Geotechnical Investigation

Proposed High-Rise Building

2145 Walkley Road
Ottawa, Ontario

Prepared for LS GP Inc.

Report PG4440-2 Revision 2 dated March 21, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by LS GP Inc. to conduct a geotechnical investigation for the proposed development to be located at 2145 Walkley 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 drawings, it is understood the proposed development will be provided with a 14-storey high-rise building will be constructed over 5 storeys of underground parking basement levels. The underground parking area is anticipated to extend east and beyond the high-rise footprint and will be provided with a one-storey above-ground garage structure. Associated access roads and landscaped areas are also expected.

It is further expected that the proposed development will be municipally serviced with water and sewer services and abut the existing parking garage to the north of the proposed structure.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on March 12 and 13, 2018 and December 11, 2018. At that time, six (6) boreholes (BH 4 to BH 9) were advanced to a maximum depth of 15 m below existing ground surface. A supplemental investigation was conducted from December 7 to 9, 2020. During the supplemental field investigation four (4) boreholes (BH 1-20 to BH 4-20) were advanced to bedrock and 3 m of rock were cored to a maximum depth of 18 m below existing ground surface. Paterson completed an additional supplemental investigation between February 26 to February 28, 2025, which consisted of advancing eight (8) boreholes to a maximum depth of 19.8 m below ground surface.

The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG4440-1 - Test Hole Location Plan in Appendix 2. The boreholes were drilled with either a truck-mounted or rubber-track low-clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test hole are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at four (4) borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Diamond drilling was carried out at BH1-20 to BH4-20, and BH 1-25, to determine the nature of the bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. 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

PVC groundwater monitoring wells were installed at BH 5, BH 1-25, BH 6-25 and BH 7-25 and a flexible polyethylene standpipe was installed at BH 4 and BH 6 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below and as completed by licensed well technicians in accordance with O.Reg 903:

- ❑ Slotted 51 mm or 32 mm diameter PVC screen at the base of the aforementioned boreholes.
- ❑ PVC riser pipe from the top of the screen to the ground surface.
- ❑ No.3 silica sand backfill within annular space around screen.
- ❑ A minimum of 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❑ Clean backfill from top of bentonite plug to the ground surface.

The groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations carried out by Paterson were determined by Paterson personnel taking into consideration of 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 PG4440-1 - 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.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing during the 2018 investigation 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.

3.5 Hydraulic Conductivity Testing

Hydraulic conductivity testing was conducted at one (1) monitoring well location to provide insight on the hydraulic properties of the bedrock at the subject site. The test data was analyzed using AQTESOLV Pro Version 4.5 aquifer analysis software package by HydroSOLVE Inc and the results were processed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous aquifer of infinite extent and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.03 m. While the idealized assumptions regarding aquifer extent and homogeneity are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. The testing results are further discussed in Subsection 4.4 of this report.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an asphalt surfaced parking area and associated access lane for the existing high-rise residential structures adjacent to the subject site. An existing parking garage is located immediately north of the proposed building, with rooftop parking matching existing grade of the subject site.

The site is bordered to the north by existing multi-storey residential buildings, to the east by landscaped areas for the existing development, to the south by landscaped areas and Walkley Road and to the west by an existing high school property. The site has a gentle gradient down from north to south across the existing parking area, and a steeper slope down from west to east within the access lane toward Walkley Road.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of an asphalt pavement structure or a topsoil layer, overlying a very stiff to firm silty clay deposit. Practical refusal to DCPT was encountered at between 12.9 and 15.5 m depth at BH 4, BH 2-25, BH 3-25, BH 4-25, BH 5-25 and BH 6-25. An asphalt and granular fill layer was encountered over the silty clay deposit. The glacial till generally consisted of silty clay with a variable amount of sand, gravel, cobbles and boulders. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

A poor (RQD values ranging between 25%-50%) to a good (RQD values ranging between 75% to 90%) quality black shale bedrock was encountered at BH 1-20, BH 2- 20, BH 3-20 and BH 4-20 at depths ranging between 12.6 and 14 m below existing ground surface. Good to excellent quality shale was encountered at BH 1-25 at depths ranging between 15.4 and 19.8 m. Based on available geological mapping, the local bedrock consists of shale of the Carlsbad formation.

Unconfined compressive strength (UCS) testing was carried out on two (2) bedrock samples recovered from the boreholes. The UCS test results are presented in Table 1.

Table 1 - Bedrock Unconfined Compressive Strength Results			
Sample Number	**Depth (m)**	**Elevation (m)**	**Compressive Strength (MPa)**
BH 1-20 RC 3	15.57	64.32	53.3
BH 4-20 RC 2	16.31	64.64	33.1
BH 1-25 RC 1	15.60	66.67	38.5
BH 1-25 RC 2	17.39	64.88	45.1

4.3 Groundwater

Groundwater levels were measured in the standpipes installed in the boreholes upon completion of the sampling program. The GWL readings are presented in Table 2 and on the Soil Profile and Test Data sheets in Appendix 1.

It should be noted that standpipe piezometers can become blocked, which can lead to lower-than-normal groundwater level readings and also groundwater levels can be influenced by surface water infiltrating the backfilled boreholes, which can lead to higher-than-normal groundwater level readings.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could 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 4	82.22	9.99	72.23	March 22, 2018
BH 5	82.47	2.80	79.67	
BH 6	81.43	2.10	79.33	
BH 1-25	82.27	6.56	75.71	March 10, 2025
BH 6-25	79.84	4.99	74.85	
BH 7-25	81.93	7.91	74.02	
Notes: The test holes were surveyed with respect to a geodetic datum.				

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity tests were conducted at one monitoring well location throughout the subject site on March 10, 2025. The testing results are summarized in Table 3 below.

Table 3 – Summary of Hydraulic Conductivity Testing Results.

Test Hole ID	Ground Surface Elevation (m)	Testing Depth Interval (m)	Testing Elevation Interval (m)	K (m/s)	Test Type	Soil Type
BH 1-25	82.27	18.24-19.76	64.03-62.53	$3.66x10^{-5}$	Falling Head	Bedrock
				$2.98x10^{-5}$	Rising Head	

Summary of Results

Hydraulic conductivity testing conducted at the monitoring well screened within the bedrock yielded hydraulic conductivity values ranging from approximately $2.98x10^{-5}$ to $3.66x10^{-5}$ m/s. Hydraulic conductivity values for the silty clay and glacial till are expected to be less than $1.00x10^{-7}$ m/s

These values are generally consistent with typical published values for glacial till, and bedrock. It should be noted that hydraulic conductivity may vary across the subject site depending on the composition/compaction and hydrostatic properties at a given location for the overburden and bedrock, respectively.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered satisfactory for the proposed development from a geotechnical perspective. It is expected that the proposed multi-storey residential building will be founded on footings placed directly on the surface sounded bedrock and/or lean concrete in-filled trenches extending to the underlying bedrock surface

Due to the presence of the silty clay deposit, grading throughout the subject site will be subjected to grade raise restrictions and is discussed in Section 5.3 of this report.

