotechnical department. The drilling procedure consisted of advancing to the required depths at select locations, sampling and testing the overburden.

Test pits were advanced using an excavator and backfilled with the excavated soil upon completion. The test pit procedure consisted of excavating to the required depth at the selected locations and sampling the overburden. The fieldwork was conducted under the full-time supervision of Paterson personnel.

The test holes undertaken by Paterson were located in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The approximate locations of the test holes are shown in Drawing PG7043-1 - Test Hole Location Plan included in Appendix 2.

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. Grab samples (G) were collected from the test pits at selected intervals. 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, split-spoon and grab 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 test holes are shown as AU, SS, G 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.

Diamond drilling was carried out at several borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1.

The thickness of the overburden was also evaluated by the use of probeholes at several test hole locations. This technique consisted of advancing augers until refusal to augering was reached by the drill rig. Select soil samples were recovered from auger flights or select split-spoon intervals as the augers were advanced to refusal.

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

During the current investigation BH 5-24 and BH 6-24 were fitted with a 32 mm diameter PVC groundwater monitoring well. During previous investigations a 51 mm diameter PVC groundwater monitoring well was installed in all boreholes to permit monitoring of the groundwater levels but are no longer present, with the exception of BH 6.

Selected previously advanced boreholes had been fitted with a groundwater monitoring well or flexible PVC standpipes to measure the groundwater levels. The groundwater observations from the current and previous investigations are discussed in Section 4.3 and are presented on the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations carried out by Paterson were determined by Paterson personnel taking into consideration site features and underground utilities. The location and ground surface elevation at each test hole location was surveyed by Paterson personnel and are referenced to a geodetic datum.

The test hole locations and ground surface elevation at each test hole location are presented on Drawing PG7043-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. One sample of recovered bedrock core was submitted for uniaxial compressive strength testing.

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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of a vacant lot layered with crushed stone across the surface and tall grass outlining the perimeter and various central portions.

Based on our review of historical aerial photographs, it is understood the subject site had been occupied as a storage yard for trucks prior to 2021 and a previously existing one-storey house on the eastern side of the subject site had been demolished in 2019.

The site is bordered by a commercial building to the north, a residential neighborhood to the east and south and Hazeldean Road to the west. The site was observed to be relatively flat and approximately at grade with adjacent roadways and neighboring properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations throughout the subject site consisted of a layer of fill underlain by silty sand and/or glacial till and further by the bedrock formation. A layer of asphalt was observed at the surface of BH 8-24, BH 9-24, PH 17-24 and BH 7 with thicknesses ranging from 40 to 130 mm.

The fill layer was observed to extend to depths between 0.1 to 2.1 m below existing ground surface. The fill layer generally consisted of silty sand with some gravel and crushed stone.

The silty sand layer was observed to extend to depths between 0.8 to 3.4 m below existing ground surface. The underlying glacial till layer was found to consist of brown and grey silty sand with clay, gravel, cobbles and boulders.

A poor to excellent quality limestone bedrock was encountered below the glacial till deposit at depths between 1.4 and 6.3 m throughout the subject site.

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

Bedrock

The bedrock was cored in BH 6-24 and BH 7-24 to a depth of 10.0 and 11.7 m below ground surface, respectively. The rock cored in BH 6-24 had a recorded average RQD value ranging from 73 to 100 %, while the recovery values were all 100 %. The rock cored in BH 7-24 had a recorded average RQD value ranging from 0 to 64 %, while recovery values ranged from 63 to 100 %.

Based on these results the quality of the bedrock for BH 6-24 and BH 7-24 ranges from good to excellent and very poor to fair quality, respectively. The bedrock was observed to consist of grey limestone.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of a limestone of the Bobcaygeon and Gull River Formations and expected to be encountered at depths varying between 3 and 10 m.

Paleozoic Faults

According to data presented in Ontario Geological Survey (OGS) File No. OGS-MRD219, Paleozoic Geology of Southern Ontario, a Paleozoic fault is shown to potentially traverse the subject site. It should be noted that the location of this feature is interpreted at a regional scale and is therefore approximate in nature. As stated by the OGS, the information contained within this publication is provided on an “as-is” basis and is intended for general reference purposes. Paterson intends on reviewing and confirming the bedrock conditions, particularly fault-related conditions, at the field level during construction and will provide additional input should subsurface conditions differ from those anticipated.

Unconfined Compressive Strength Testing of Bedrock Core Samples

One (1) select bedrock core obtained by Paterson as part of the current investigation was tested for unconfined compressive strength. The results of the test are summarized in Table 1 below and presented on Unconfined Compressive Strength Testing Results on Appendix 1.

Table 1 – Summary of Unconfined Bedrock Compressive Strength Testing Results

Borehole	Sample	Test Core Depth (m)	Test Core Elevation (m)	Unconfined Compressive Strength (MPa)
BH 6-24	RC2	6.68	116.44	73.6

Potential for Karstic Bedrock

It should be noted that available provincial geological mapping depicts the subject site within an area of inferred Karst. The following discussion presents our assessment of site-specific geological and subsurface information to evaluate the potential presence of karstic bedrock at the subject site.

Based on the results of the geotechnical investigation, it was observed that the bedrock at the subject site was generally encountered at depths varying between 4 to 5 m depth. Bedrock recovery values ranged from 63 to 100 % for the bedrock cores retrieved throughout the investigation, with the majority of sampled intervals yielding 100%, which suggests the bedrock encountered throughout the subject site generally intact and would not suggest Karst topography (dissolution of bedrock resulting in highly fractured bedrock, water-bearing fractures, etc.).

Based on our findings and experience with Karst topography throughout the area of the subject site, Karst topography and associated features are not expected to be present at the subject site given the overall intactness and quality of the bedrock encountered within the test holes despite the inferred presence of Karst by available provincial geological mapping. Our findings throughout the various site-specific field investigations are consistent with the subsurface conditions typically encountered over the above-noted formations (i.e. a relatively thick glacial till layer overlying a weathered poor to fair quality upper bedrock surface, further underlain by an unweathered good to very good quality bedrock. Additionally, based on our visual inspection of the recovered bedrock core samples, there were no apparent signs of dissolution of the rock, a typical indication of the presence of karstic bedrock.

In our experience, Karst topography is more prone in areas with surficial bedrock outcrops and where surface water can readily migrate and infiltrate bedrock formations. However, given the depth to the bedrock throughout the subject site and presence of relatively impermeable overburden, the subject site is not considered to be prone to the development of Karst topography.

Paterson personnel will work closely with the earthworks contractor to review all exposed bedrock at the founding elevation and to undertake appropriate mitigation measures should Karst features be encountered at that time. Generally, measures could include removal of bedrock in areas of notable fracture-paths and reinstatement (where required) by the use of lean-concrete below foundation structures. Should water-bearing fractures be identified throughout the bedrock footprint, measures will be undertaken to mitigate long-term potential influx of groundwater that may be conducive to the development of Karst topography.

In summary, it is not anticipated that Karst topography will be encountered at the founding level of the proposed structure despite the current mapping suggesting the area of the subject site is an area of inferred Karst. Further, Paterson does not consider a site-specific karst investigation warranted at this time. However, Paterson will verify the conditions encountered throughout the bedrock excavation to confirm Karst is not present.

4.3 Groundwater

The measured groundwater level (GWL) readings are presented in Table 2 below and further presented in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that groundwater levels fluctuate seasonally and may vary at the time of construction.

Table 2 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-24	123.57	1.45	122.12	March 28, 2024
		2.92	120.65	July 21, 2025
BH 2-24	123.47	1.07	122.40	March 28, 2024
		1.61	121.86	July 21, 2025
BH 3-24	123.09	1.06	122.03	March 28, 2024
		1.51	121.58	July 21, 2025
BH 4-24	123.48	1.03	122.45	March 28, 2024
BH 5-24	123.31	0.95	122.36	March 28, 2024
BH 6-24	123.12	1.34	121.78	March 28, 2024
		1.90	121.22	July 21, 2025
BH 7-24	123.68	0.51	123.17	March 28, 2024
BH 8-24	123.38	0.58	122.80	March 28, 2024
BH 9-24	123.36	0.89	122.47	March 28, 2024
BH 10-24	123.09	0.91	122.18	March 28, 2024
		1.25	121.84	July 21, 2025
BH 1	123.44	1.03	122.38	December 11, 2018
BH 2	123.68	1.39	122.29	
BH 3	123.14	1.43	121.71	
BH 4	123.02	0.92	122.10	
BH 5	123.28	2.85	120.43	
BH 6	123.67	1.53	122.14	March 28, 2024
		1.17	122.50	December 11, 2018

Notes: The test holes were surveyed with respect to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed high-rise buildings be founded on conventional spread footing foundations placed directly upon a clean, surface sounded bedrock and/or compact to dense, in-situ, native glacial till bearing surface.

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 organics or construction debris, 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 perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below finished grade.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming and/or controlled blasting will be required to remove sound 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 in conjunction with conventional excavation techniques, such as the use of a hydraulic excavator.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock bolts in conjunction with chain link fencing covered with a layer of woven geotextile fastened to the excavation face is expected to be required across the perimeter of the bedrock excavation where the weathered bedrock layer (i.e. anticipated to be the upper 1 to 1.5 m of the bedrock surface (and higher in areas of deeper segments of fractured bedrock) would be located overhead of workers within the base of the excavation footprint.

Further, shotcrete in conjunction with rock bolts may be required to in-fill localized bedrock pop-outs and over-excavations in the sidewalls resulting from the bedrock removal program to provide suitable surfaces to blind-pour foundation wall structures upon. Provisions should be carried for providing a smooth bedrock surface if blind-sided pours are considered against the bedrock surface.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the bedrock removal program. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of composite foundation drainage board and/or other foundation waterproofing assemblies. All loose bedrock fragments should be removed by a combination of grinding and pressurized air. It is recommended that Paterson review the bedrock excavation program at the time of construction.

Overbreak in Bedrock

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

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

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete. As a preliminary recommendation and precaution, provisions should be carried anticipating up to 300 mm of overbreak below footings, however, this is not a prescriptive and well-established depth. It is expected this depth will vary across the subject site and may not be representative of the amount of overbreak that could occur.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. All efforts should be taken by the excavation contractor to minimize overbreak, backbreak and to minimize overbreak within detailed bedrock excavations. Consideration could be given to using a combination of smaller-sized excavation equipment and pressurized air for detailed footing excavations, however, would be best assessed and evaluated by the earthwork's contractor at the pre-construction stage. The contractor should consider the lower quality bedrock samples that were recovered from the test holes when assessing intactness of in-situ bedrock against minimizing overbreak.

This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that bedrock bearing surfaces be reviewed and approved by Paterson once the bedrock surface has attained the design founding elevation and should not be lowered to a deeper depth until reviewed and approved by Paterson at the time of construction.

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

Vibration Considerations

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

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards.

Fill Placement

Fill placed for grading beneath the building area 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 fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 50 mm and matching an OPSS Granular B Type II gradation. Where the fill is too open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions.

Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized. Providing a heat source during winter construction may be recommended should compacted fill material is intended to be exposed for long periods of time.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface or lean-concrete mud slab surface placed upon a previously approved clean, surface sounded limestone bedrock surface could be designed for a factored bearing resistance value at ultimate limit states (ULS) of **4,000 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.

Conventional spread footings placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **750 kPa**.

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

Lateral Support

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

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings placed upon an undisturbed, in-situ, compact to dense glacial till bearing surface using the above-noted values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on an acceptable surface sounded bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lean Concrete Filled Trenches

Alternatively, where bedrock is not encountered at the design underside of footing elevation for footings where a bedrock bearing resistance value and bearing medium is sought as part of the foundation design, consideration may be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**17 MPa** 28-day compressive strength).

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying sound bedrock. The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

5.4 Design for Earthquakes

Seismic Shear Wave Velocity Testing

Shear wave velocity testing was completed to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. The results of the shear wave velocity testing are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array location is presented on Drawing PG7043-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also 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 located 20, 3, and 2 m away from the first geophone, 15, 3, and 2 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods.

The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the proposed building foundations.

The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

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

Determination of Vs_{30}

Based on our testing results, the average overburden shear wave velocity is **300 m/s**, while the bedrock shear wave velocity is **2,731 m/s**. Further, the testing results indicate the average overburden thickness to be approximately 6 m.

It is understood all footings will be founded on or within 2.7 m of the bedrock surface, and as would be confirmed and verified during the detailed design phase by the structural design consultant and Paterson. Based on the above, the Vs_{30} was calculated considering the standard equation for average shear wave velocity provided in the OBC 2024 and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{2.7\ m}{300\ m/s} + \frac{27.3\ m}{2{,}731\ m/s}\right)}$$

$$V_{s30=}\ 1{,}579\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed building with foundations up to 2.7 m from bedrock surface is **1,579 m/s**. Therefore, a **Site Class A** is applicable for the proposed building, as per Table 4.1.8.4.A of the OBC 2024. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the in-situ soil and/or bedrock surfaces will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone crushed stone.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes should be advised by Paterson during the design phase and once the footing and sump pit locations are known. The footprint would be confirmed at the time of construction once groundwater infiltration can be best assessed, if any. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions could be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. A portion of the basement walls are expected 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 dry unit weight of 23.5 kN/m^3 (effective unit weight of 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

The seismic earth pressure is expected to be transferred to the underground floor slabs, which should be designed to accommodate the pressures. Undrained conditions are anticipated (i.e., below the groundwater level). Therefore, the applicable effective unit weight of the retained soil should be 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight for the overburden.

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

Static Earth Pressures

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

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per the OBC 2024.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes.

The anchor can fail either by shear failure along the grout/rock interface or by pullout from a 60 to 90-degree cone with the apex near the middle of the anchor bonded length. Interaction may develop between the failure cones of adjacent anchors resulting in a total group capacity less than the sum of the individual anchor load capacity.

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

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. 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 does not flow from one hole to an adjacent empty one.

Regardless of whether an anchor is of the passive or post tensioned type, the anchor is recommended to be provided with a fixed length at the anchor base, which will provide the anchor capacity, and a free length between the rock surface and 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 has a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is at the bottom portion of the 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. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed buildings, 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 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.

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. For design purposes, all rock anchors were assumed to be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

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

Table 3 – Parameters Used in Rock Anchor Review

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

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter hole are provided in Table 4.

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and flushed clean with water prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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

Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.6	1.0	2.6	500
	2.0	1.0	3.0	750
	2.8	1.0	3.8	1,000
	4.0	1.0	5.0	1,500
125	1.4	1.0	2.4	500
	2.0	1.0	3.0	750
	2.9	1.0	3.9	1,000
	4.2	1.0	5.0	1,500

5.8 Pavement Design

The recommended pavement structures for the subject site are shown in Table 5, Table 6 and Table 7.

Table 5 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 6 – Recommended Pavement Structure – Local Residential Roadways, 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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 7 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

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.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm for the above-noted pavement structures. The bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised during the bedrock removal program to ensure that the bedrock subgrade does not have depressions that will trap the water.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Foundation Drainage System

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure where double-sided pours will be undertaken. In areas where blind-sided pours will be considered, the perimeter drainage pipe should be placed along the interior side of the foundation wall and connected to sleeves placed within the foundation wall at a 6 m center-to-center spacing. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

It is anticipated that underfloor drainage will be required to control water infiltration below the proposed basement level. The layout of the perimeter and underfloor drainage systems should be determined by Paterson during the design phase once the foundation structure and sump pit locations are known. The perimeter drainage pipe would connect to a series of underfloor drainage lines which would direct water to sump pit(s) within the lower basement area.

The top endlap for all portions of the foundation drainage board that terminate against the top surface of a podium level should be sealed at the top with a waterproofing membrane to mitigate the potential for surface water to migrate behind the foundation drainage board layer.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts and pools located within the lowest basement level. A continuous PVC waterstop should be installed within the interface between the concrete base slab below the elevator shaft foundation walls. It is recommended that Paterson review and advise on all basement waterproofing/drainage system designs during the design phase.

Foundation Backfill

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 composite drainage system, such as CCW MiraDRAIN 2000 or Delta-Teraxx or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed waterproofing membrane to mitigate the potential for water to migrate between the drainage board and foundation wall. Further, the bottom end lap of the podium deck waterproofing should extend a minimum of 300 mm below the cold-joint between the underside of the podium deck suspended slab and top of the foundation wall. The design of the tie-in should be reviewed and advised upon by Paterson at the appropriate design stage.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized or current structural and architectural drawings for the proposed building to provide specific waterproofing and drainage design recommendations for design and tender. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

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

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 heated structures. Such exterior structures require additional frost protection, such as 2.1 m of soil cover, or a reduced thickness of soil cover if rigid insulation is used.

6.3 Excavation Side Slopes

Unsupported Excavations

Excavation side slopes above the groundwater level extending to a maximum vertical height of 3 m should be cut back at 1H:1V or flatter for the compact to dense glacial till soils and 1.5H:1V for the fill and silty sand/sand soils. Flatter slopes are required for unsupported excavations undertaken below the groundwater table. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors or other means determined by Paterson at the time of construction and as described in Section 5.2 of this report.

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 Paterson in order to detect if the slopes are exhibiting signs of distress.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 2 m centres.