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

5.2 Site Grading and Preparation

Stripping Depth

It is expected that all existing building elements, infrastructure and overburden throughout the proposed buildings footprint will be removed. 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 some bedrock removal will be required to complete the underground parking level, and that higher quantities of this volume would be removed throughout larger footing footprints such as for shear walls and elevator shafts. 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.

Due to the fragmented and weathered nature of the shale bedrock formation, it is recommended that detailed excavations for footings within 300 mm of target depths be undertaken with a combination of smaller-sized excavators/mini-excavators and compressed air to minimize over-breaking and disturbance of the bedrock surface by heavy equipment and larger hydraulic excavators.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing of the overburden.

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require the above equipment. Vibrations, caused by blasting or construction operations could cause detrimental affects on the adjoining buildings and structures. Therefore, all vibrations are recommended to be limited.

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

The guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

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 or conventional excavation techniques may be controlled to reduce backbreak and overbreak, upon blasting and advancement of shovels, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile and excavation depth. 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 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 overbreak given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts/excavations and the bedrocks in-situ characteristics.

Consideration should be given to planning to setback line-drilling and blasting efforts from the extent of the excavation and from below the shoring footprint to minimize the potential for backbreak and overbreak beyond the footprint of the shoring system. The setback would be advanced towards the excavation perimeter by grinding the bedrock rather than blasting in an effort to minimize the potential backbreak that would occur and to provide a reasonably smooth surface for installing waterproofing membranes and shotcrete; however, this may incur additional time to prepare the bedrock. Any overbreak that occurs beyond the footprint of the shoring system should be planned to be immediately in-filled to mitigate potential for sloughing of retained overburden from behind the shoring system. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

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. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. 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.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in maximum 300 mm thick lifts and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls.

Under winter conditions, if snow and ice is present within the imported fill placed below future basement slabs, higher than tolerable amounts of 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 would 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.

Protection of Potential Expansive Bedrock

It is anticipated that expansive shale will be encountered at the subject site. Although the effects of expansive shale will not affect the proposed building structure, it is possible that it will affect the proposed basement floor slabs founded close to the shale bedrock.

A potential for heaving and rapid deterioration of the shale bedrock exists at this site. To reduce the long-term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed development footprint should be protected from excessive dewatering and exposure to ambient air. These requirements should be evaluated by Paterson during the excavation operations and should be discussed with Paterson during the design stage.

It is anticipated bedrock sidewall and bearing surface preparation efforts detailed in the following sections of this report will mitigate long-term concerns associated with expansive shale impacting the proposed structure.

5.3 Foundation Design

Bearing Resistance Values – High-Rise Building and Parking Garage

Footings placed on a minimum 75 mm thick mud slab placed upon a surface sounded shale bedrock can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

Due to the weathered nature of shale, it is expected the upper 1 m of bedrock may contain soil-bearing seams. Area of soil bearing seams may be advised to be removed from below the bearing surface if encountered at the design underside of footing elevation. Weathered-like shale that does not contain soil bearing seams may be left in place and as approved by Paterson field personnel.

Sub-Footing Mud Slabs Over Bedrock

It is expected the shale bearing medium will be fractured and weathered at the base of the excavation. Provided appropriate measures are taken to minimize disturbing the surface when completing detailed excavations for footings (reference should be made to the *Bedrock Removal* portion of Subsection 5.2), it is recommended that all surface sounded shale footing bearing mediums, approved and reviewed by Paterson personnel, be covered with a minimum 75 mm thick mud slab prior to footing construction. It is recommended this mud slab be placed within 48 hours of exposing the proposed bearing medium to mitigate potential further weathering and oxidization of the shale bedrock.

Based on this, it is recommended to provide a minimum 75 mm thick mud slab layer of lean concrete (minimum 15 MPa 28-day compressive strength) below the footing footprints to provide adequate bearing surface rigidity over the Paterson-reviewed and -approved shale bedrock bearing medium.

The mud slab is recommended to extend a minimum of 100 mm beyond all faces of the overlying footing structure. Further, the bearing surface which the mud slab will be cast upon should be reviewed and approved for conformity to the geotechnical design by Paterson field personnel prior to being placed over the bearing surface.

Lean Concrete Filled Trenches

Where bedrock is not encountered at the design underside of footing elevation, it is recommended to sub-excavate near-vertical trenches below the footing footprint to expose the underlying bedrock surface and backfill the trenches with lean concrete (minimum 15 MPa 28-day compressive strength). Where this is required, this lean concrete trench would substitute the aforementioned mud slab.

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 trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation surface should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a depth of 1.5 m). Once approved by Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. Given the highly weathered nature of the in-situ shale formation, care and additional efforts will need to be taken to minimize overbreak and over-excavation of the shale bedrock surface once encountered if smaller-sized equipment cannot enter the trench safely.

It is recommended that Paterson personnel review the sub-excavation work prior to completing trenches to verify the confirm the depth of the bedrock surface to mitigate potential over-excavation into the shale formation. Once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

Bearing Resistance Values – Auxiliary At-Grade Structures

The following conventional spread footing bearing resistance values may be considered for portions of the proposed structure located beyond the building and underground parking footprint and consisting of lightly loaded ancillary structures founded relatively close to finished grade.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, very stiff brown silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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

Permissible Grade Raise

A permissible grade raise restriction of **2 m** can be used for design purposes. If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Settlement

Footings bearing on an acceptable surface sounded shale bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements. Auxiliary footings placed upon an undisturbed, in-situ, very stiff silty clay 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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site designation for the proposed buildings in accordance with Table 4.1.8.4.A of the Ontario Building Code 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array was located as presented in Drawing PG4440-1 - 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 two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer 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 18, 10, 3 and 3, 10 and 18 m away from the first and last geophone, and at the centre of the geophone 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, reflected 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 buildings foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases. Based on our testing results, the bedrock shear wave velocity is 1,834 m/s.

For foundations placed directly or indirectly (i.e., using lean-concrete in-filled trenches) upon a clean, sounded bedrock surface, the V_{s30} was calculated using the standard equation for average shear wave velocity calculation provided in the Ontario Building Code 2024 (OBC 2024), and as presented below, where in this case Layer 1 represents the overburden and Layer 2 depicts the bedrock. Since there will not be overburden present between the footings and bedrock, Layer 1 is not considered in this application.

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

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building is **1,834 m/s**. Therefore, as per OBC 2024 a **Site Designation $X_{1,834}$** is applicable for the proposed for the design of the proposed structure. The soil 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.