Soil stockpiles, debris, and other forms of weight should not be considered for the purpose of securing the tarpaulins along the top of the slope. However, consideration may be given to restraining the tarpaulins with soil, sandbags, stone, etc. along the bottom of the side-slope. The tarpaulins should extend beyond the overburden and onto the bedrock surface.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A minimum of 1 m horizontal ledge should remain between the unsupported excavation and bedrock surface.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced.

Given the sandy nature of the soils present throughout the subject site, the designer should consider provisions to mitigate the potential for excessive losses of retained soil during the lagging installation process if consideration is given to using a soldier pile and lagging system. It is expected these efforts will be undertaken in the dry and to mitigate risks associated with running sand or loose layers of sandy/non-cohesive soils.

It is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. Shoring designs should clearly indicate measures to ensure adequate contact between lagging and retained soils to minimize sloughing and disturbance of retained soils that could result in the formation of void that would form without adequate lagging-overburden contact. Further, lift heights and bay widths of the excavation supported by a timber lagging and soldier pile system should be specified to consider the non-cohesive and loose nature of the in-situ fill and sandy subsoils.

The shoring system is recommended to be adequately supported to resist toe failure. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 8.

Table 8 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

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 is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. Fill subgrade should be proof-rolled/re-compacted under the supervision of Paterson personnel. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes.

The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A crushed stone. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 99% of the SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations through the overburden materials should be low and controllable using open sumps. Higher infiltration rates are anticipated to be encountered as of approximately 1 m of the bedrock surface; however, infiltration is expected to be controlled using open sumps.

It is recommended that a specialized dewatering contractor review the information provided herein regarding groundwater levels and hydraulic conductivities for planning excavations throughout the subject site. This is recommended since the building’s excavations are anticipated to be undertaken below the groundwater table and could require larger capacity systems than open sumps for temporarily dewatering the subject site during the construction phase.

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

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three 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.

Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impact to Neighboring Properties

Based on the proposed grading and design details, the proposed structures will be founded within compact to dense glacial till and sound bedrock layers and not within subsoils sensitive to long-term dewatering. It is expected nearby neighboring structures are supported by similar subsoils such that there would be no negative impacts due to long-term dewatering that are anticipated to occur to neighboring structures and infrastructure as a result of the construction of the proposed structures from a geotechnical perspective.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where 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/or glycol lines 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 foundation is 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.

Under winter conditions, if snow and ice is present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is greater 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 non-aggressive to moderately corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

Since the structures are not anticipated to be founded upon silty clay soils affected by the depth of root penetration (i.e., and will be founded upon bedrock), City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines*) from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once future details of the proposed development have been prepared:

- Review grading, servicing and structural plan(s) from a geotechnical perspective.
- Review of architectural plans pertaining to the buildings foundation drainage and/or waterproofing system and associated drainage systems.

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

- Observation of all bearing surfaces prior to the placement of concrete.
- Observation of all waterproofing membranes, sub-slab drainage system and all associated systems and assemblies.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

The recommendations 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 SCALIA Properties 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.

N Versolato

Nicholas F. R. Versolato, CPI, B.Eng

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
January 13, 2026
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

6310 & 6320 Hazeldean Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348744.47 **NORTHING:** 5014117.98 **ELEVATION:** 123.33

PROJECT: Proposed High-Rise Development

ADVANCED BY: Excavator

REMARKS:

DATE: July 21, 2025

FILE NO. : PG7043

HOLE NO. : TP 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Crushed stone

0.40m [122.93m]

FILL: Brown silty sand, with crushed stone

0.80m [122.53m]

Compact, brown **SILTY SAND**

1.30m [122.03m]

GLACIAL TILL: Compact to dense, brown, silty sand to sandy silt, with gravel, cobbles and boulders

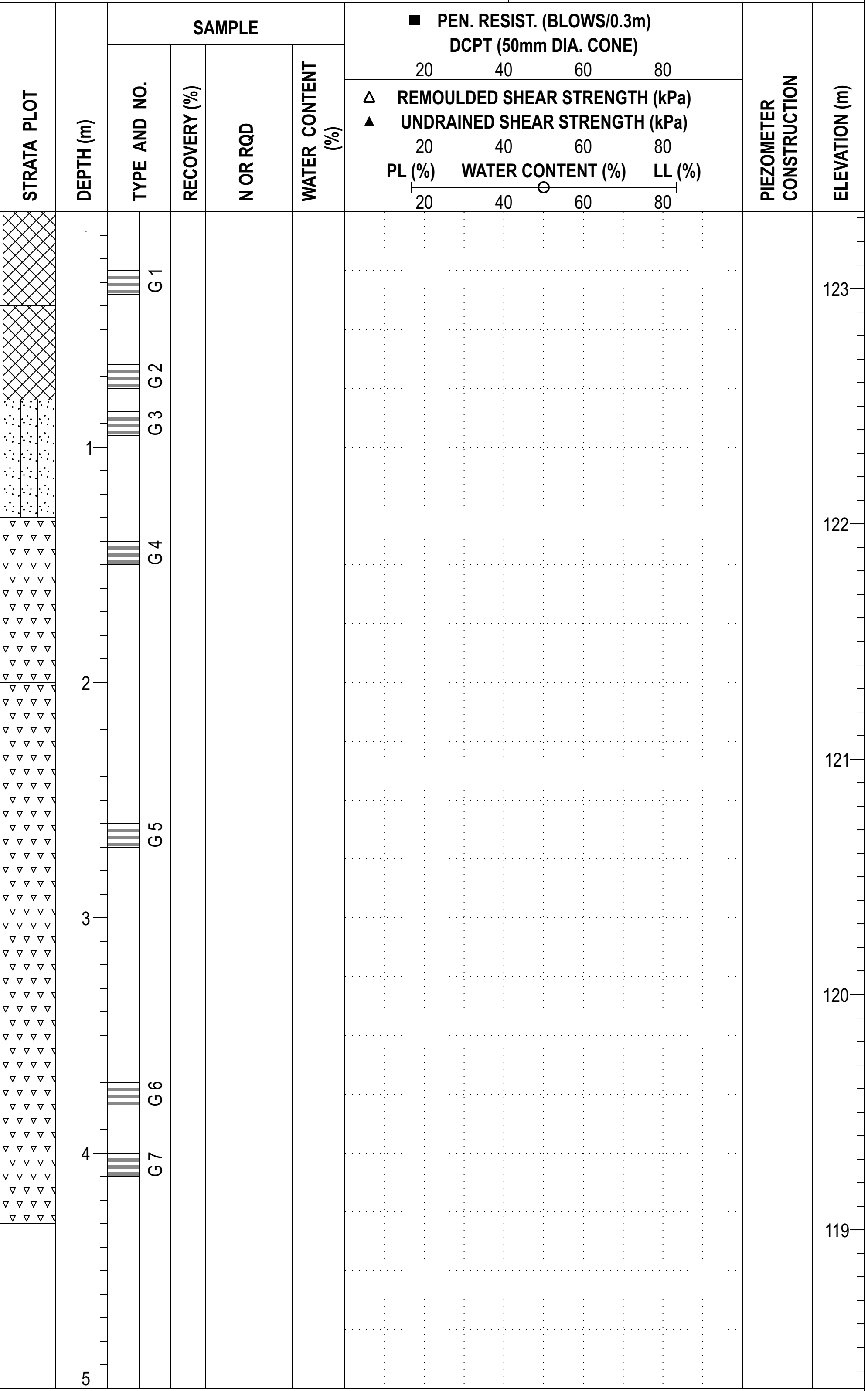
2.00m [121.33m]

GLACIAL TILL: Dense to very dense, grey silty sand to sandy silt, with gravel, cobbles and boulders

4.30m [119.03m]

End of Test Pit

Test pit dry upon completion of excavation



P:/AutoCAD Drawings/Test Hole Data Files/PG70xx/PG7043/data.sqlite 2025-07-28, 14:45 Paterson_Template DR

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348690.506 **NORTHING:** 5014085.407 **ELEVATION:** 123.57
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 18, 2024

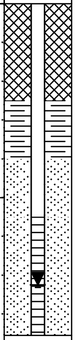
FILE NO. PG7043
HOLE NO. BH 1-24

SAMPLE DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	123.57
FILL: Brown silty sand with gravel and crushed stone - trace topsoil by 0.1m depth	0.46	SS	1	92	29		
Loose, brown **SILTY SAND**		SS	2	67	7	1	122.57
	1.45						
GLACIAL TILL: Compact, brown silty sand to sandy silt, some gravel, occasional cobbles and boulders		SS	3	95	26	2	121.57
		SS	4	90	+50		
	3.08					3	120.57
End of Borehole Practical refusal to augering at 3.08m depth (GWL @ 1.45m - March 28, 2024)		SS	5	100	+50		

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
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348713.863 **NORTHING:** 5014164.326 **ELEVATION:** 123.47

DATUM: Geodetic

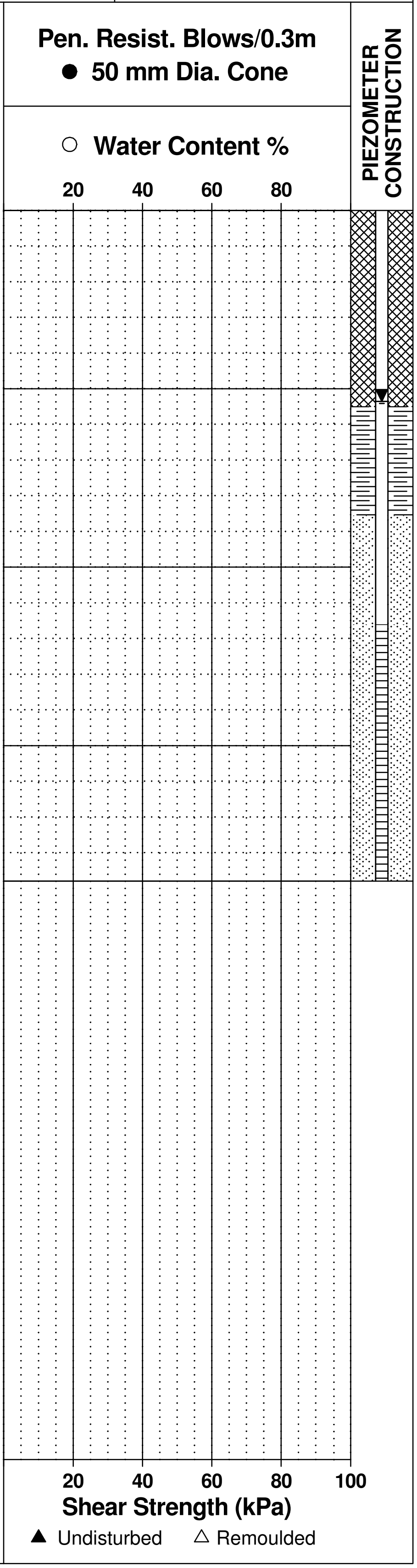
REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 18, 2024

FILE NO. PG7043

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	123.47
FILL: Brown silty sand with gravel and crushed stone		SS	1	13	37		
0.76							
TOPSOIL 0.94							
Compact to loose, brown **SILTY SAND**		SS	2	67	15	1	122.47
		SS	3	71	8		
- silt seam at 1.93m depth						2	121.47
2.74		SS	4	75	16		
GLACIAL TILL: Dense, brown silty sand, trace to some gravel						3	120.47
- grey by 3.4m depth 3.43		SS	5	75	38		
GLACIAL TILL: Dense, grey sandy silt, trace clay and gravel 3.76							
End of Borehole Practical refusal to augering at 3.76m depth (GWL @ 1.07m - March 28, 2024)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348765.812 **NORTHING:** 5014168.532 **ELEVATION:** 123.09

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 18, 2024

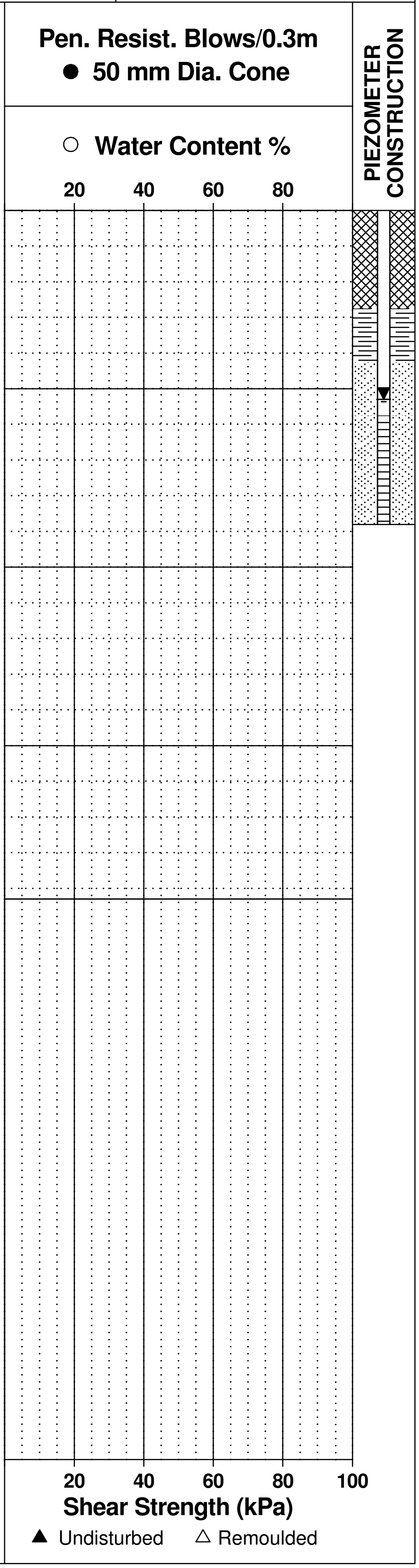
FILE NO. PG7043

HOLE NO. BH 3-24

SAMPLE DESCRIPTION	Depth (m)
Ground Surface	
FILL: Topsoil with organics and gravel	0.25
FILL: Brown silty sand with gravel and crushed stone	0.91
TOPSOIL	1.12
Loose to compact, brown **SILTY SAND**	3.20
GLACIAL TILL: Compact to dense, grey sandy silt, trace clay and gravel	3.86
End of Borehole Practical refusal to augering at 3.86m depth (GWL @ 1.06m - March 28, 2024)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	58	15
SS	2	75	7
SS	3	71	8
SS	4	67	4
SS	5	83	27
SS	6	100	+50

DEPTH (m)	ELEV. (m)
0	123.09
1	122.09
2	121.09
3	120.09



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

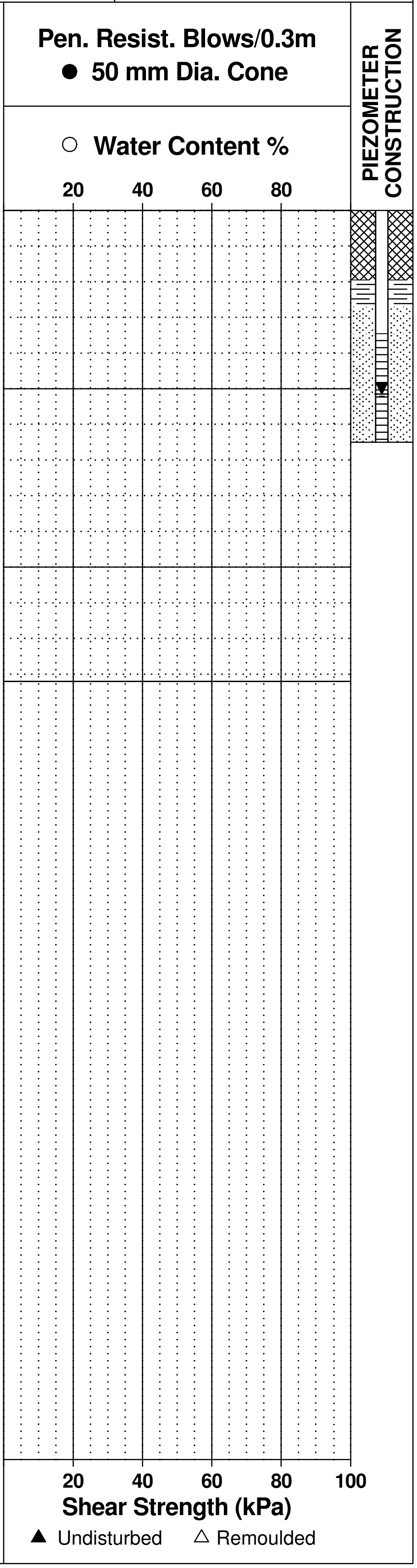
EASTING: 348723.688 **NORTHING:** 5014129.506 **ELEVATION:** 123.48
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 18, 2024

FILE NO. PG7043
HOLE NO. BH 4-24

SAMPLE DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	123.48
FILL: Topsoil and organics with crushed stone and gravel 0.30						
FILL: Brown silty sand with crushed stone and gravel 0.96	SS	1	58	11		
Loose to compact, brown **SILTY SAND**	SS	2	13	5	1	122.48
2.21	SS	3	83	16	2	121.48
GLACIAL TILL: Compact, grey silty sand to sandy silt with gravel, occasional cobbles and boulders 2.64	SS	4	94	+50		
End of Borehole						