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 pipes should be connected to a sump pump located within the lowest basement level. It is recommended that the layout be provided by Paterson once the basement column layout is known. The spacing may be subject to change based on groundwater conditions encountered at the time of construction and as reviewed by Paterson field personnel.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. Where the soil is to be retained, there are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case. Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to latest revision of the Ontario Building Code. 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 latest revision of the Ontario Building Code.

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 unconfined compressive strength of shale bedrock ranges between 40 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system.

A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183** and **0.00009**, 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 4 – 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) - Fair Quality Shale Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale bedrock	40 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 5 – 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	1.2	3.3	4.5	250
	2.3	4.0	6.3	500
	4.4	4.7	9.0	1,000
	9.1	5.7	14.8	2,000
125	1.0	3.5	4.5	250
	1.5	4.6	6.1	500
	2.9	5.5	8.4	1,000
	5.5	6.2	11.7	2,000

5.8 Pavement Design

The recommended pavement structures for the subject site are shown in Table 6, Table 7 and Table 8.

Table 6 – 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 7 – Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas	
Thickness (mm)	**Material Description**
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 8 – Recommended Rigid Pavement Structure – Lower Parking Level	
Thickness (mm)	**Material Description**
Specified by Others	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
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.	

To control cracking due to shrinking of the concrete slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete slab. The control joints are generally recommended to be spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete slab.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) may 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 water, as this could impact the service life of the pavement structure.

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.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Watertight Foundation Waterproofing

It is recommended that a water-tight foundation waterproofing system be implemented for the proposed buildings foundation walls. This is recommended to mitigate potential long-term dewatering of the clay deposit that may occur if a conventional foundation drainage system with associated drainage at the footing level is considered. The bedrock and associated mud slabs will provide sufficient resistance to dewatering of the clay deposit such that additional measures are not required throughout the base of the excavation, however, a sub-slab drainage system is recommended to be provided throughout the basement level.

It is expected that this system will be constructed in a blind-cast manner and against a wood-lagging and similarly textured surface. As such, contractors should provide manufacturer details for waterproofing at transition zones between lagging and soldier piles and tieback locations.

It is expected that bedrock removal undertaken below the shoring system will result in an unsuitable substrate for the placement of waterproofing membranes. Provisions should be carried to grind the vertical bedrock face to provide a smoother surface than would be accomplished by solely undertaking hoe-ramming and/or blasting methods. The bedrock surface is recommended to be covered with either a sprayed elastomeric membrane or shotcrete to seal the bedrock sidewall from exposure to air and dewatering which would affect the expansive shale located throughout the subject site. If shotcrete is considered for this purpose, it should be troweled to provide a suitably smooth surface to be an appropriate waterproofing membrane substrate. The installer and manufacturer of an elastomeric membrane should review and advise if that product may be used to seal the bedrock from water intrusion and as the foundation waterproofing membrane, or if an additional product would be required for that purpose.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 100 to 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock. It is recommended to consider the implementation of a 150 mm diameter pipe, however, Paterson may review and advise on the potential to reduce the diameter to 100 mm once the excavation base has been attained and may be reviewed for potentially higher infiltration rates.

The underfloor drainage pipe should be placed in several rows spanning each direction of the basement floor and connected to a perimeter drainage pipe placed along the face of the perimeter footing. The spacing of the underfloor drainage system should be provided by Paterson once the foundation and column layout and sump system location(s) have been finalized and during the design phase of the project (i.e., prior to tender). The underfloor drainage pipe may be placed flat and level and is not required to be designed or constructed with a pitch provided a gravity connection to an associated sump pit is provided. It is recommended that Paterson reviews the installation of the proposed underfloor drainage system.

The invert of the pipe is recommended to be no deeper than 150 mm above the underside of the proposed footings to ensure all footings remain in a permanent submerged condition to mitigate long-term dewatering of the underlying shale bearing medium.

Elevator Shaft and Additional Sub-Floor Structures Waterproofing

Elevator shafts located below the underslab drainage system should be provided full-depth positive-side waterproofing and provided with a PVC waterstop at the shaft wall and footing interface. Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts, pools, cisterns and other water-tight structures that will be located within the lowest basement level.

Foundation Backfilling

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. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. This material should be placed in 300 to 400 mm thick loose lifts and compacted with suitably-sized compaction equipment to a minimum of 95% of the materials SPMDD, or, as considered suitable by staff experienced in visually reviewing the compaction of soil fill.

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.

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

6.3 Excavation Side Slopes

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. The flatter slope is required for excavation below groundwater level. 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.

Consideration may also be given to providing a near-vertical 1 m high wedge at the bottom of the temporary side-slope in overburden materials and directly above the soil-bedrock interface. 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.

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.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring 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 potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system, interlocking steel sheet piling or interlocking and augered cast-in-place caissons. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 9 - 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

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60-to-90-degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended. Further, the bonded portion of the rock anchor should be fully extended below the sound bedrock surface.

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

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

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

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

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Underpinning

Founding conditions of adjacent structures bordering the site should be assessed and underpinning requirements should be evaluated at the time of construction. As a general precaution, excavation details and recommendations provided for the implementation of the shoring system along the portion of the excavation supporting the existing parking garage should be prepared to minimize the potential for soils to slough and reduce overall subsoil support to the existing structure. Periodic review of the excavation plan throughout that portion of the building excavation should be carried out by the shoring designers team to ensure all efforts are taken to minimize disturbance to the subject structure and in accordance with their designs requirements.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should be increased to a minimum thickness of 300 mm where located over a bedrock subgrade. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material's SPMDD. It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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 minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps for the majority of the excavation due to the presence of the silty clay deposit. Higher volumes and influx may be encountered once the excavation advances below the silty clay deposit and into the underlying glacial till deposit and bedrock formation. 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.

Based on the results of site-specific hydraulic conductivity testing, it is recommended that a temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) be attained for this project since more than 400,000 L/day of ground and/or surface water is anticipated to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which encounters the building's perimeter water infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 10,000 L/day with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighboring Structures

It is understood that five levels of underground parking extending below the bedrock surface are planned for the proposed building. Since the building will be provided with a watertight foundation and located in mostly low permeability soils, the proposed development will not negatively impact the neighboring structures in the long-term. Therefore, no issues are expected with respect to long-term groundwater lowering that would cause damage to adjacent structures surrounding the proposed building.

During the construction phase, dewatering efforts are anticipated to be required to control water influx for excavations undertaken below the bedrock surface. Given the permeability of the underlying bedrock formation and overlying moisture sensitive overburden, as a precautionary measure, it is recommended that a pre-construction and post-construction survey and associated settlement monitoring be carried out for buildings located within 50 m of the proposed excavation to ensure there are no negative impacts to existing adjacent structures. However, potential impacts are anticipated to negligible at this time.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. This should be detailed and reviewed by the excavation and/or shoring designer and Paterson at the time of detailed design and review.

Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable. In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means.

The base of the excavations (especially where buildings will be founded upon soil, such as the southern building) should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the buildings and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice is present within the blast rock or other 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 an aggressive to severe corrosive environment.

6.8 Landscaping Considerations for Proposed Structure

The proposed building will be supported by footings founded directly upon the bedrock surface and will be provided a watertight foundation. Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to be experienced by the proposed structure.

Since the proposed structure is not anticipated to be founded upon clay soils sensitive to moisture depletion, new and existing trees planted adjacent to the proposed structure are not 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 the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- Review of architectural plans pertaining to the buildings foundation 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 the geotechnical consultant. 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, mud slabs, 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 LS GP Inc. 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.

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

Drew Petahtegoose, P.Eng.

Report Distribution:

- ❑ LS GP Inc. (1 email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

COORD. SYS.: MTM ZONE 9 **EASTING:** 373450.84 **NORTHING:** 5027830.72 **ELEVATION:** 82.27

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 26, 2025

FILE NO. : PG4440

HOLE NO. : BH 1-25

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE						
ASPHALT 0.05m [82.22m]	-	AU 1				82
FILL: Crushed stone and topsoil 0.41m [81.86m]						
Hard to stiff, brown **SILTY CLAY**	1	SS 2	100	3-6-10-13 16		81
	2	SS 3	100	3-5-7-8 12		80
		SS 4	100	2-3-6-5 9		
	3	SS 5	100	3-1-2-2 3		79
- Firm and grey by 4.1 m depth	4	SS 6	100	1-1-1-1 2		78
	5	SS 7	100	P		77
		SS 8	100	P		
	6	SS 9	100	P		76
	7	SS 10	100	P		75
	8	SS 11	100	P		74
		SS 12	100	P		
	9	SS 13	100	P		73
	10	SS 14	100	P		72
	11	SS 15	100	P		

MONITORING WELL CONSTRUCTION: 6.56 m 2025-03-10

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373450.84 **NORTHING:** 5027830.72 **ELEVATION:** 82.27

PROJECT: Proposed High-Rise Building

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DATE: February 26, 2025

FILE NO. : PG4440

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	ELEVATION (m)
12.65m [69.62m]	11–12.65	SS 15			71
		SS 16	100	1-4-1-1 5	70
		SS 17	100	P	
GLACIAL TILL: Grey silty clay, some sand with gravel, cobbles, boulders	12.65–14.78	SS 18	25	1-1-1-5 2	69
14.78m [67.49m]		SS 19	0	1-2-6-5 8	68
GLACIAL TILL: Dense, grey sand, silt with gravel, cobbles, boulders, trace clay	14.78–15.44	SS 20	102	20-50-/-/ 50/0.05	67
15.44m [66.83m]					
BEDROCK: Good quality shale bedrock	15.44–19.84	RC 1		RQD 80	66
		RC 2		RQD 78	65
		RC 3		RQD 81	64, 63
19.84m [62.43m]		RC 4		RQD 100	
End of Borehole	20–22				62, 61

(GWL at 6.56 m depth - March 10, 2025)

Chart headings: PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa); ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80; WATER CONTENT (%); STRATA PLOT; MONITORING WELL CONSTRUCTION: 18.24m, 19.76m

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P:/AutoCAD Drawings/Test Hole Data Files/PG44xx/PG4440/data.sqlite 2025-03-21, 12:15 Paterson_Template AA

COORD. SYS.: MTM ZONE 9 **EASTING:** 373434.38 **NORTHING:** 5027827.05 **ELEVATION:** 82.47

PROJECT: Proposed High-Rise Building
ADVANCED BY: CME-55 Low Clearance Drill
REMARKS:
DATE: February 27, 2025

FILE NO. : PG4440
HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
OVERBURDEN		-							82
		1							81
1.52m [80.95m]									
Dynamic Cone Penetration Test commenced at 1.52 m depth		2							80
		3							79
		4							78
		5							77
		6						6.56 m ▼ 2025-03-10	76
		7							75
		8							74
		9							73
		10							72
		11							

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373434.38 **NORTHING:** 5027827.05 **ELEVATION:** 82.47

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
15.42m [67.05m]		11–15					100 (at 15.42 m)		71–67
End of Borehole Cone pushed from 1.52 to 15.42 m depth Practical refusal to DCPT at 15.42 m depth on inferred bedrock		16–22							66–61

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373450.70 **NORTHING:** 5027839.67 **ELEVATION:** 82.45

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
OVERBURDEN		0							82
		1							81
1.52m [80.93m]									
Dynamic Cone Penetration Test commenced at 1.52 m depth		2							80
		3							79
		4							78
		5							77
		6							76
		7							75
		8							74
		9							73
		10							72
		11					■ 11		

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373450.70 **NORTHING:** 5027839.67 **ELEVATION:** 82.45

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

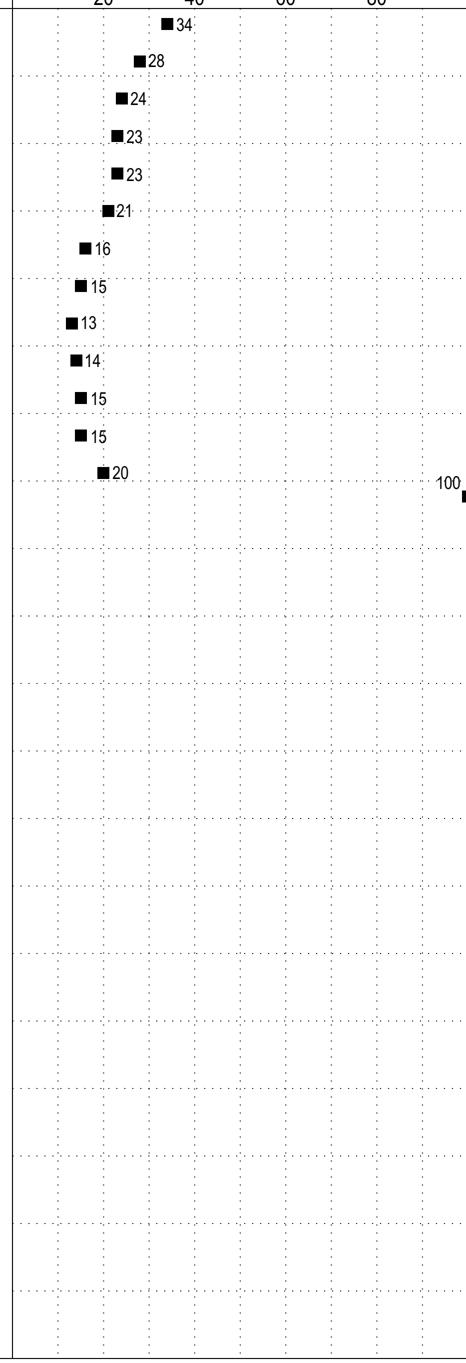
REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) / PL (%) WATER CONTENT (%) LL (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
15.01m [67.44m]		11–15							71–68
End of Borehole Cone pushed from 1.52 to 11.46 m depth Practical refusal to DCPT at 15.01 m depth on inferred bedrock		15–22							67–61