Practical refusal to augering at 2.64m depth

(GWL @ 1.03m - March 28, 2024)



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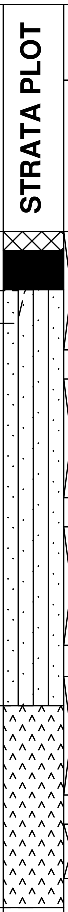
SOIL PROFILE AND TEST DATA

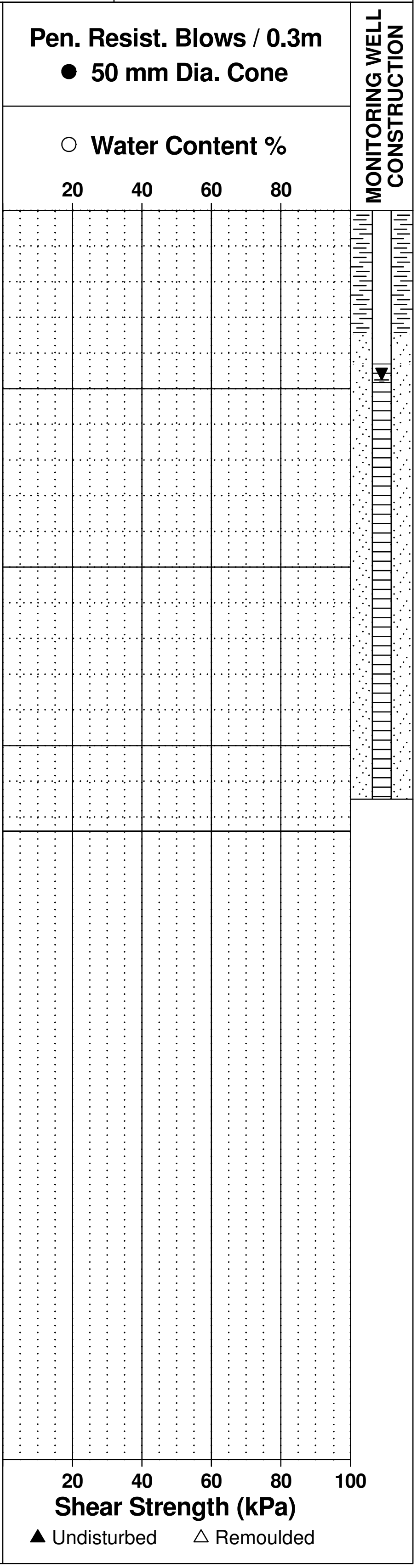
Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348725.399 **NORTHING:** 5014143.197 **ELEVATION:** 123.31
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 18, 2024

FILE NO. PG7043
HOLE NO. BH 5-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	123.31
FILL: Brown silty sand with crushed stone and gravel 0.10						
TOPSOIL with sand 0.30	SS	1	50	5		
Loose to compact, brown **SILTY SAND**						
	SS	2	33	4	1	122.31
- grey by 1.8m depth	SS	3	63	17	2	121.31
2.44						
GLACIAL TILL: Compact, grey silty sand to sandy silt, trace to some gravel	SS	4	100	23		
					3	120.31
	SS	5	100	+50		
3.48						
End of Borehole Practical refusal to augering at 3.48m depth (GWL @ 0.95m - March 28, 2024)						





SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348746.377 **NORTHING:** 5014187.551 **ELEVATION:** 123.12
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 19, 2024

FILE NO. PG7043
HOLE NO. BH 6-24

SAMPLE DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Brown silty sand, some crushed stone and gravel	0.46
Loose, brown **SILTY SAND** to **SANDY SILT**	1.70
GLACIAL TILL: Compact, brown silty sand to sandy silt, some gravel, trace clay, occasional cobbles and boulders	2.21
GLACIAL TILL: Dense, grey silty sand to sandy silt with gravel, occasional cobbles and boulders	3.96
BEDROCK: Good to excellent quality, grey limestone bedrock - vertical fractures from 7.7m to 7.9m depth	10.03
End of Borehole (GWL @ 1.44m - March 28, 2024)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	50	14
SS	2	100	7
SS	3	63	11
SS	4	83	35
SS	5	67	49
SS	6	100	+50
RC	1	100	73
RC	2	100	97
RC	3	100	85
RC	4	100	100

DEPTH (m)	ELEV. (m)
0	123.12
1	122.12
2	121.12
3	120.12
4	119.12
5	118.12
6	117.12
7	116.12
8	115.12
9	114.12
10	113.12

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

STRATA PLOT

MONITORING WELL CONSTRUCTION

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348699.065 **NORTHING:** 5014129.498 **ELEVATION:** 123.68

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 19, 2024

FILE NO. PG7043

HOLE NO. BH 7A-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown silty sand, some gravel and crushed stone	0.61				
FILL: Grey silty sand - trace gravel extending to 1.5m depth	1.83				
FILL: Brown silty sand to sandy silt with crushed stone	2.13				
Loose, grey **SILTY SAND**	3.40	SS	4	67	3
		SS	5	71	5
GLACIAL TILL: Very dense, grey silty sand to sandy silt, some gravel, occasional cobbles and boulders	4.95	SS	6	11	+50
		SS	7	70	+50
BEDROCK: Poor to fair quality, grey limestone bedrock		RC	1	90	38
		RC	2	100	56
- very poor quality from 7.1m to 10.4m depth		RC	3	63	7
		RC	4	100	0
	11.66	RC	5	100	64

End of Borehole

(GWL @ 0.51m - March 28, 2024)

DEPTH (m)	ELEV. (m)
0	123.68
1	122.68
2	121.68
3	120.68
4	119.68
5	118.68
6	117.68
7	116.68
8	115.68
9	114.68
10	113.68
11	112.68

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

PIEZOMETER CONSTRUCTION

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

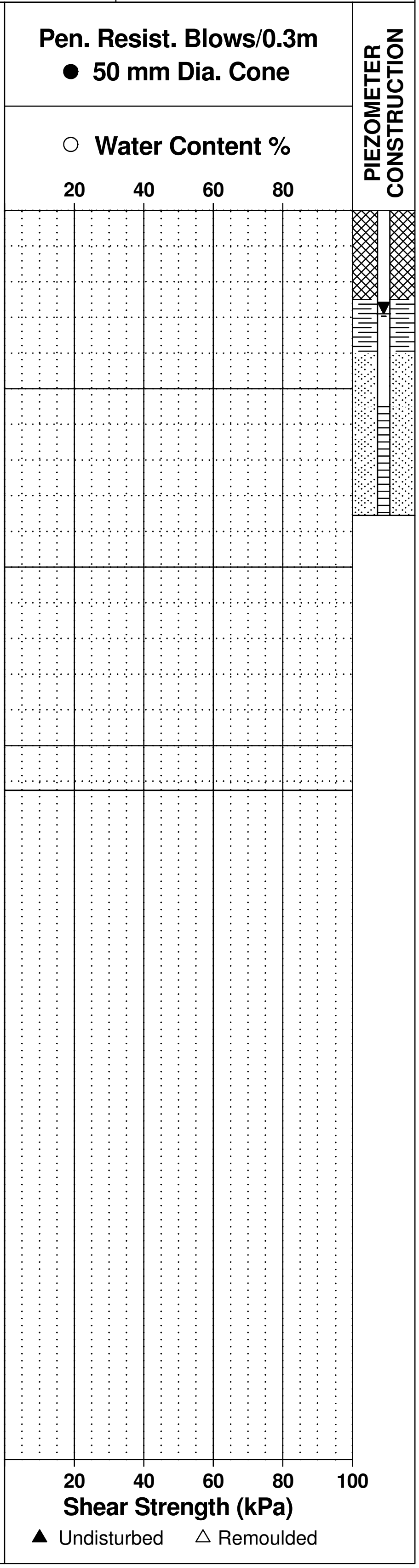
Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348719.151 **NORTHING:** 5014142.429 **ELEVATION:** 123.38
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. BH 8-24

SAMPLE DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
ASPHALTIC CONCRETE	0.03				
FILL: Brown silty sand, some gravel and crushed stone	0.69	SS	1	78	12
Loose to compact, brown **SILTY SAND**	1.83	SS	2	67	8
		SS	3	83	15
GLACIAL TILL: Very dense, brown silty sand to sandy silt, some gravel, trace clay, occasional cobbles and boulders		SS	4	100	+50
- no clay by 3.0m depth	3.25	SS	5	100	+50
End of Borehole					
(GWL @ 0.58m - March 28, 2024)					

DEPTH (m)	ELEV. (m)
0	123.38
1	122.38
2	121.38
3	120.38



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348719.658 **NORTHING:** 5014148.432 **ELEVATION:** 123.36
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

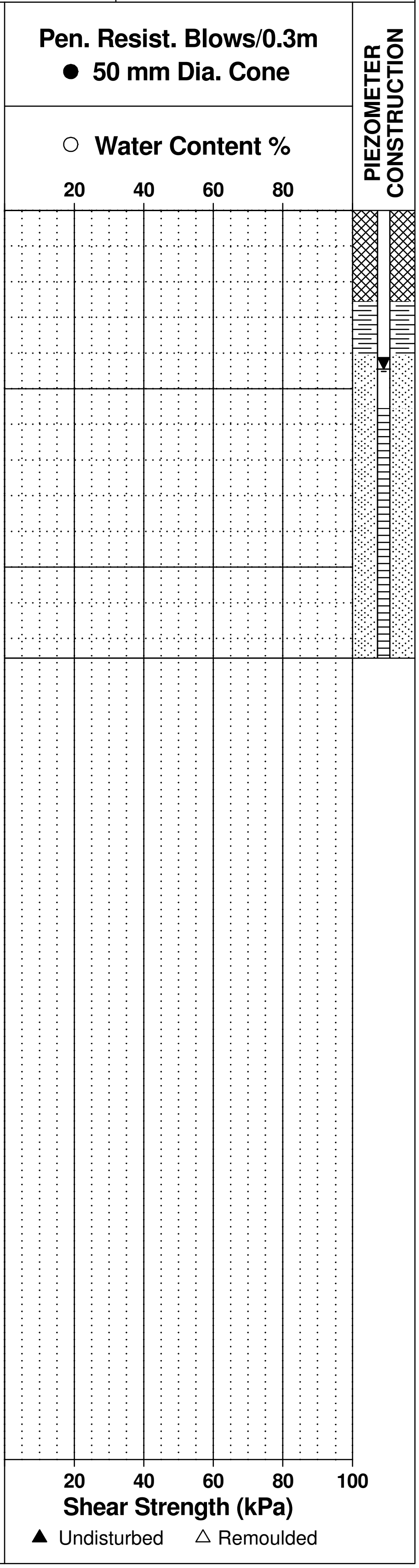
FILE NO. PG7043
HOLE NO. BH 9-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	123.36
ASPHALTIC CONCRETE	0.04						
FILL: Brown silty sand, some gravel and crushed stone	0.69	SS	1	89	11		
Loose to compact, brown **SILTY SAND** to **SANDY SILT**		SS	2	67	7	1	122.36
	2.21	SS	3	71	13	2	121.36
GLACIAL TILL: Very dense, brown silty sand, some gravel, trace clay, occasional cobbles and boulders	2.51	SS	4	100	+50		

End of Borehole

Practical refusal to augering at 2.51m depth

(GWL @ 0.89m - March 28, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348755.874 **NORTHING:** 5014178.497 **ELEVATION:** 123.09

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

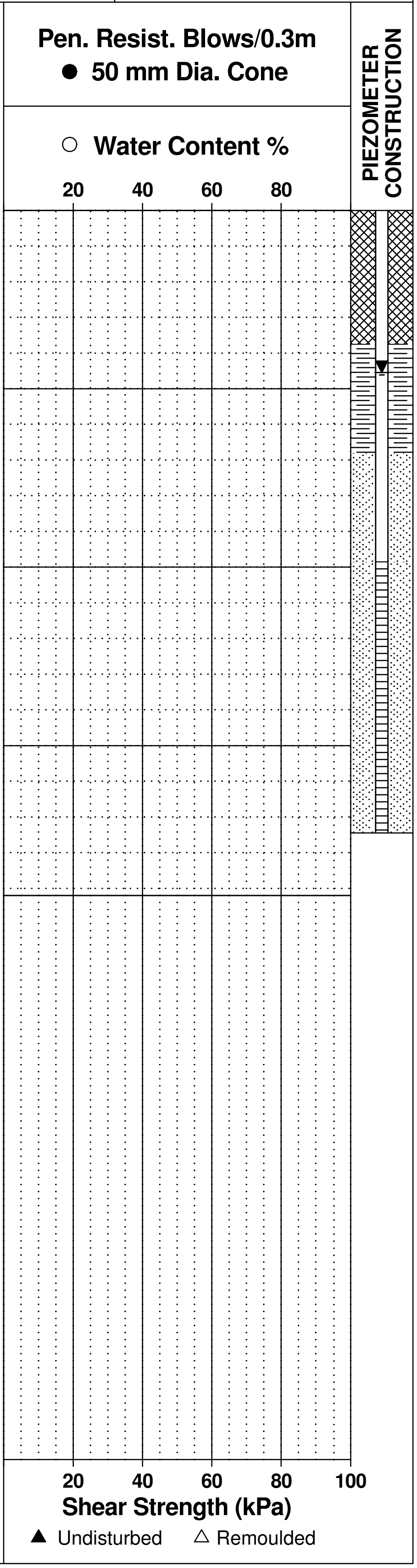
FILE NO. PG7043

HOLE NO. BH10-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	123.09
FILL: Brown silty sand with crushed stone and gravel	SS	1	54	24		
0.74						
Loose, sandy **TOPSOIL**	SS	2	79	4	1	122.09
1.22						
Loose to compact, brown **SILTY SAND** and **SANDY SILT**						
- trace clay from 1.8m to 2.0m depth	SS	3	75	6	2	121.09
	SS	4	71	12		
					3	120.09
- grey by 3.0m depth						
3.28						
GLACIAL TILL: Compact, grey silty sand to sandy silt, some gravel, trace clay, occasional cobbles and boulders	SS	5	95	20		
3.84						
End of Borehole						

Practical refusal to augering at 3.84m depth

(GWL @ 0.91m - March 28, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348785.713 **NORTHING:** 5014165.321 **ELEVATION:** 123.41

DATUM: Geodetic

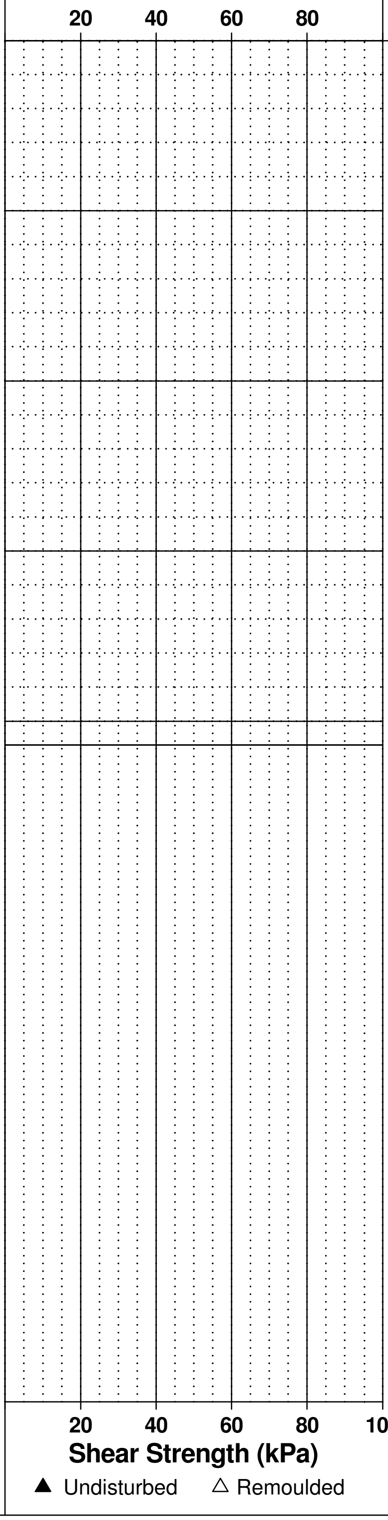
REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043

HOLE NO. PH 1-24

SAMPLE 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 %	PIEZOMETER CONSTRUCTION
Ground Surface									
OVERBURDEN						0	123.41		
						1	122.41		
						2	121.41		
						3	120.41		
						4	119.41		
4.14									
End of Borehole Practical refusal to augering at 4.14m depth									