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SOIL PROFILE AND TEST DATA
Geotechnical Investigation
2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373461.42 **NORTHING:** 5027822.74 **ELEVATION:** 81.95

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
OVERBURDEN									
1.52m [80.43m]		1							81
Dynamic Cone Penetration Test commenced at 1.52 m depth		2							80
		3							79
		4							78
		5							77
		6							76
		7							75
		8							74
		9							73
		10							72
		11					■17		71

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373461.42 **NORTHING:** 5027822.74 **ELEVATION:** 81.95

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

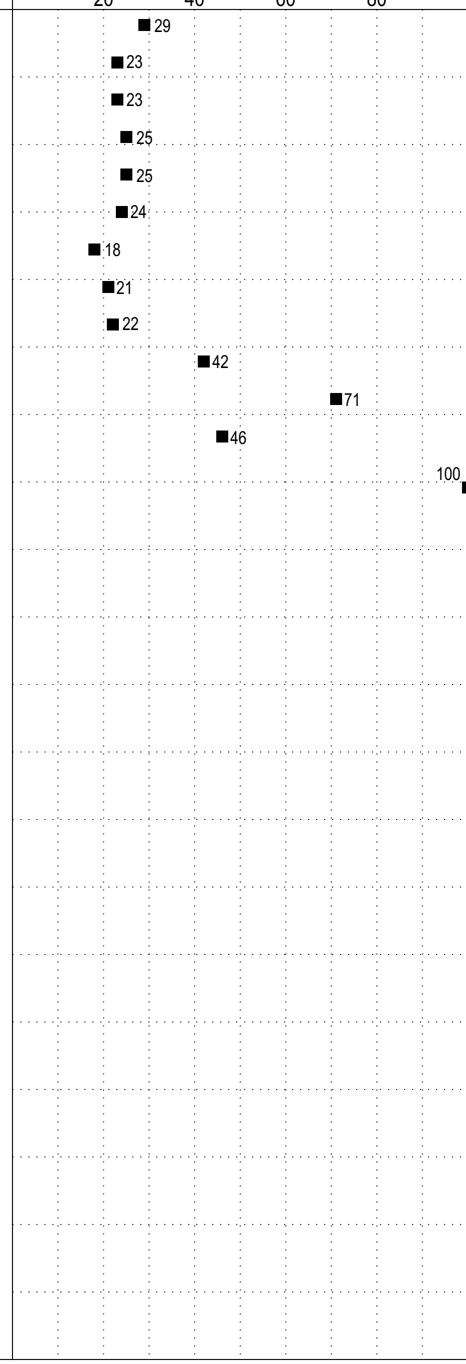
REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) / PL (%) WATER CONTENT (%) LL (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
15.16m [66.79m]		11 – 15.16					See plot		70 – 67
End of Borehole Cone pushed from 1.52 to 10.72 m depth Practical refusal to DCPT at 15.16 m depth on inferred bedrock		15.16 – 22							67 – 60



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SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation
2145 Walkey Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 373457.45 **NORTHING:** 5027834.68 **ELEVATION:** 82.22

PROJECT: Proposed High-Rise Building
ADVANCED BY: CME-55 Low Clearance Drill
REMARKS:
DATE: February 27, 2025

FILE NO. : PG4440
HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
OVERBURDEN		0 – 1.52							82 – 80.70
1.52m [80.70m]									
Dynamic Cone Penetration Test commenced at 1.52 m depth		1.52 – 11					■17 (at ~11 m)		80.70 – 71

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11
Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373457.45 **NORTHING:** 5027834.68 **ELEVATION:** 82.22

PROJECT: Proposed High-Rise Building

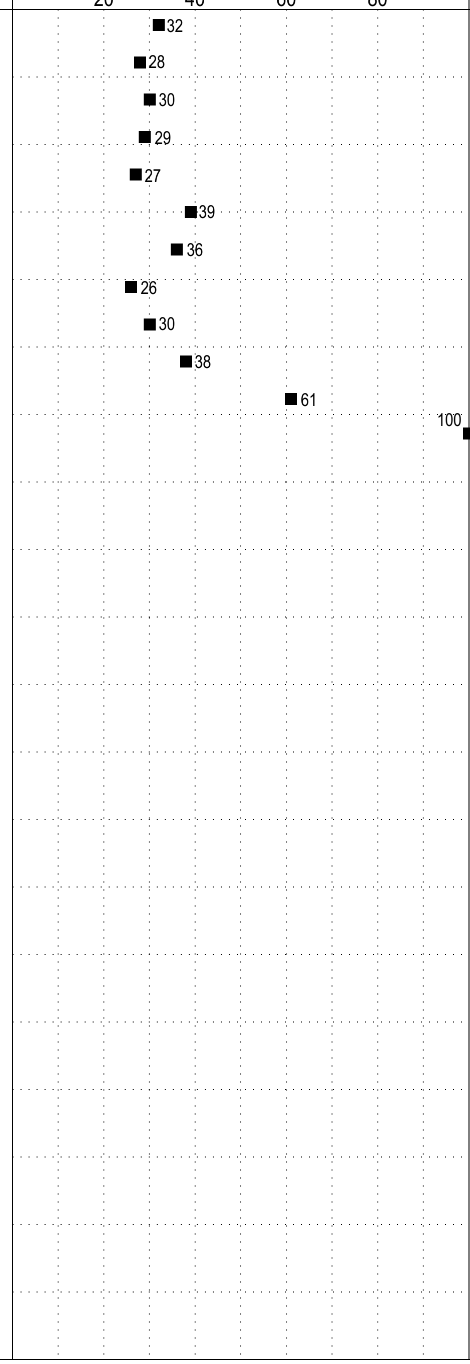
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
14.58m [67.64m]		11–22							71–61
End of Borehole Cone pushed from 1.52 to 10.77 m depth Practical refusal to DCPT at 14.58 m depth on inferred bedrock									

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373481.11 **NORTHING:** 5027846.08 **ELEVATION:** 79.84

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)	ELEVATION (m)
GROUND SURFACE							
ASPHALT 0.08m [79.76m]		AU 1					
FILL: Brown silty clay, with sand and gravel 0.43m [79.41m]							
Hard to stiff, brown **SILTY CLAY**	1	SS 2	83	8-8-7-4 15			79
	2	SS 3	100	4-2-2-2 4			78
		SS 4	100	1-2-1-2 3			77
- Firm and grey by 3.0 m depth	3	SS 5	100	2-1-2-1 3			76
	4	SS 6	100	1-1-1-1 2			75
5.18m [74.66m]	5	SS 7	100	1-1-1-1 2			
OVERBURDEN	6						74
Dynamic Cone Penetration Test commenced at 5.18 m depth	7						73
	8						72
	9						71
	10						70
10.67m [69.17m]							
GLACIAL TILL: Grey silty clay with sand, gravel,	11	SS 8	8	P		11	69