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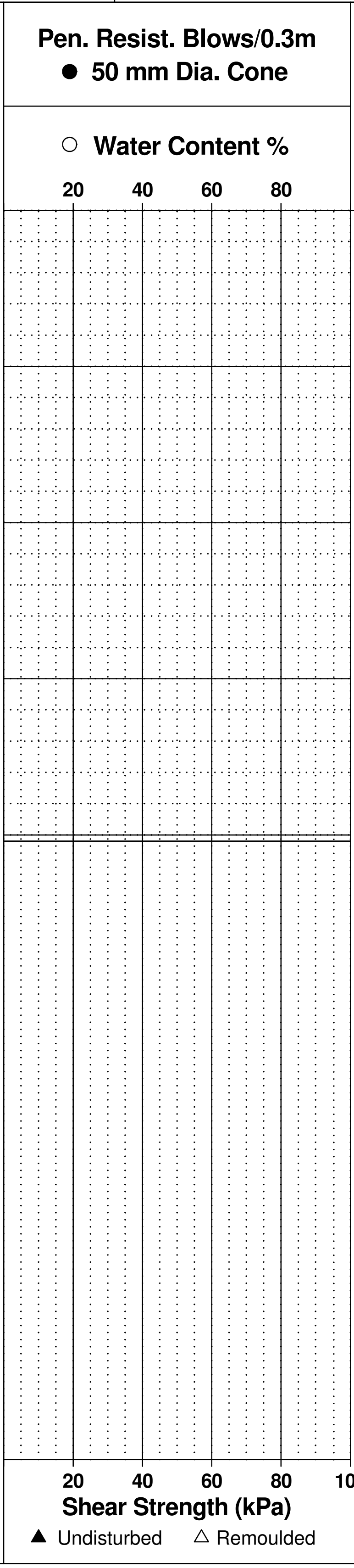
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348781.975 **NORTHING:** 5014142.904 **ELEVATION:** 123.01
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. PH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
Ground Surface						0	123.01	
FILL: Topsoil with gravel, cobbles and boulders		AU	1					
0.91								
Loose, sandy **TOPSOIL**		SS	2	88	4	1	122.01	
1.37								
OVERBURDEN						2	121.01	
						3	120.01	
4.04						4	119.01	
End of Borehole Practical refusal to augering at 4.04m depth								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348750.962 **NORTHING:** 5014124.082 **ELEVATION:** 123.29
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. PH 3-24

SAMPLE 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									
FILL: Brown silty sand with crushed stone and gravel		AU	1			0	123.29		
0.71									
Loose, brown **SILTY SAND**		SS	2	67	6	1	122.29		
1.37									
OVERBURDEN						2	121.29		
						3	120.29		
						4	119.29		
						5	118.29		
						6	117.29		
6.32									
End of Borehole Practical refusal to augering at 6.32m depth									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348751.45 **NORTHING:** 5014142.519 **ELEVATION:** 123.20

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043

HOLE NO. PH 4-24

SAMPLE 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 %	PIEZOMETER CONSTRUCTION
Ground Surface						0	123.20		
FILL: Brown silty sand with crushed stone and gravel	▨	AU	1						
- trace clay by 0.8m depth	▨								
1.07						1	122.20		
TOPSOIL with wood and roots	■	SS	2	67	4				
1.37									
OVERBURDEN						2	121.20		
						3	120.20		
3.43									
End of Borehole Practical refusal to augering at 3.43m depth									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348725.295 **NORTHING:** 5014092.472 **ELEVATION:** 123.43

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043

HOLE NO. PH 5-24

SAMPLE 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									
TOPSOIL with gravel and crushed stone 0.76		AU	1			0	123.43		
Loose, brown **SILTY SAND**, trace gravel 1.37		SS	2	25	10	1	122.43		
OVERBURDEN 2.57						2	121.43		
End of Borehole Practical refusal to augering at 2.57m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348710.604 **NORTHING:** 5014110.005 **ELEVATION:** 123.51
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. PH 6-24

SAMPLE 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 %	PIEZOMETER CONSTRUCTION
Ground Surface									
FILL: Brown silty sand with crushed stone and gravel, trace topsoil (to 0.71)		AU	1			0	123.51		
Loose, brown **SILTY SAND** (to 1.37)		SS	2	83	10	1	122.51		
OVERBURDEN (to 2.95)						2	121.51		
End of Borehole Practical refusal to augering at 2.95m depth									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348670.398 **NORTHING:** 5014090.461 **ELEVATION:** 123.68
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. PH 7-24

SAMPLE 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	123.68		
FILL: Brown silty sand with gravel and crushed stone		AU	1						
		SS	2	88	17	1	122.68		
1.22									
Loose, brown **SILTY SAND**									
1.37									
OVERBURDEN						2	121.68		
2.87									
End of Borehole Practical refusal to augering at 2.87m depth									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348696.255 **NORTHING:** 5014066.969 **ELEVATION:** 123.69
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043
HOLE NO. PH 8-24

SAMPLE 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	123.69		
FILL: Brown silty sand, trace gravel and cobbles		AU	1						
- topsoil from 0.7 to 0.8m depth									
		SS	2	75	10	1	122.69		
1.37									
GLACIAL TILL									
						2	121.69		
						3	120.69		
3.94									
End of Borehole Practical refusal to augering at 3.94m depth									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348697.326 **NORTHING:** 5014098.304 **ELEVATION:** 123.55

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043

HOLE NO. PH 9-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
Ground Surface						0	123.55		
FILL: Brown silty sand with gravel and crushed stone, trace topsoil and asphalt 0.61		AU	1						
GLACIAL TILL: Loose, brown silty sand to sandy silt, trace clay and gravel 1.37		SS	2	75	6	1	122.55		
OVERBURDEN 2.46						2	121.55		
End of Borehole Practical refusal to augering at 2.46m depth									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348733.344 **NORTHING:** 5014111.234 **ELEVATION:** 123.43

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 20, 2024

FILE NO. PG7043

HOLE NO. PH10-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
Ground Surface						0	123.43		
FILL: Brown silty sand with gravel and cobbles, trace topsoil and asphalt 0.64	▨	AU	1						
Compact, brown **SILTY SAND** to **SANDY SILT** 1.37		SS	2	67	14	1	122.43		
OVERBURDEN 1.93									
End of Borehole Practical refusal to augering at 1.93m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

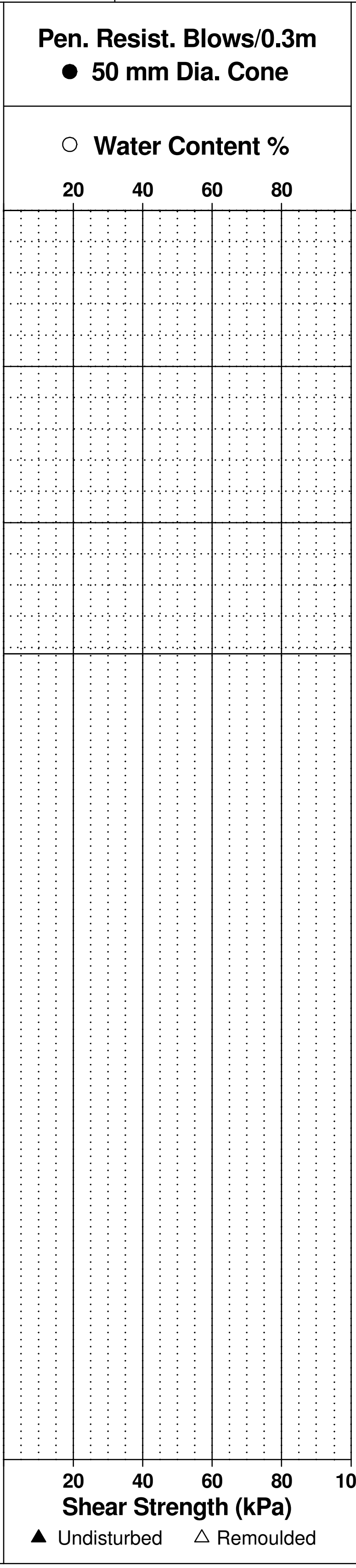
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348717.307 **NORTHING:** 5014123.692 **ELEVATION:** 123.59
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH11-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	123.59
FILL: Brown silty sand, some to trace gravel, cobbles and crushed stone	▨	AU	1				
		SS	2	75	10	1	122.59
		SS	3	50	+50		
1.98						2	121.59
GLACIAL TILL: Very dense, grey silty sand to sandy silt, some gravel and cobbles, occasional boulders		SS	4	100	+50		
2.84							
End of Borehole Practical refusal to augering at 2.84m depth							



PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348742.329 **NORTHING:** 5014131.806 **ELEVATION:** 123.37
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH12-24

SAMPLE 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									
FILL: Brown silty sand with crushed stone and gravel 0.76		AU	1			0	123.37		
TOPSOIL 1.07									
Loose, brown **SILTY SAND** 1.37		SS	2	46	6	1	122.37		
OVERBURDEN						2	121.37		
						3	120.37		
4.65						4	119.37		
End of Borehole Practical refusal to augering at 4.65m depth									

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

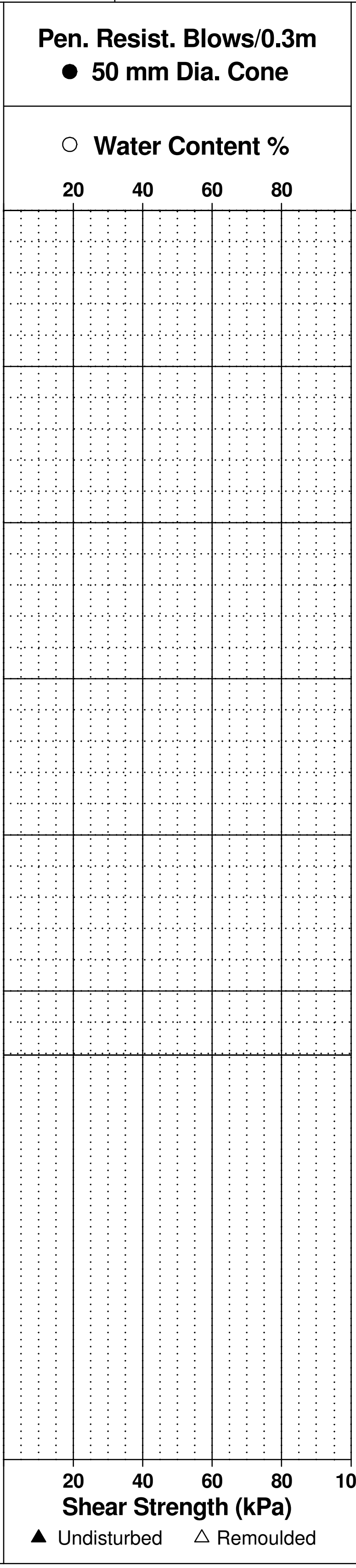
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348741.648 **NORTHING:** 5014121.675 **ELEVATION:** 123.40
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH13-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface							
FILL: Brown silty sand with crushed stone and gravel, trace topsoil	☒	AU	1			0	123.40
0.56							
Loose, brown **SILTY SAND**		SS	2	58	7	1	122.40
1.37							
OVERBURDEN						2	121.40
						3	120.40
						4	119.40
						5	118.40
5.41							
End of Borehole Practical refusal to augering at 5.41m depth							



PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348753.227 **NORTHING:** 5014136.956 **ELEVATION:** 123.26

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043

HOLE NO. PH14-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface				
FILL: Brown silty sand with crushed stone and gravel (0.84)	AU	1		
TOPSOIL, trace gravel (1.22)	SS	2	67	2
Very loose, brown **SILTY SAND** (1.37)				
OVERBURDEN (3.96)				
End of Borehole Practical refusal to augering at 3.96m depth				

DEPTH (m)	ELEV. (m)
0	123.26
1	122.26
2	121.26
3	120.26

STRATA PLOT

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348796.844 **NORTHING:** 5014152.613 **ELEVATION:** 123.42
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH15-24

SAMPLE 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									
FILL: Brown silty sand with crushed stone and gravel		AU	1			0	123.42		
		SS	2	71	13	1	122.42		
1.22									
TOPSOIL, trace gravel									
1.37									
OVERBURDEN						2	121.42		
						3	120.42		
						4	119.42		
4.45									
End of Borehole Practical refusal to augering at 4.45m depth									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348740.82 **NORTHING:** 5014167.109 **ELEVATION:** 123.14

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043

HOLE NO. PH16-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
Ground Surface		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
FILL: Brown silty sand with crushed stone and gravel, trace topsoil		AU	1			0	123.14		
0.76									
Sandy **TOPSOIL**						1	122.14		
1.07		SS	2	75	8				
Loose, brown **SILTY SAND**									
1.37									
OVERBURDEN						2	121.14		
						3	120.14		
3.91									
End of Borehole Practical refusal to augering at 3.91m depth									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348722.864 **NORTHING:** 5014185.024 **ELEVATION:** 123.42
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH17-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
Ground Surface		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
ASPHALTIC CONCRETE 0.13						0	123.42		
FILL: Brown silty sand, some crushed stone and gravel 0.56		AU	1						
Compact, brown **SILTY SAND** to **SANDY SILT** 1.37		SS	2	67	12	1	122.42		
End of Borehole Practical refusal to augering at 1.37m depth									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348697.922 **NORTHING:** 5014069.993 **ELEVATION:** 123.67

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043

HOLE NO. PH18-24

SAMPLE 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									
FILL: Brown silty sand with crushed stone and gravel 0.66		AU	1			0	123.67		
GLACIAL TILL: Loose, brown silty sand, trace gravel and cobbles 1.22		SS	2	75	5	1	122.67		
GLACIAL TILL: Dense, brown silty sand to sandy silt, some gravel, occasional cobbles and boulders 2.13		SS	3	100	34	2	121.67		
OVERBURDEN 2.77									
End of Borehole Practical refusal to augering at 2.77m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348736.751 **NORTHING:** 5014118.913 **ELEVATION:** 123.36

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043

HOLE NO. PH19-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
Ground Surface						0	123.36		
FILL: Brown silty sand, some crushed stone and gravel, trace topsoil 0.53		AU	1						
Loose, brown **SILTY SAND** 1.22		SS	2	71	9	1	122.36		
GLACIAL TILL: Compact, brown silty sand with gravel and cobbles, occasional boulders 1.37									
OVERBURDEN						2	121.36		
						3	120.36		
4.67						4	119.36		
End of Borehole Practical refusal to augering at 4.67m depth									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev. - 6310 & 6320 Hazeldean Road
Ottawa, Ontario

EASTING: 348681.377 **NORTHING:** 5014120.089 **ELEVATION:** 123.54
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-mounted auger **DATE:** March 21, 2024

FILE NO. PG7043
HOLE NO. PH20-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	123.54
FILL: Brown silty sand with crushed stone and gravel		AU	1				
0.64							
Compact, brown **SILTY SAND** to **SANDY SILT** - trace gravel by 1.3m depth		SS	2	100	13	1	122.54
1.37							
OVERBURDEN						2	121.54
						3	120.54
3.86							
End of Borehole Practical refusal to augering at 3.86m depth							

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

PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

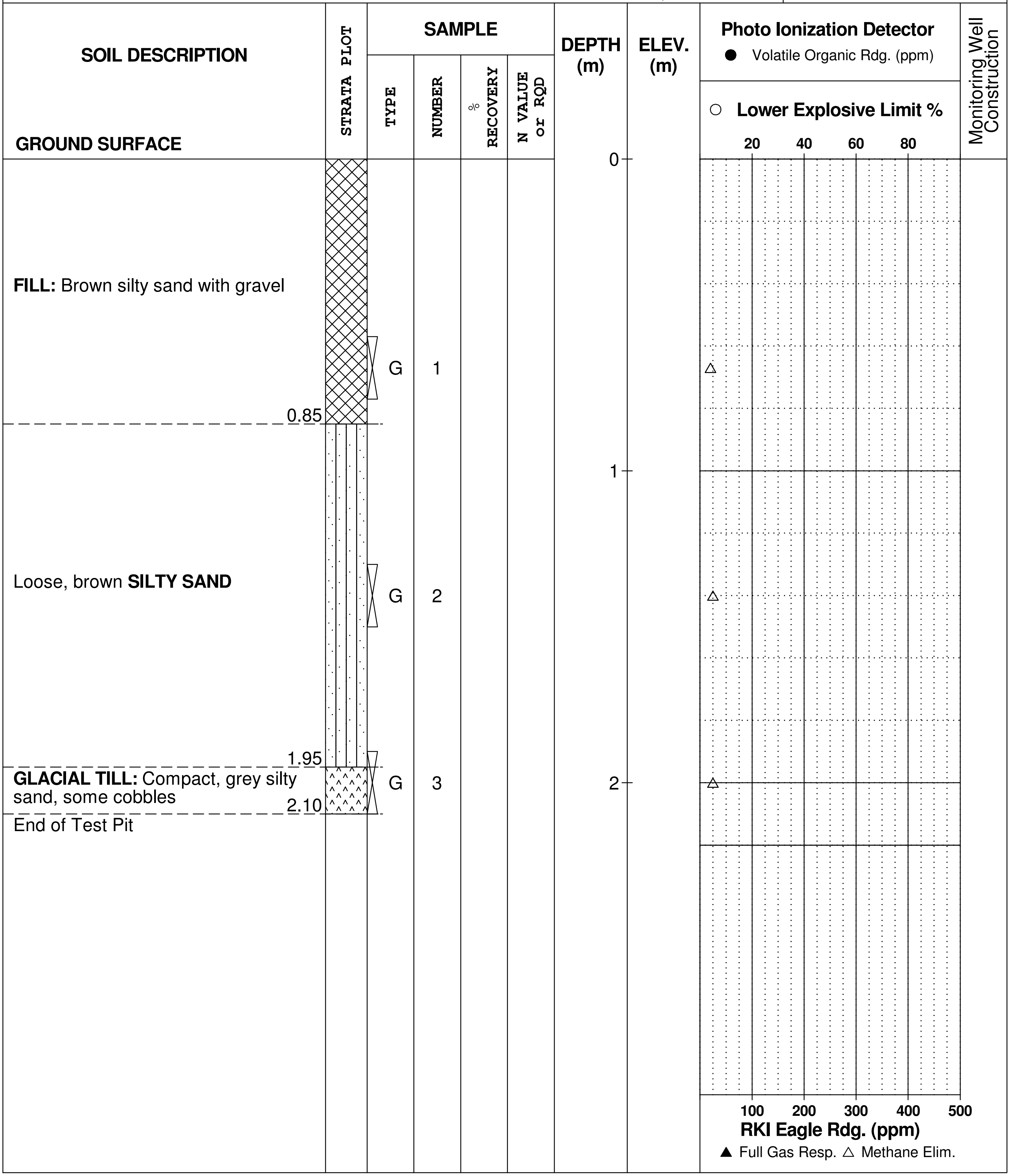
REMARKS

BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 1-20



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
FILL: Brown silty sand with gravel	G	1				
0.85						
Loose, brown **SILTY SAND**					1	
	G	2				
1.95						
GLACIAL TILL: Compact, grey silty sand, some cobbles	G	3			2	
2.10						
End of Test Pit						

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

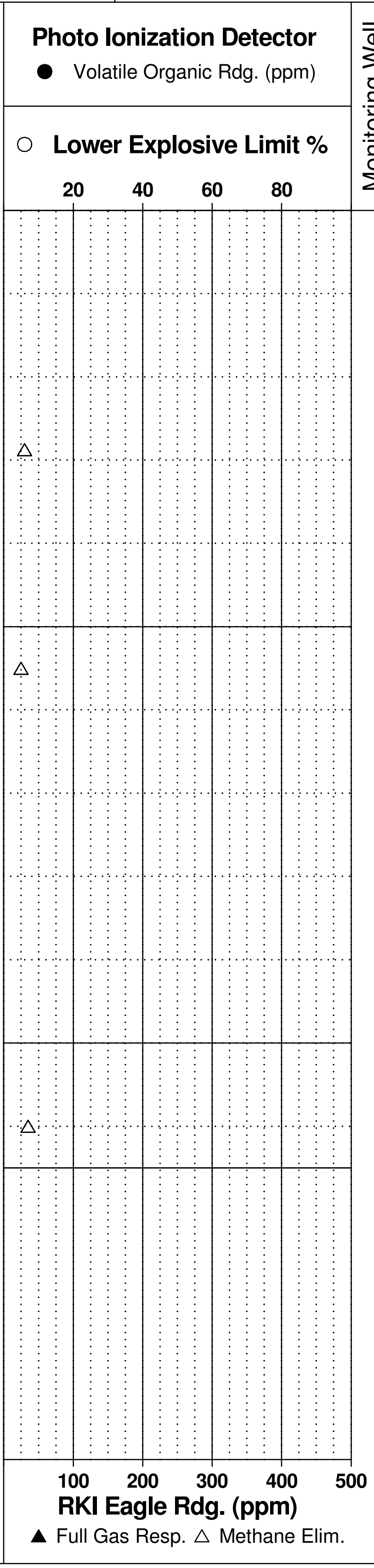
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 2-20

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		
FILL: Brown silty sand with organics		G	1					
0.70								
Loose, brown **SILTY SAND** - grey by 1.2m depth		G	2			1		
		G	3			2		
2.30								
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

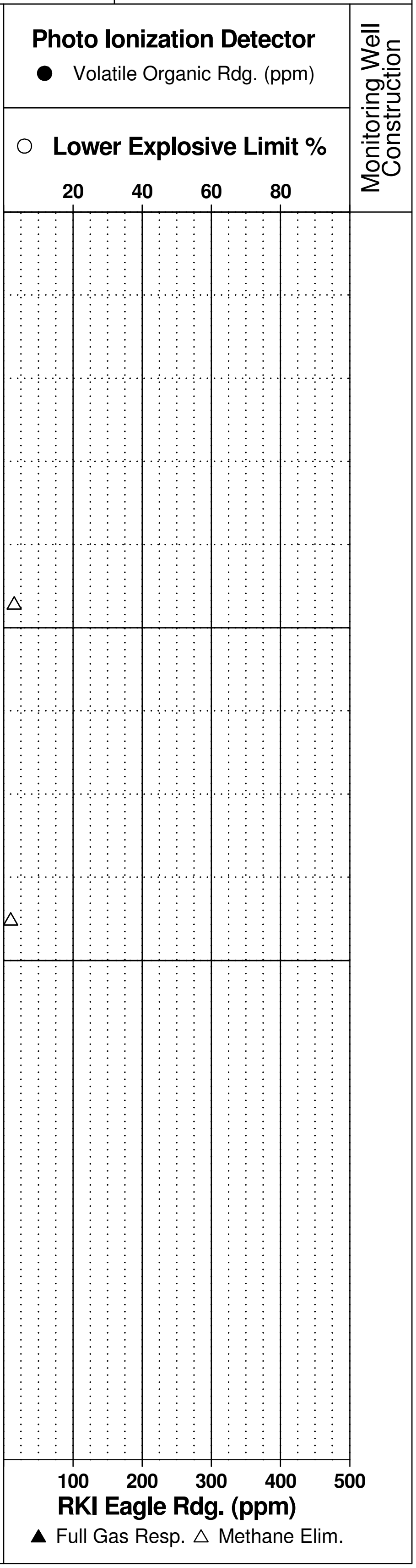
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Loose, brown **SILTY SAND**		G	1			1	
1.04							
GLACIAL TILL: Compact, grey silty sand with cobbles		G	2				
1.80							
End of Test Pit							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

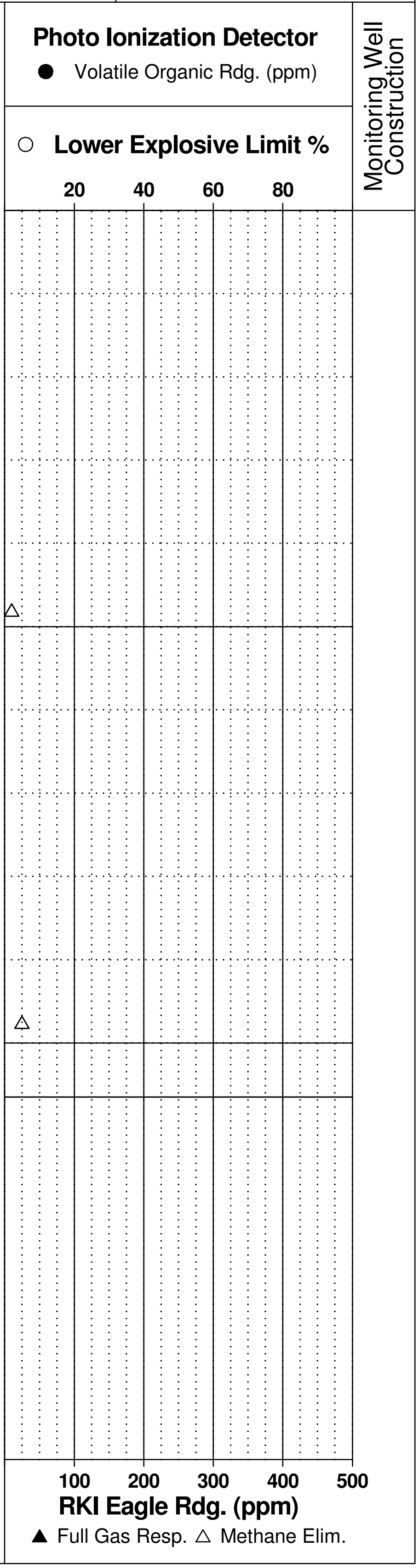
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	
Loose to compact, brown **SILTY SAND**		G	1			1	
- grey by 1.6m depth		G	2			2	
2.13 End of Test Pit							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

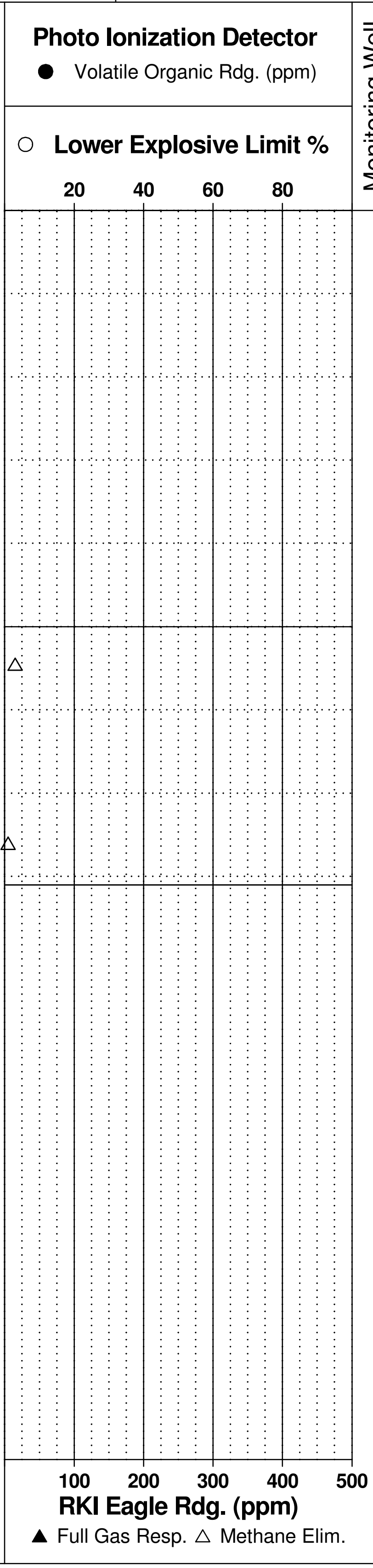
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 5-20

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		
Loose to compact, brown **SILTY SAND**								
						1		
		G	1					
		G	2					
1.62								
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

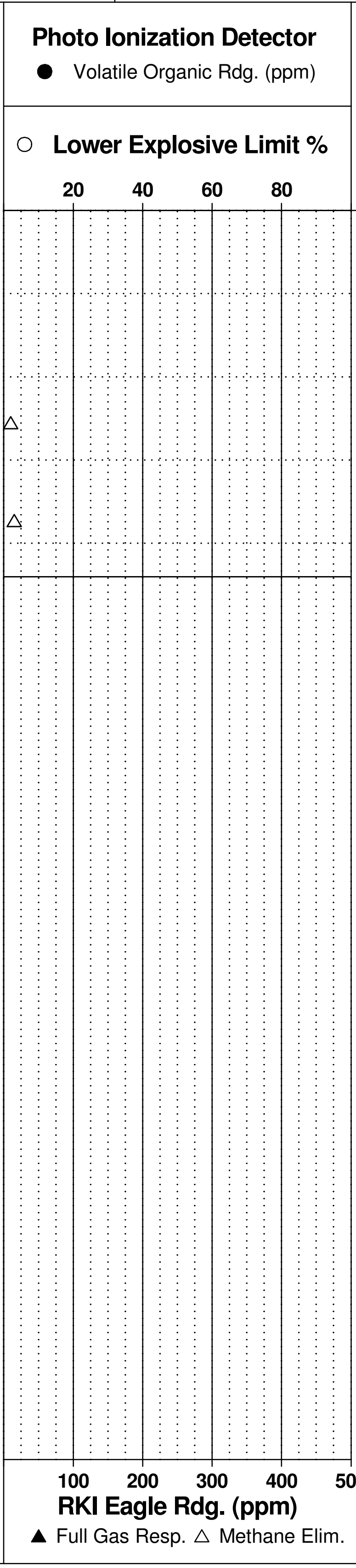
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 6-20

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		
FILL: Brown silty sand, some cobbles		G	1					
0.61								
Loose, brown **SILTY SAND**		G	2					
0.88								
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

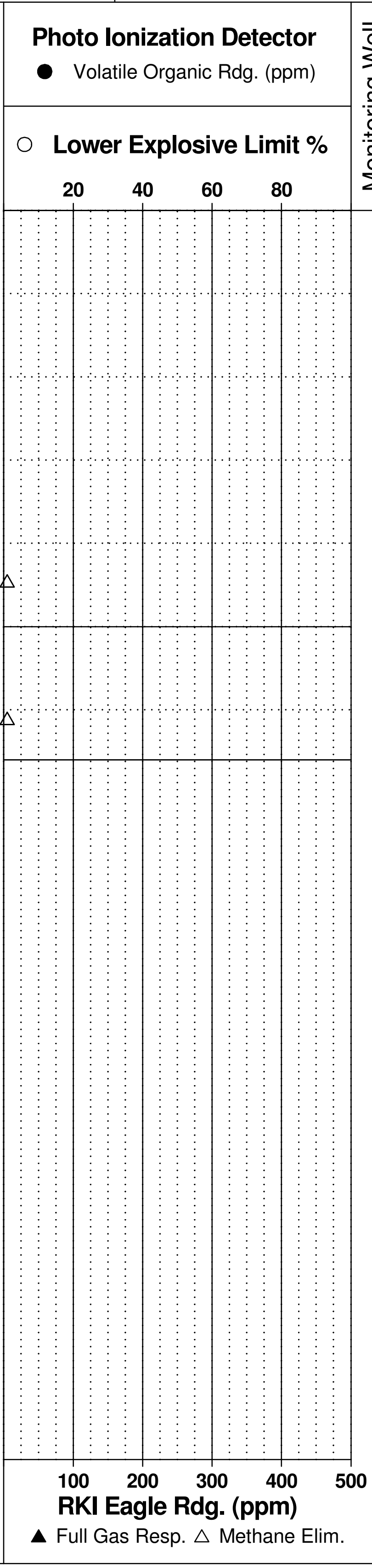
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 7-20

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		
FILL: Asphaltic concrete over gravel 0.68								
FILL: Dark brown silty sand 0.99		G	1					
						1		
Loose, brown **SILTY SAND** 1.32		G	2					
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

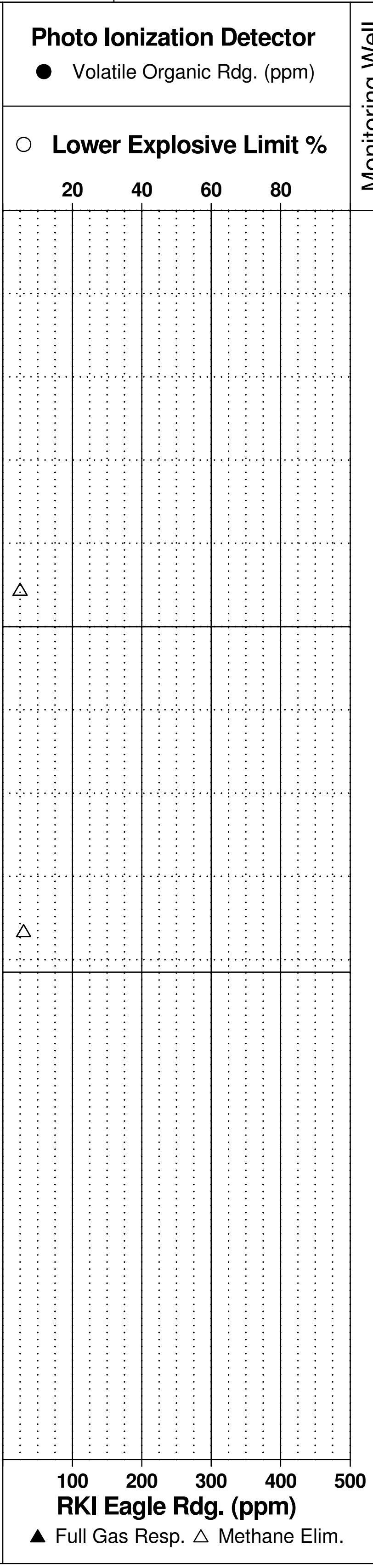
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 8-20

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		
FILL: Asphaltic concrete over gravel 0.41								
FILL: Brown silty sand with gravel 1.21		G	1			1		
Loose, brown **SILTY SAND** 1.83		G	2					
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