Chart axes: PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

STRATA PLOT

MONITORING WELL CONSTRUCTION: 4.99 m 2025-03-10; 10.14m

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SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 373481.11 **NORTHING:** 5027846.08 **ELEVATION:** 79.84

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

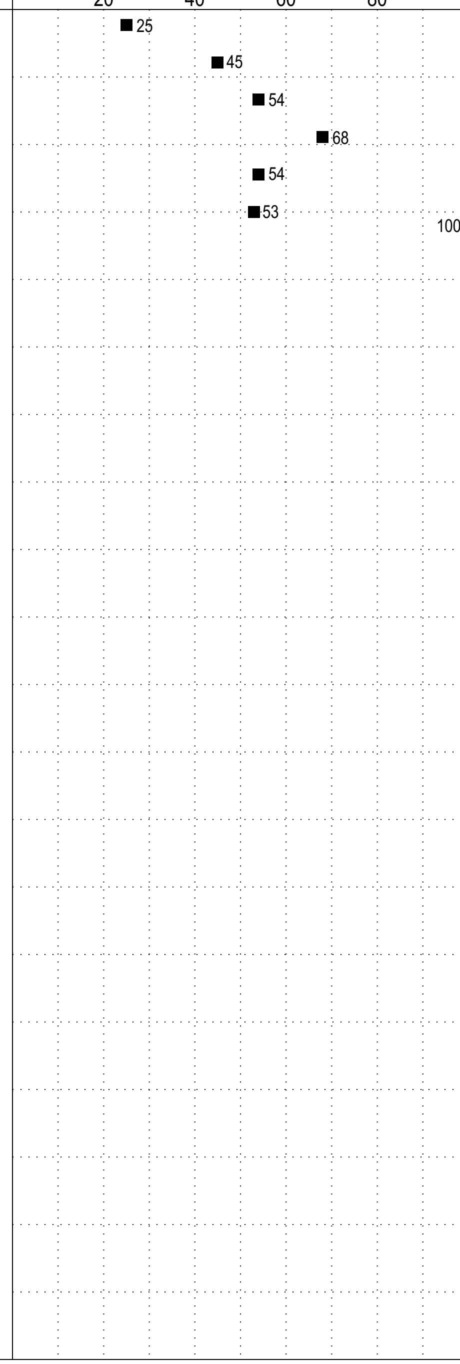
HOLE NO. : BH 6-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
cobbles, boulders 11.35m [68.49m]	11	SS 8				
GLACIAL TILL: Dense, grey sand, silt with gravel, cobbles, boulders, trace clay		SS 9	58	10-13-18-20 31		68
	12	SS 10	33	11-13-22-28 35		67
12.95m [66.89m]						
End of Borehole Cone pushed from 1.52 to 10.74 m depth Practical refusal to DCPT at 12.95 m depth on inferred bedrock (GWL at 4.99 m depth - March 10, 2025)	13					66
	14					65
	15					64
	16					63
	17					62
	18					61
	19					60
	20					59
	21					58
	22					

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80



MONITORING WELL CONSTRUCTION

11.68m

P:/AutoCAD Drawings/Test Hole Data Files/PG44xx/PG4440/data.sqlite 2025-03-21, 12:15 Paterson_Template AA

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 373467.76 **NORTHING:** 5027829.11 **ELEVATION:** 81.92

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.05m [81.88m]									
OVERBURDEN									
Dynamic Cone Penetration Test commenced at 0.05 m depth		1							81
		2							80
		3							79
		4							78
		5							77
		6							76
		7							75
		8					7.91 m ▼ 2025-03-10		74
		9							73
		10							72
		11							71

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SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation

2145 Walkey Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 373467.76 **NORTHING:** 5027829.11 **ELEVATION:** 81.92

PROJECT: Proposed High-Rise Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 27, 2025

FILE NO. : PG4440

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	ELEVATION (m)
11.43m [70.50m]	11				
Firm, grey **SILTY CLAY** 11.73m [70.19m]		SS 1	100	1-1-3-2 4	70
GLACIAL TILL: Compact, grey silty clay with sand, gravel, cobbles, boulders	12	SS 2	100	8-2-2-1 4	69
	13	SS 3	100	1-2-10-8 12	68
14.33m [67.59m]	14	SS 4	75	3-8-17-30 25	
End of Borehole (GWL at 7.91 m depth - March 10, 2025)	15–22				67–60

Column headings: STRATA PLOT; WATER CONTENT (%); ■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80; MONITORING WELL CONSTRUCTION

Monitoring well screen: 12.20m – 13.71m

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SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation
2145 Walkey Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 373446.88 **NORTHING:** 5027882.93 **ELEVATION:** 79.82

PROJECT: Proposed High-Rise Building
ADVANCED BY: CME-55 Low Clearance Drill
REMARKS:
DATE: February 28, 2025

FILE NO. : PG4440
HOLE NO. : BH 8-25

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80
△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80
PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION (GROUND SURFACE)	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
TOPSOIL: with organics — 0.08m [79.74m]		AU 1				
FILL: Brown silty clay, trace gravel and topsoil - Some sand at 0.91 m depth	1	SS 2	50	6-5-9-12 14		79
— 1.68m [78.14m]						
Hard to very stiff, brown **SILTY CLAY**	2	SS 3	83	3-5-6-9 11		78
		SS 4	100	4-6-9-8 15		77
	3	SS 5	100	2-2-2-3 4		76
	4	SS 6	100	2-3-3-3 6		
- Grey and stiff by 4.5 m depth — 5.18m [74.64m]	5	SS 7	100	1-2-2-2 4		75
End of Borehole	6					74
	7					73
	8					72
	9					71
	10					70
	11					69

STRATA PLOT | PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Residential Development
Walkley Road at Halifax Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG4440

REMARKS

HOLE NO. BH 1-20

BORINGS BY CME-55 Low Clearance Drill

DATE December 7, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	79.89		
						1	78.89		
						2	77.89		
						3	76.89		
						4	75.89		
						5	74.89		
OVERBURDEN						6	73.89		
						7	72.89		
						8	71.89		
						9	70.89		
						10	69.89		
						11	68.89		
						12	67.89		
13.62						13	66.89		
BEDROCK: Fair quality, black shale		RC	1	100	53	14	65.89		
		RC	2	100	58	15	64.89		
		RC	3	100	43	16	63.89		
		RC	4	100	68	17	62.89		
17.98									
End of Borehole									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Residential Development
Walkley Road at Halifax Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG4440