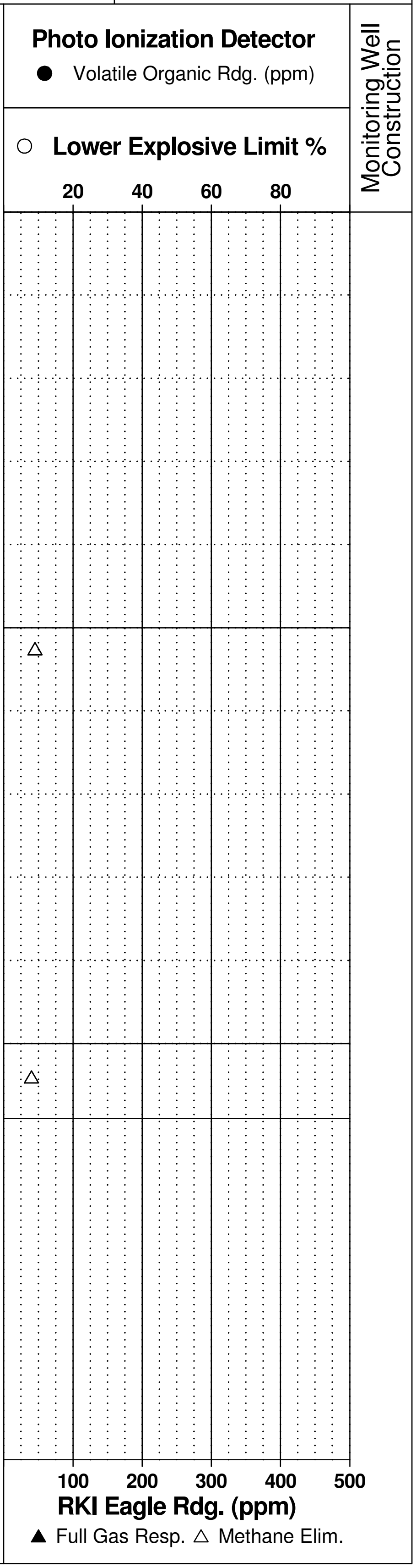
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP 9-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Brown silty sand, some organics and cobbles							
		G	1			1	
1.21							
Loose, brown **SILTY SAND**							
		G	2			2	
2.18							
End of Test Pit							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

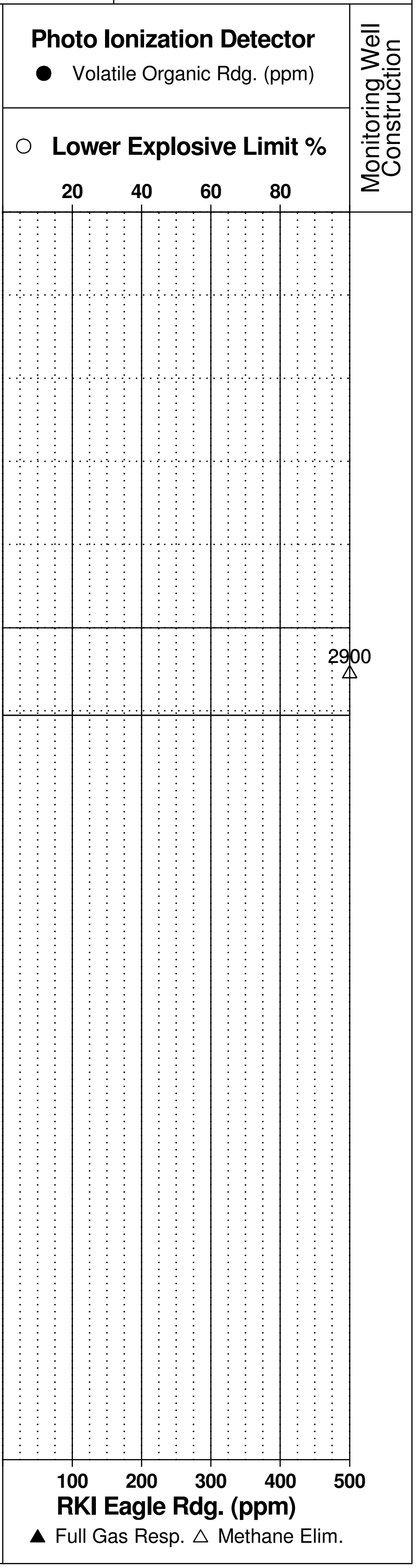
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP10-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Dark brown silty sand	▨						
	▨	G	1			1	
1.21							
End of Test Pit							



patersongroup Consulting Engineers

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment Update
6310 and 6320 Hazeldean Road
Ottawa, Ontario

DATUM

REMARKS

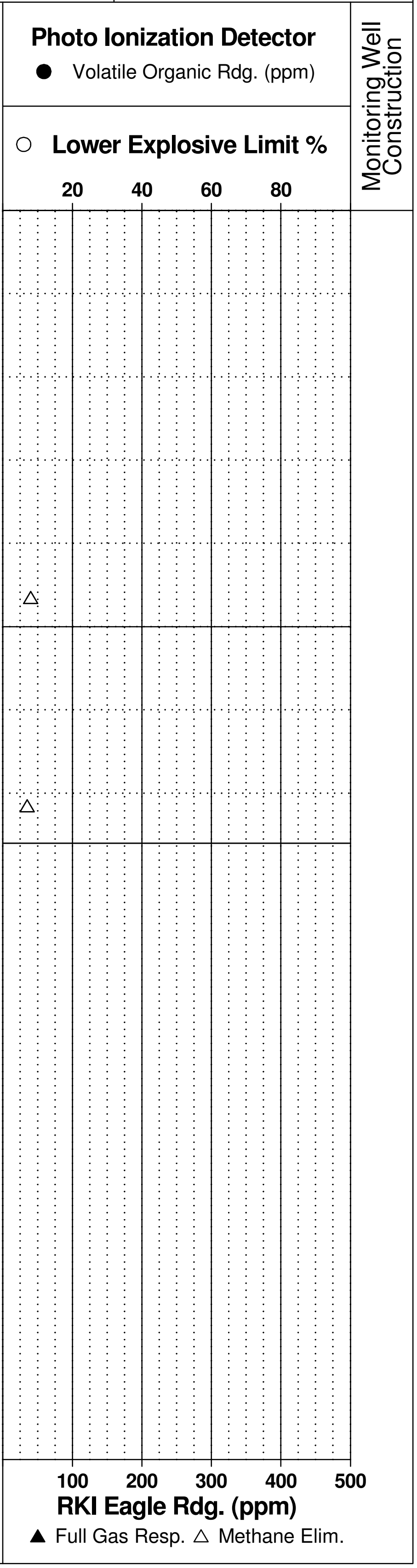
BORINGS BY Backhoe

DATE December 17, 2020

FILE NO. PE4484

HOLE NO. TP11-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Asphaltic concrete over gravel							
0.78							
Loose to compact, brown **SILTY SAND**		G	1			1	
- grey-brown by 1.3m depth		G	2				
1.52							
End of Test Pit							



PATERSON GROUP

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Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. PG4757

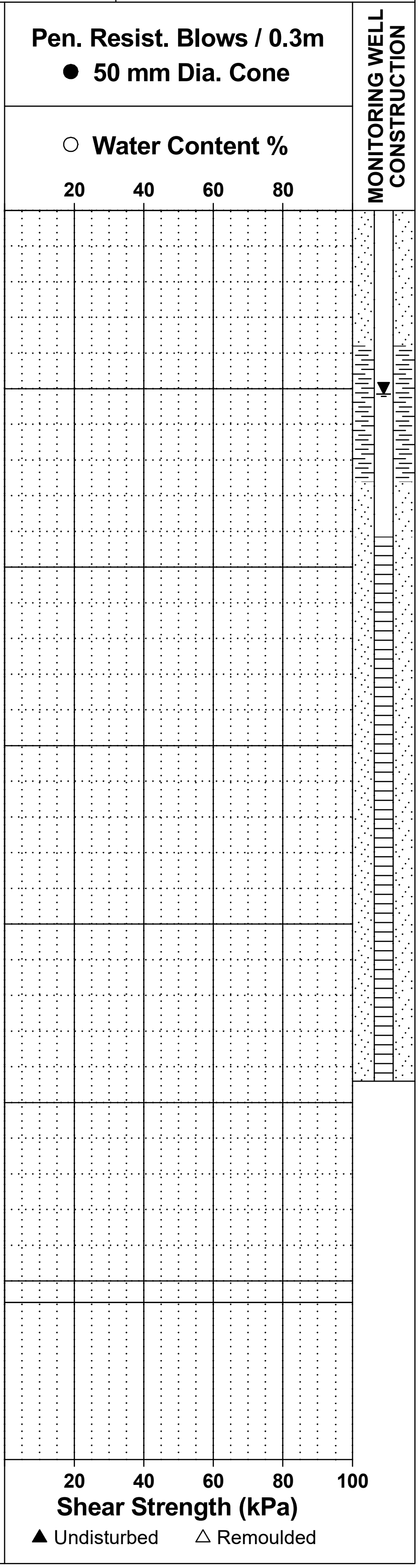
REMARKS:

HOLE NO. BH 1

BORINGS BY: CME 55 Power Auger

DATE: November 29, 2018

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.44
FILL: Crushed stone 0.25							
FILL: Brown silty sand, trace clay 0.60		AU	1				
Very loose, brown **SILTY SAND,** trace clay 1.52		SS	2	75	3	1	122.44
Compact, brown **SAND,** some silt 1.98		SS	3	83	22	2	121.44
GLACIAL TILL: Very dense to dense, grey silty sand with gravel, cobbles and boulders, some clay		SS	4	83	97	3	120.44
		SS	5	100	50+	4	119.44
		SS	6	75	50+		
		SS	7	100	55	5	118.44
		SS	8	100	30	6	117.44
6.12		SS	9				
End of Borehole Practical refusal to augering at 6.12m depth (GWL @ 1.03m - Dec. 11, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. **PG4757**

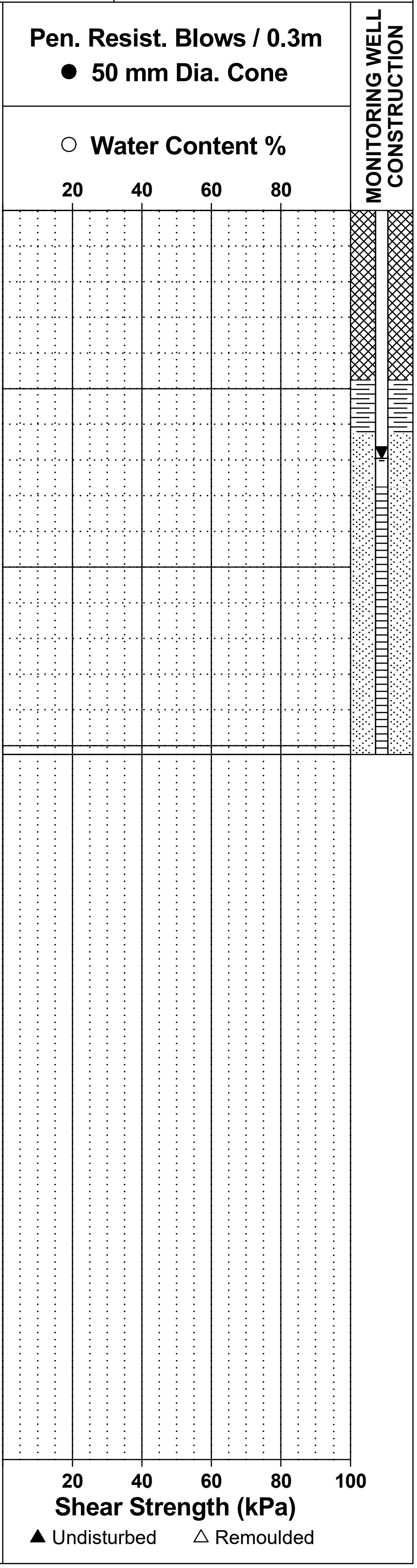
REMARKS:

HOLE NO. **BH 2**

BORINGS BY: CME 55 Power Auger

DATE: November 29, 2018

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.68
FILL: Crushed stone	0.30	AU	1				
Very loose, brown **SILTY SAND**	0.76						
GLACIAL TILL: Brown clayey silt with sand and gravel to sand with silt, gravel, cobbles and boulders, some clay		SS	2	83	2	1	122.68
		SS	3	92	38	2	121.68
	3.05	SS	4	100	50+	3	120.68
End of Borehole (GWL @ 1.39m - Dec. 11, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. PG4757

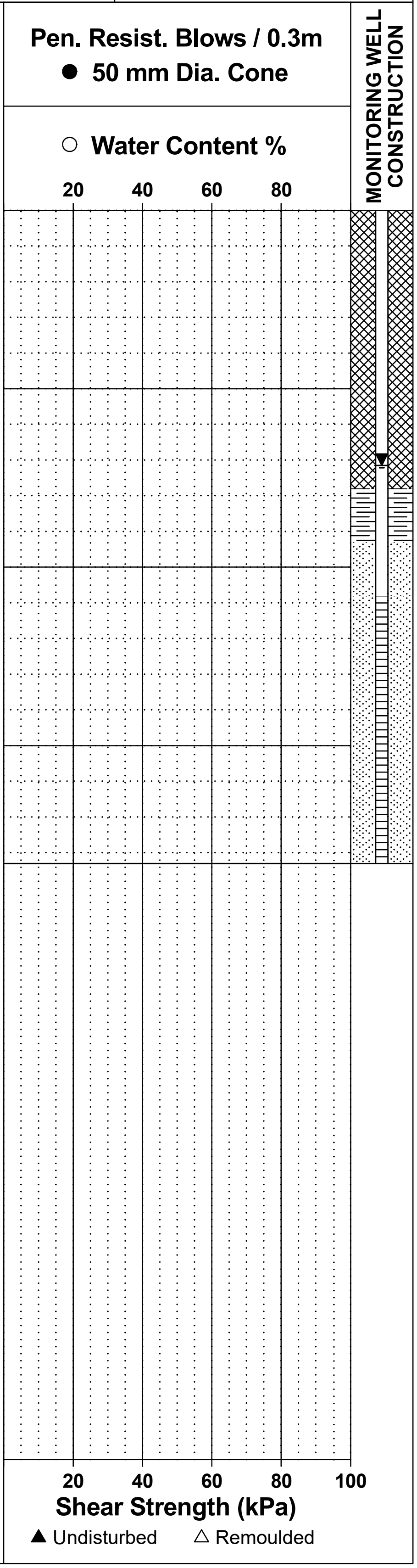
REMARKS:

HOLE NO. BH 3

BORINGS BY: CME 55 Power Auger

DATE: November 29, 2018

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.14
FILL: Crushed stone 0.30		AU	1				
FILL: Brown silty sand, some gravel 1.00		SS	2	83	6	1	122.14
TOPSOIL 1.68							
Compact, brown **SAND,** some silt, trace clay 2.13		SS	3	83	22	2	121.14
Compact to loose, brown **SILTY SAND,** trace clay		SS	4	58	10	3	120.14
- grey by 3.4m depth 3.66		SS	5	100	9		
End of Borehole (GWL @ 1.43m - Dec. 11, 2018)							



PATERSON GROUP

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. PG4757

REMARKS:

HOLE NO. BH 4

BORINGS BY: CME 55 Power Auger

DATE: November 29, 2018

SAMPLE DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Crushed stone	0.30
FILL: Brown silty sand, some clay and gravel	0.76
TOPSOIL	1.07
Compact to loose, brown **SAND,** some silt	2.59
GLACIAL TILL: Compact to very dense, grey silty sand, some clay, gravel, cobbles and boulders	3.96

End of Borehole

Practical refusal to augering at 3.96m depth

(GWL @ 0.92m - Dec. 11, 2018)

STRATA PLOT

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	92	14
SS	3	58	8
SS	4	83	23
SS	5	100	50+
SS	6	100	50+

DEPTH (m)	ELEV. (m)
0	123.02
1	122.02
2	121.02
3	120.02

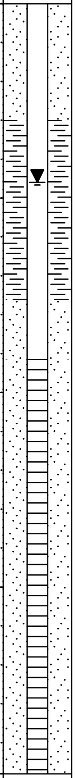
Pen. Resist. Blows / 0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

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

MONITORING WELL CONSTRUCTION



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. PG4757

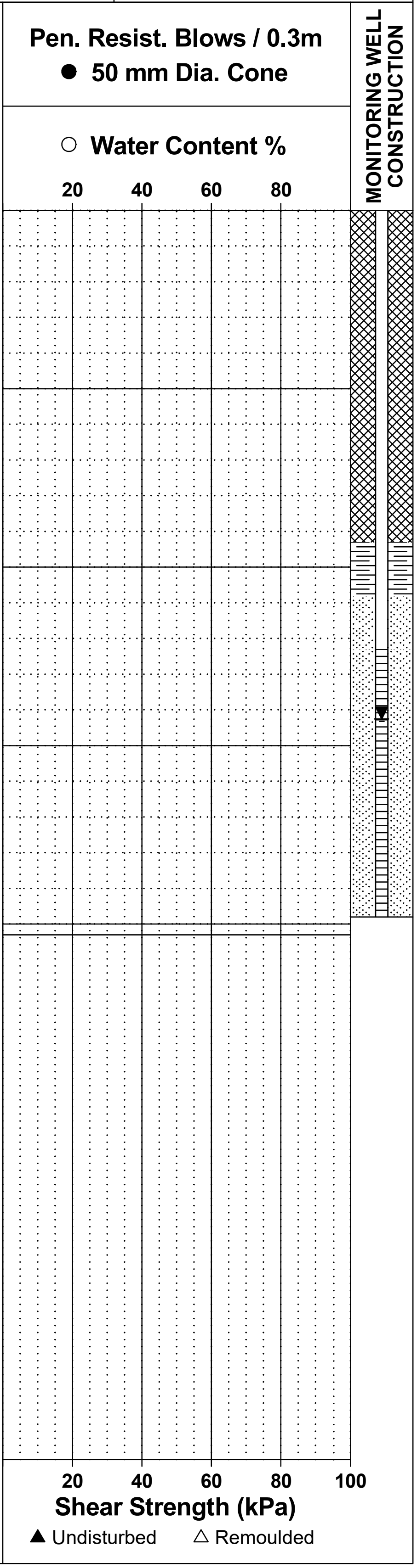
REMARKS:

HOLE NO. BH 5

BORINGS BY: CME 55 Power Auger

DATE: November 29, 2018

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.28
FILL: Crushed stone	0.30	AU	1				
FILL: Brown silty sand, some clay and gravel	0.76						
TOPSOIL	1.07	SS	2	75	5	1	122.28
Loose to compact, brown **SAND,** some silt		SS	3	58	5		
						2	121.28
	2.59	SS	4	83	32		
						3	120.28
GLACIAL TILL: Compact to very dense, grey silty sand with sand, gravel, cobbles and boulders		SS	5	67	23		
	3.96	SS	6	100	50+		
End of Borehole Practical refusal to augering at 3.96m depth (GWL @ 2.85m - Dec. 11, 2018)						4	119.28



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. **PG4757**

REMARKS:

HOLE NO. **BH 6**

BORINGS BY: CME 55 Power Auger

DATE: November 30, 2018

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.67
FILL: Crushed stone	0.20						
FILL: Brown sand with silt, clay and gravel	0.46	AU	1				
Loose, brown **SILTY SAND,** some clay	0.76						
		SS	2	92	9	1	122.67
		SS	3	100	70	2	121.67
GLACIAL TILL: Brown sand to silty sand with clay, gravel, cobbles and boulders		SS	4	100	50+		
		SS	5	100	50+	3	120.67
	4.17	SS	6	100	50+	4	119.67

End of Borehole

Practical refusal to augering at 4.17m depth

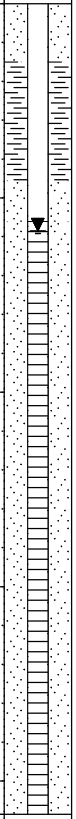
(GWL @ 1.17m - Dec. 11, 2018)

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



PATERSON GROUP

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Ottawa, Ontario
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TEL: (613) 226-7381

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 6310 Hazeldean Rd.
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in front of garage. Elevation = 125.26m.