REMARKS

HOLE NO. BH 2-20

BORINGS BY CME-55 Low Clearance Drill **DATE** December 8, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.79
						1	77.79
						2	76.79
						3	75.79
						4	74.79
						5	73.79
OVERBURDEN						6	72.79
						7	71.79
						8	70.79
						9	69.79
						10	68.79
						11	67.79
						12	66.79
12.80		RC	1	89	44	13	65.79
BEDROCK: Poor to good quality, black shale		RC	2	100	83	14	64.79
14.96 End of Borehole							

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Residential Development
Walkley Road at Halifax Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG4440

REMARKS

HOLE NO. BH 3-20

BORINGS BY CME-55 Low Clearance Drill

DATE December 8, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	79.87
						1	78.87
						2	77.87
						3	76.87
						4	75.87
						5	74.87
OVERBURDEN						6	73.87
						7	72.87
						8	71.87
						9	70.87
						10	69.87
						11	68.87
12.57						12	67.87
		RC	1	20		13	66.87
BEDROCK: Fair quality, black shale		RC	2	91	68	14	65.87
						15	64.87
16.74		RC	3	100	72	16	63.87
End of Borehole							

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Residential Development
Walkley Road at Halifax Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG4440

REMARKS

HOLE NO. BH 4-20

BORINGS BY CME-55 Low Clearance Drill **DATE** December 9, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	80.95
						1	79.95
						2	78.95
						3	77.95
						4	76.95
						5	75.95
						6	74.95
OVERBURDEN						7	73.95
						8	72.95
						9	71.95
						10	70.95
						11	69.95
						12	68.95
						13	67.95
14.02						14	66.95
		RC	1	59	50	15	65.95
BEDROCK: Poor quality, black shale		RC	2	93	48	16	64.95
		RC	3	100	38	17	63.95
17.75 End of Borehole							

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 12, 2018

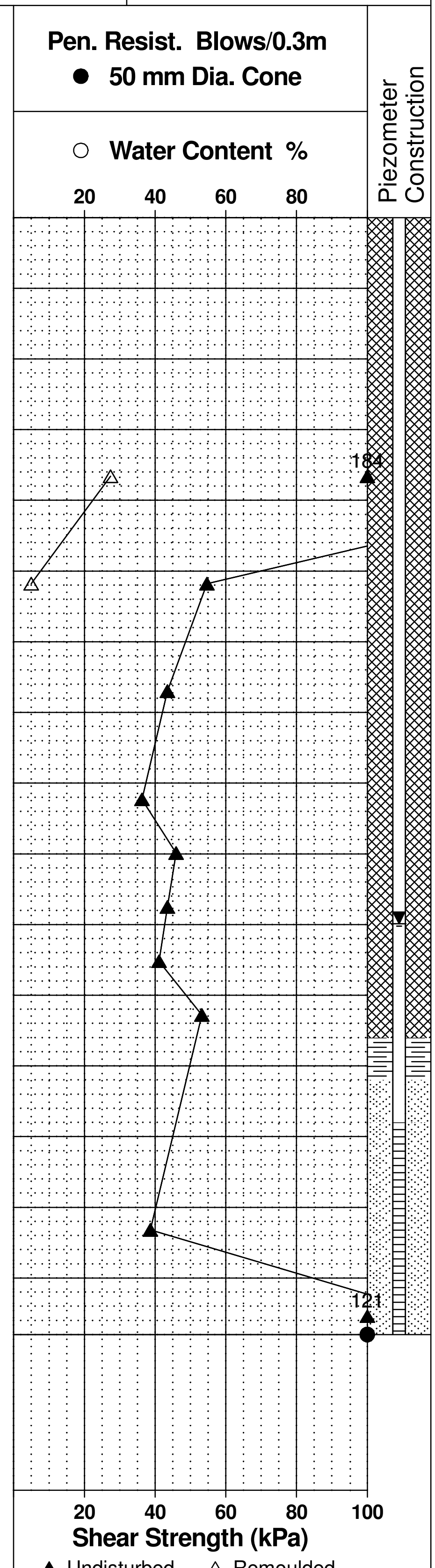
FILE NO. PG4440

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	82.22
Asphaltic concrete 0.10		AU	1				
FILL: Crushed stone with silt and sand 0.38						1	81.22
		SS	2	100	12	2	80.22
Very stiff, brown **SILTY CLAY,** some sand						3	79.22
						4	78.22
- stiff to firm and grey by 4.6m depth						5	77.22
						6	76.22
						7	75.22
						8	74.22
						9	73.22
						10	72.22
						11	71.22
						12	70.22
		SS	3	100	4	13	69.22
						14	68.22
- trace gravel by 15.5m depth 15.80						15	67.22

Dynamic Cone Penetration Test commenced at 15.8m depth. Practical DCPT refusal at 15.80m depth.

(GWL @ 9.99m - March 22, 2018)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 12, 2018

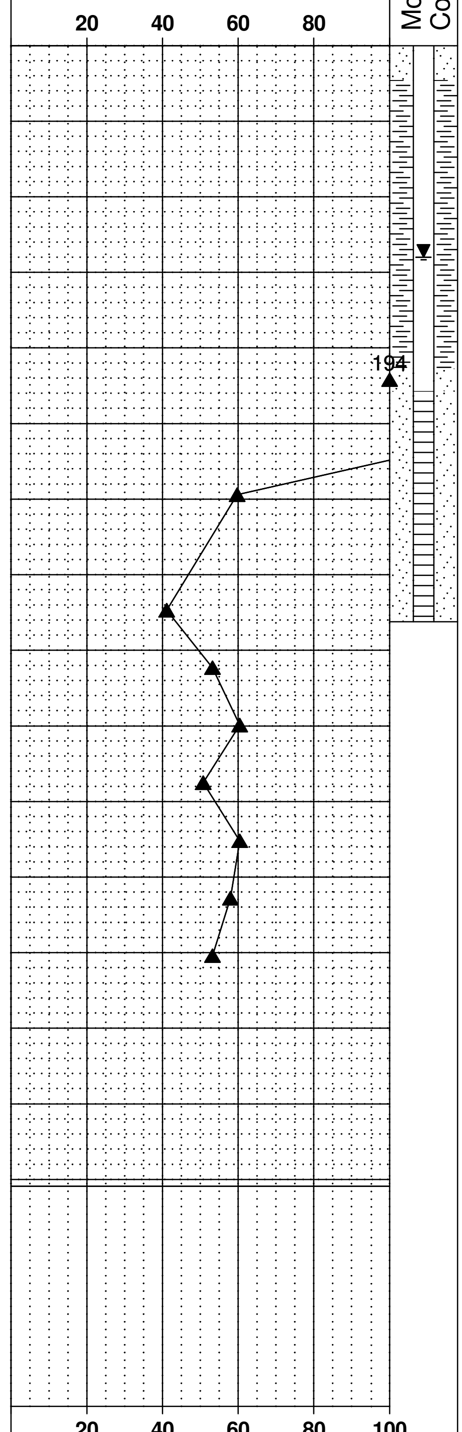
FILE NO. PG4440

HOLE NO. BH 5

SOIL DESCRIPTION	Depth of layer base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	82.47
Asphaltic concrete	0.08	AU	1				
FILL: Crushed stone with silt and sand	0.46	SS	2	50	14	1	81.47
Compact, brown **SAND** with silt, trace clay	1.37					2	80.47
Very stiff, brown **SILTY CLAY,** some sand - stiff and grey by 5.3m depth	12.95	SS	3	100	9	3	79.47
						4	78.47
						5	77.47
						6	76.47
						7	75.47
						8	74.47
						9	73.47
						10	72.47
						11	71.47
						12	70.47
GLACIAL TILL: Grey silty clay with some sand and gravel	15.09	SS	4	46	2	13	69.47
						14	68.47
		SS	5	100	1	15	67.47
End of Borehole (GWL @ 2.80m - March 22, 2018)							