FILE NO. PG4757

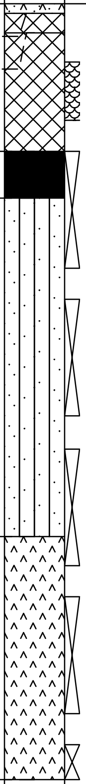
REMARKS:

HOLE NO. BH 7

BORINGS BY: CME 55 Power Auger

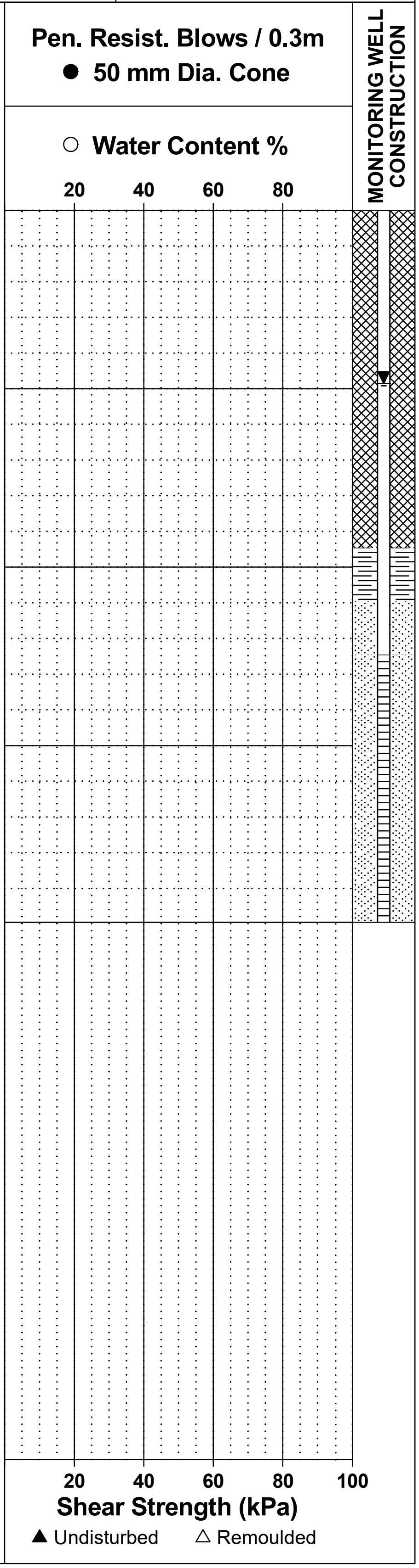
DATE: November 30, 2018

SAMPLE DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone	0.15
FILL: Brown silty sand, some clay and gravel	0.76
TOPSOIL	1.00
Compact, grey **SILTY SAND**	2.74
GLACIAL TILL: Compact to very dense, grey silty sand with clay, gravel , cobbles and boulders	3.99
End of Borehole (GWL @ 0.97m - Dec. 11, 2018)	



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	6
SS	3	83	15
SS	4	92	24
SS	5	83	78
SS	6	86	50+

DEPTH (m)	ELEV. (m)
0	123.43
1	122.43
2	121.43
3	120.43



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)

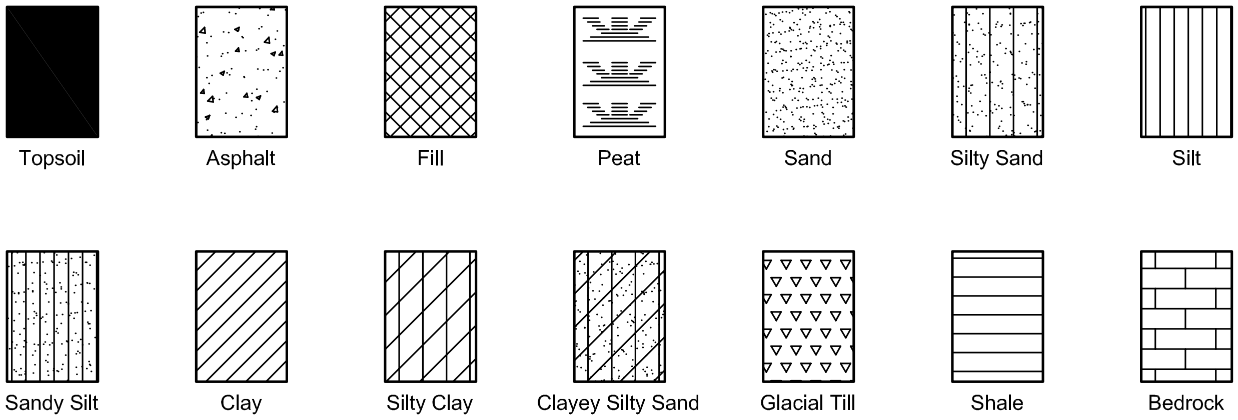
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

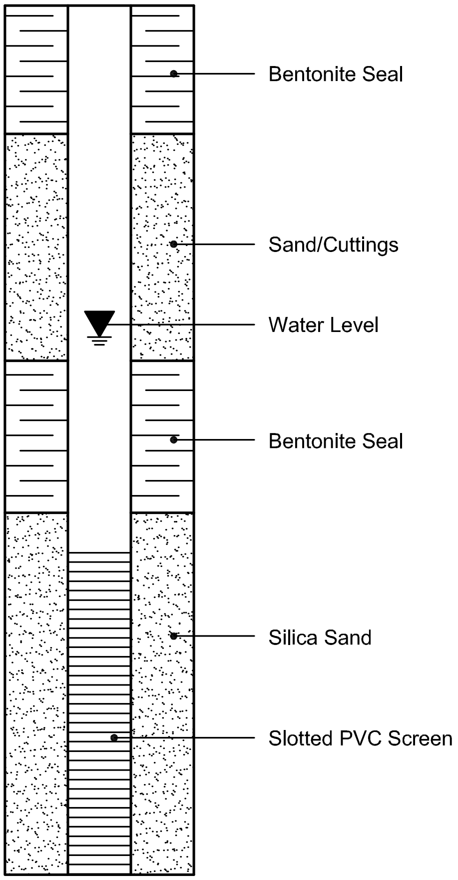
SYMBOLS AND TERMS (continued)

STRATA PLOT

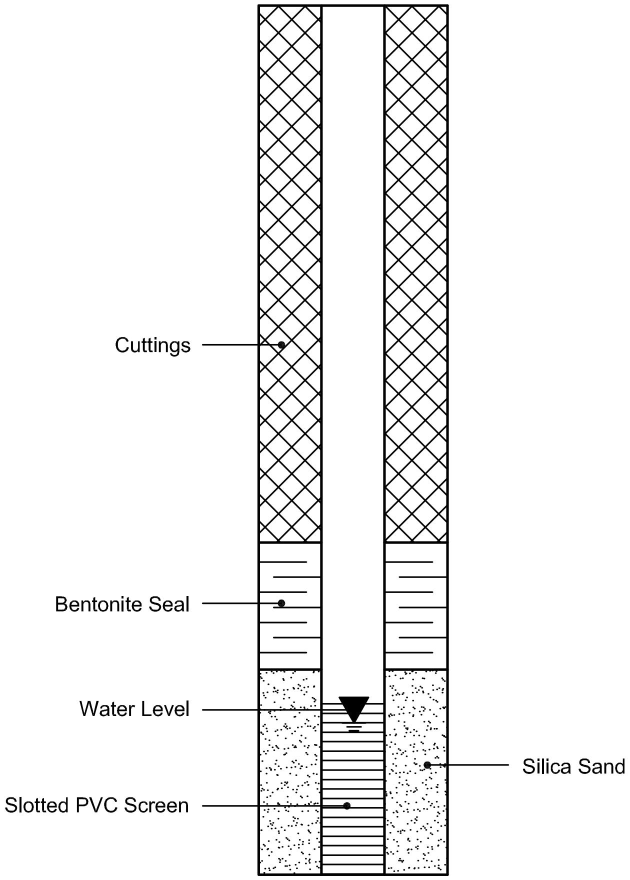


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**9441-6302 Quebec Inc.**	FILE No.:	**PG7043**
PROJECT:	Lab Testing	REPORT No.:	1
SITE ADDRESS:		DATE REPT'D:	25-Mar-24
STRUCTURE TYPE & LOCATION:	Grey Limestone Core		

SAMPLE INFOROMATION			
LAB NO.:	**51334**		
SAMPLE NO.:	-		
LOCATION:	BH6-24 / RC2 / 21'11"- 22'3"		
SAMPLE DATES			
DATE CAST	-		
DATE CORED	19-Mar-24		
DATE RECEIVED	22-Mar-24		
DATE TESTED	25-Mar-24		
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00		
HEIGHT (mm)	94.00		
WEIGHT (g)	460		
AREA (mm^2)	1735		
VOLUME (cm^3)	163		
UNIT WEIGHT (kg/m^3)	**2821**		
TEST RESULTS			
H / D RATIO	2.00		
CORRECTION FACTOR	1.000		
LOAD (lbs)	28700		
GROSS Mpa	73.6		
MPa CORRECTED	**73.6**		
FORM OF BREAK	**Type A**		
DIRECTION OF LOADING	PARALLEL		
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		

REMARKS

TECHNICAL PERSONNEL

TECHNICIAN:	**VERIFIED BY:**	**C. Beadow**	**APPROVED BY:**	**Joe Forsyth, P. Eng.**

CERTIFIED LAB

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

Certificate of Analysis

Report Date: 27-Mar-2024

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 22-Mar-2024

Client PO: 59731

Project Description: PG7043

	Client ID:	BH3-24 SS3	-	-	-		
	Sample Date:	22-Mar-24 09:00	-	-	-	-	-
	Sample ID:	2412426-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	81.0	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.23	-	-	-	-	-
Resistivity	0.1 Ohm.m	95.7	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	11	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG7043-1 – TEST HOLE LOCATION PLAN

DRAWING PG7043-2 – BEDROCK CONTOUR PLAN

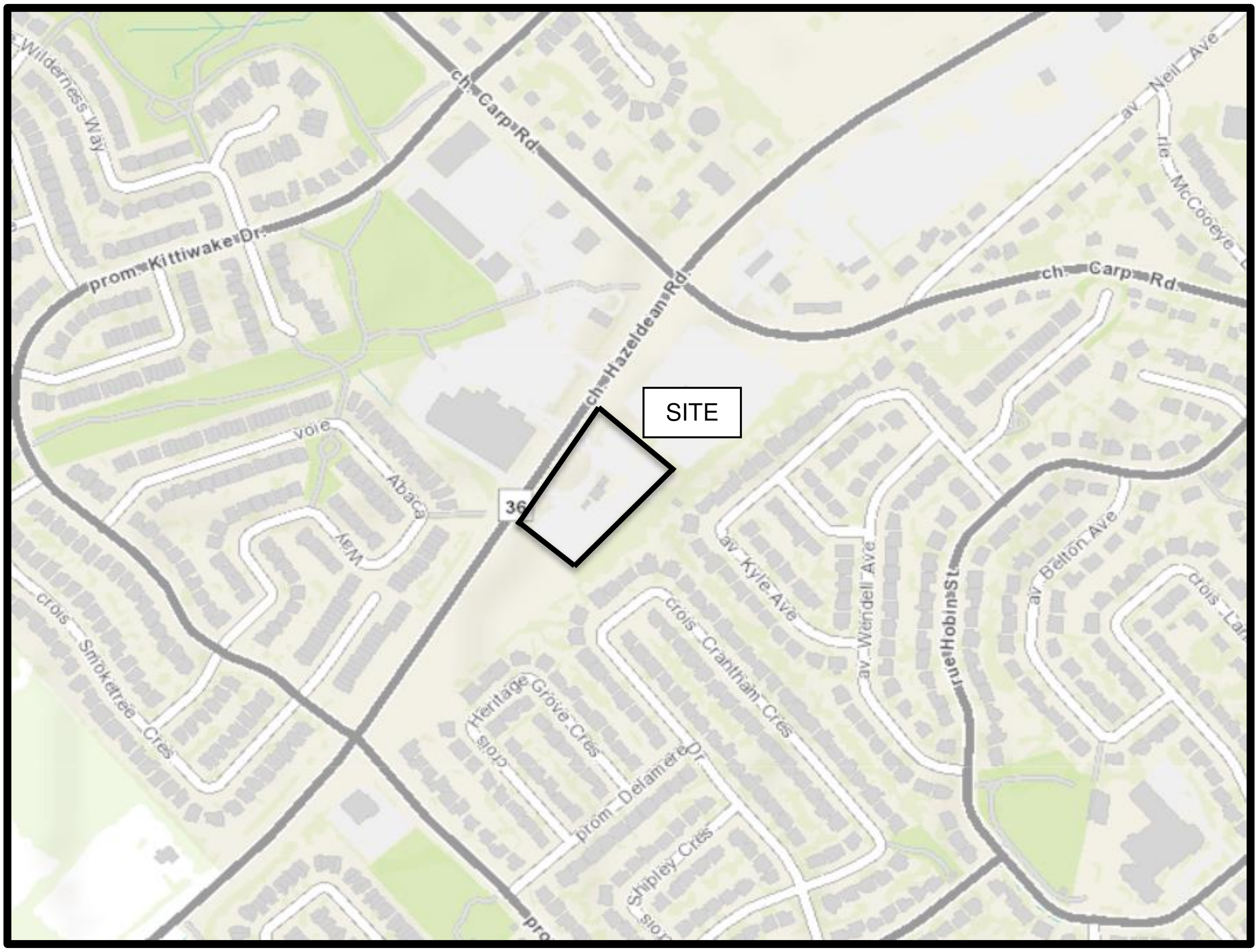


FIGURE 1

KEY PLAN

PATERSON
GROUP

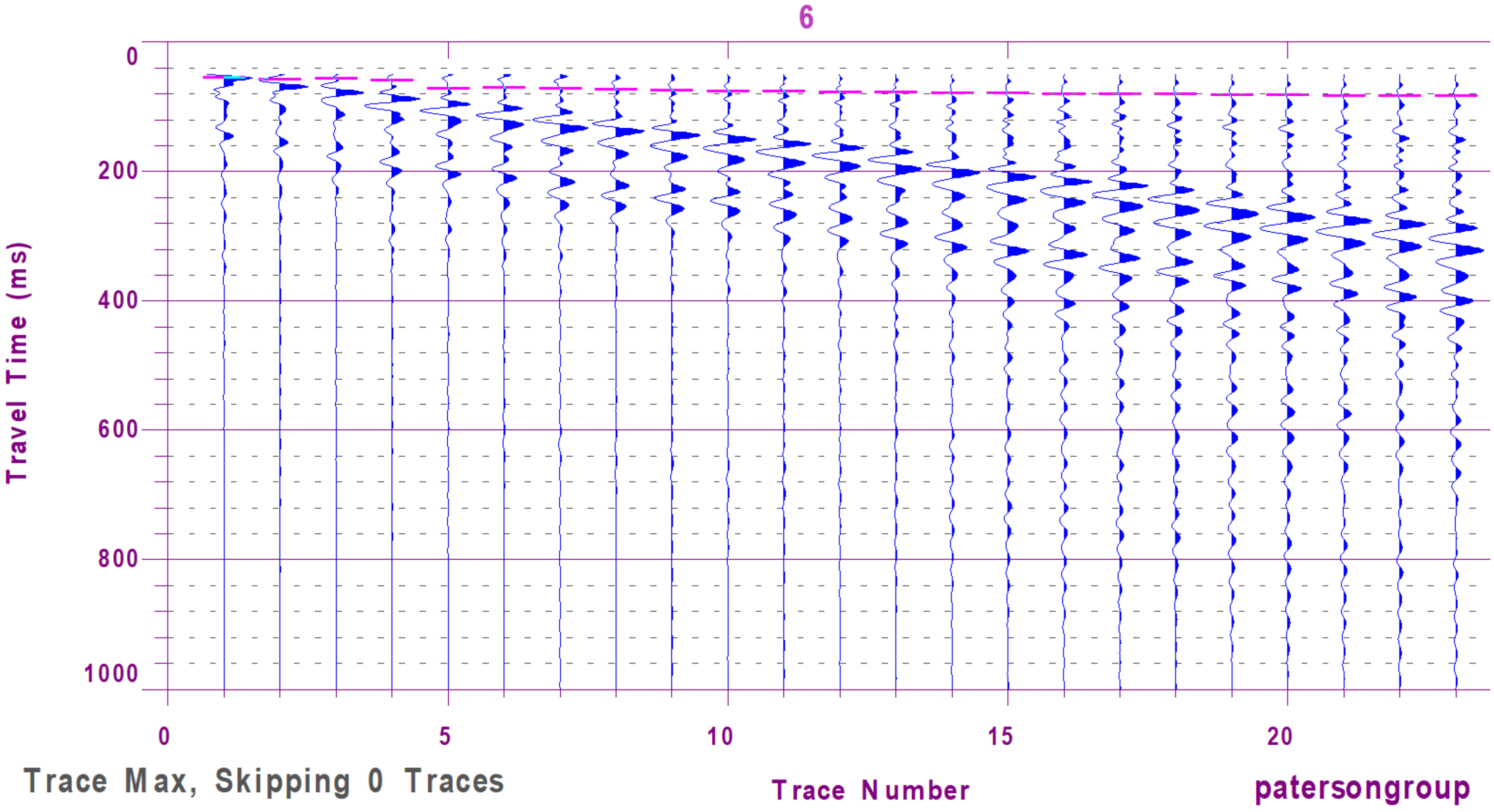


Figure 2 – Shear Wave Velocity Profile at Shot Location -3 m

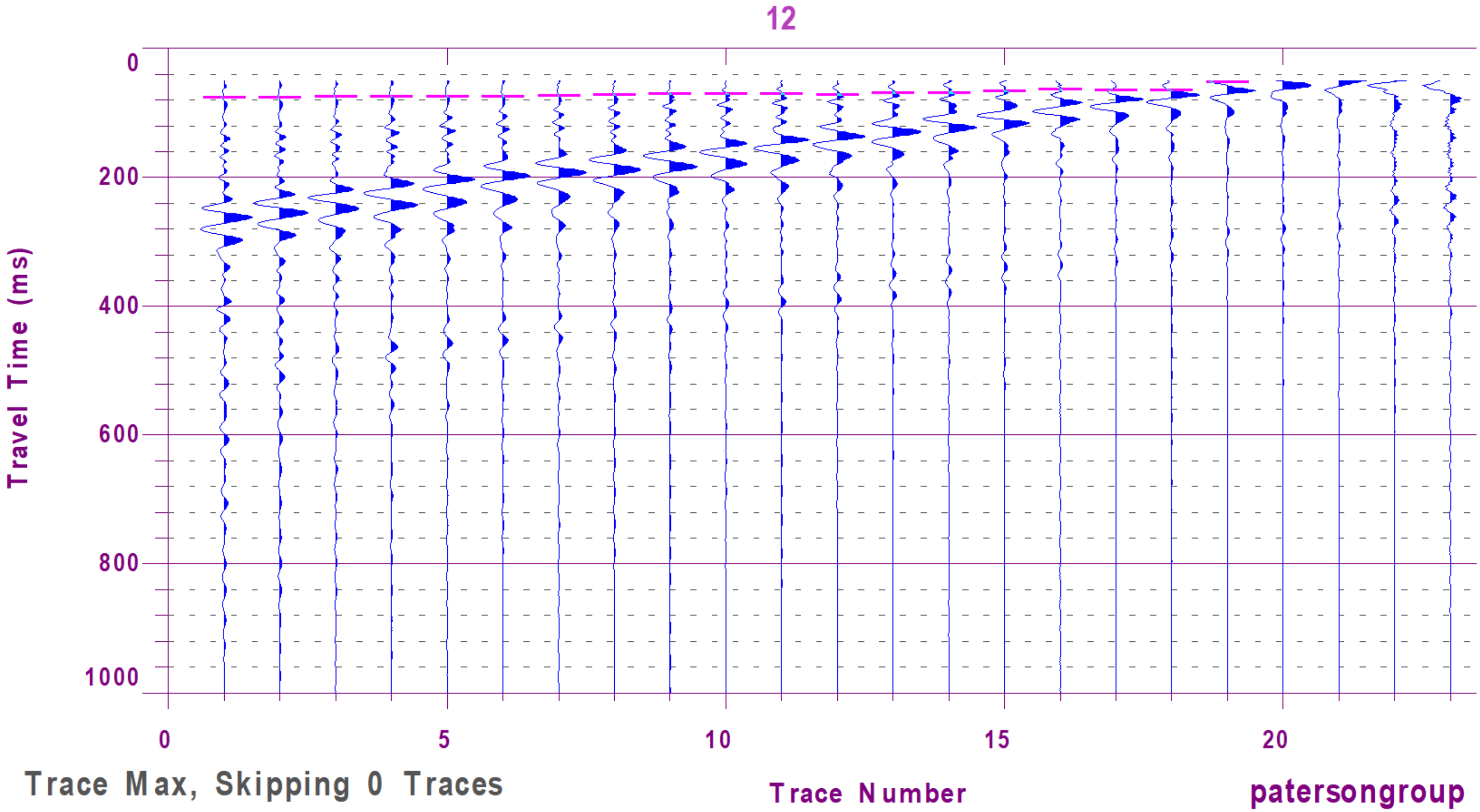
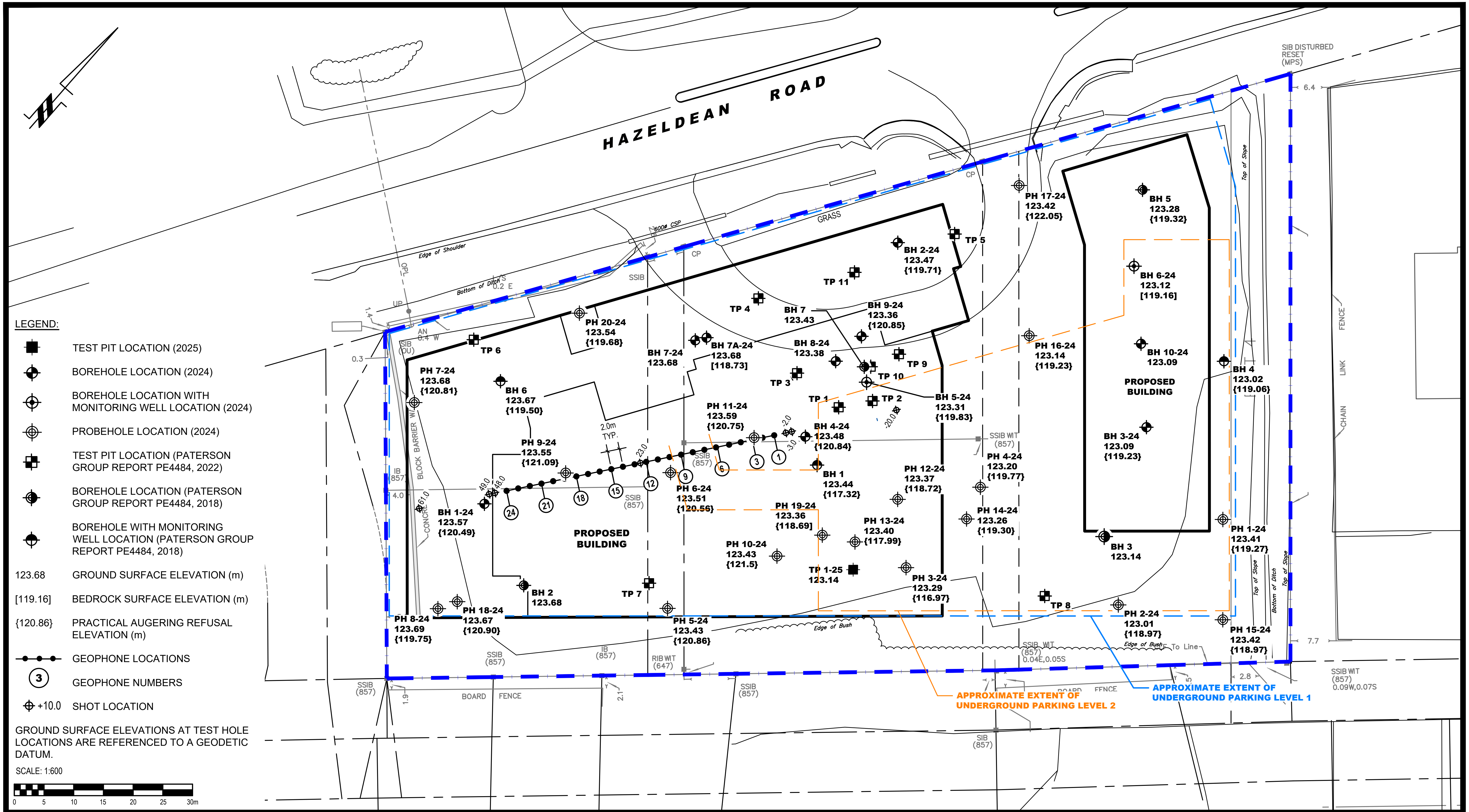


Figure 3 – Shear Wave Velocity Profile at Shot Location 48 m



LEGEND:

■	TEST PIT LOCATION (2025)
⊕	BOREHOLE LOCATION (2024)
⊕	BOREHOLE LOCATION WITH MONITORING WELL LOCATION (2024)
⊕	PROBEHOLE LOCATION (2024)
▣	TEST PIT LOCATION (PATERSON GROUP REPORT PE4484, 2022)
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT PE4484, 2018)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT PE4484, 2018)
123.68	GROUND SURFACE ELEVATION (m)
[119.16]	BEDROCK SURFACE ELEVATION (m)
{120.86}	PRACTICAL AUGERING REFUSAL ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBERS
⊕ +10.0	SHOT LOCATION

GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:600

0 5 10 15 20 25 30m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	AS PER REVISED CONCEPTUAL SITE PLAN	03/11/2025	DP
1	TP 1-25 ADDED	28/07/2025	DP

9441-6302 QUEBEC INC. (STITTSVILLE)
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE RESIDENTIAL DEVELOPMENT
6310 & 6320 HAZELDEAN ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 04/2025
Drawn by: YA	Report No.: PG7043-1
Checked by: KB	Dwg. No.: PG7043-1
Approved by: DP	Revision No.: 2

11x17

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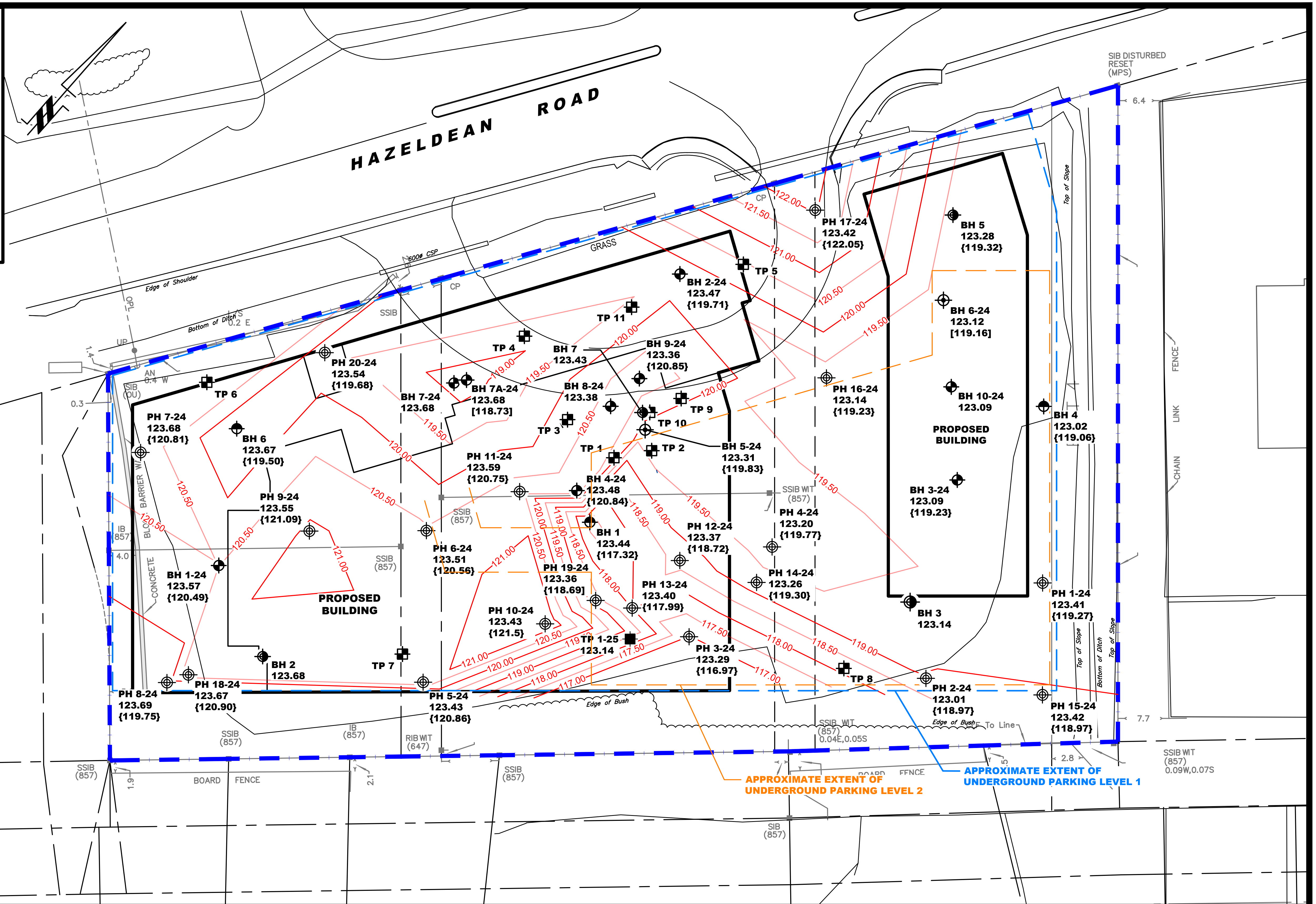
NOTE FOR BEDROCK CONTOUR LINES:

BEDROCK CONTOURS DEPICTED HEREIN ARE BASED ON LINEAR INTERPOLATION BETWEEN TEST HOLES WHERE BEDROCK HAS BEEN ENCOUNTERED BY PATERSON THROUGHOUT THE SUBJECT SITE AND IS LIMITED TO THAT INFORMATION.

ACTUAL SITE CONDITION AND BEDROCK DEPTHS/ELEVATIONS MAY VARY BEYOND TEST HOLE LOCATIONS AND AS DEPICTED BY THE CONTOUR LINES.

BASED ON THIS, IT SHOULD BE UNDERSTOOD THAT THE BEDROCK SURFACE MAY VARY WITHIN PLUS OR MINUS 0.5 TO 1.5 M AT CONTOUR LINE LOCATIONS WHERE THE BEDROCK SURFACE HAS NOT BEEN DISCRETELY CONFIRMED BY A TEST HOLE.



LEGEND:

■	TEST PIT LOCATION (2025)
⊕	BOREHOLE LOCATION (2024)
⊕	BOREHOLE LOCATION WITH MONITORING WELL LOCATION (2024)
⊕	PROBEHOLE LOCATION (2024)
⊞	TEST PIT LOCATION (PATERSON GROUP REPORT PE4484, 2022)
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT PE4484, 2018)
●	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT PE4484, 2018)
—121.00—	BEDROCK CONTOURS (m)
123.68	GROUND SURFACE ELEVATION (m)
[119.16]	BEDROCK SURFACE ELEVATION (m)
{120.86}	PRACTICAL AUGERING REFUSAL ELEVATION (m)

GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:600

0 5 10 15 20 25 30m

PATERSON GROUP

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OTTAWA, ON
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NO.	REVISIONS	DATE	INITIAL
2	AS PER REVISED CONCEPTUAL SITE PLAN	03/11/2025	DP
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9441-6302 QUEBEC INC. (STITTSVILLE)
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE RESIDENTIAL DEVELOPMENT
6310 & 6320 HAZELDEAN ROAD

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:600
Date: 04/2025
Drawn by: ZS
Report No.: PG7043-1
Checked by: KB
Dwg. No.: PG7043-2
Approved by: DP
Revision No.: 2

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