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

REMARKS

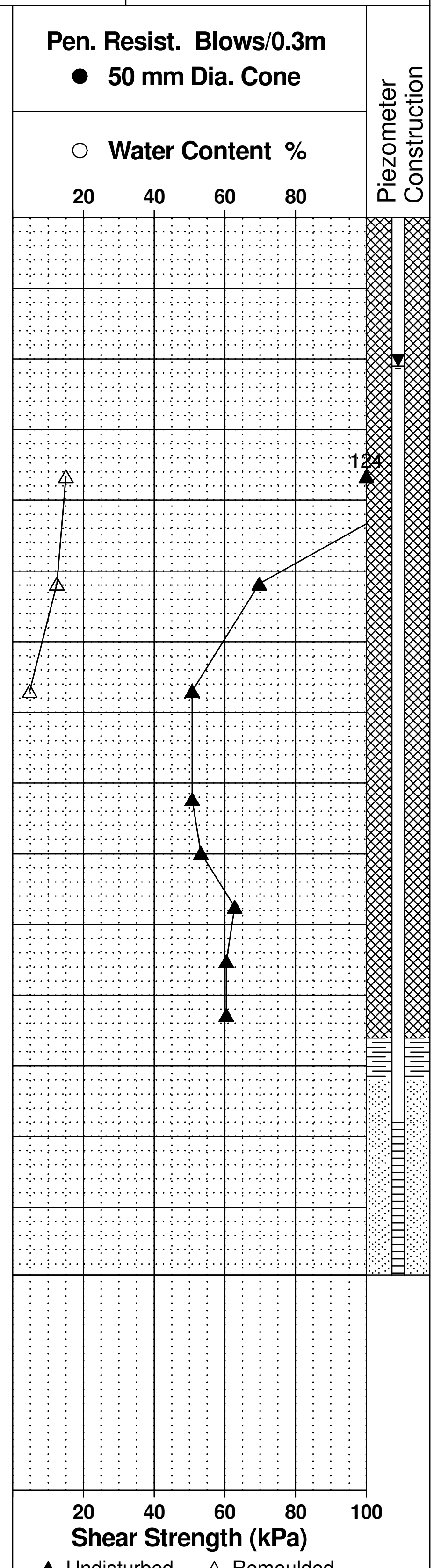
BORINGS BY CME 55 Power Auger

DATE March 12, 2018

FILE NO. PG4440

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.43
Asphaltic concrete 0.10		AU	1				
FILL: Crushed stone with silt and sand 0.51						1	80.43
		SS	2	100	12	2	79.43
						3	78.43
Very stiff, brown **SILTY CLAY,** some sand						4	77.43
- stiff and grey by 4.9m depth						5	76.43
						6	75.43
						7	74.43
						8	73.43
						9	72.43
						10	71.43
						11	70.43
12.19						12	69.43
GLACIAL TILL: Grey silty clay, some sand and gravel		SS	3	42	4	13	68.43
13.72							
GLACIAL TILL: Grey silty sand with gravel, trace clay		SS	4	71	46	14	67.43
14.96		SS	5	0	50+		
End of Borehole							
Practical refusal to augering at 14.96m depth							
(GWL @ 2.10m - March 22, 2018)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

REMARKS

BORINGS BY CME 55 Power Auger

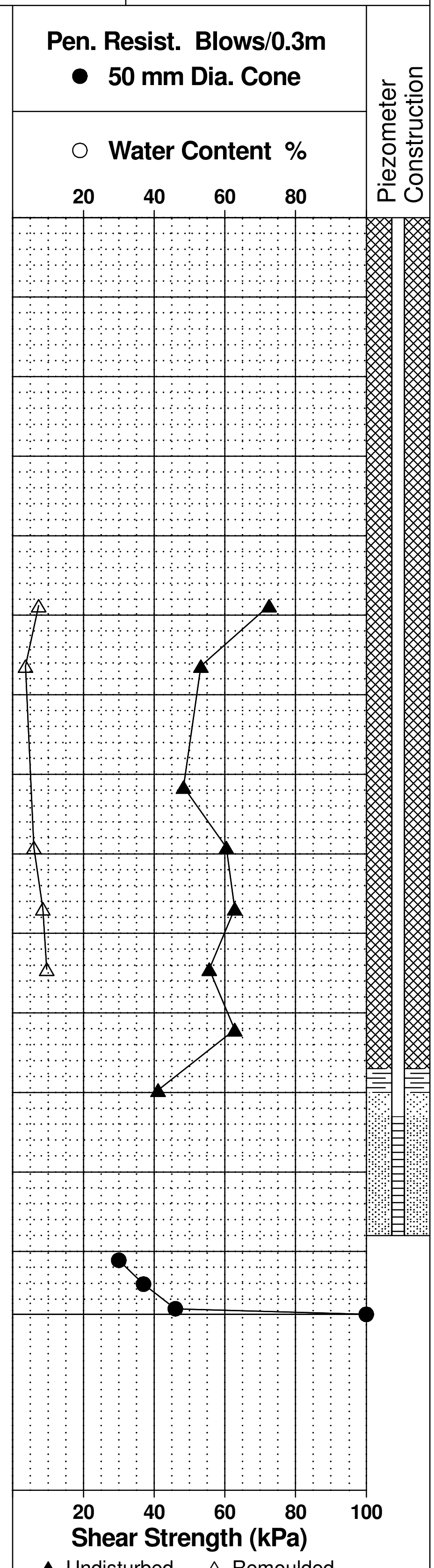
DATE December 11, 2018

FILE NO. PG4440

HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
OVERBURDEN	4.57				
Stiff to firm, grey **SILTY CLAY**	11.58	SS	1	38	W
GLACIAL TILL: Compact to dense, grey silty sand with gravel, cobbles and boulders	12.80	SS	2	62	26
		SS	3	58	37
Dynamic Cone Penetration Test commenced at 12.80m depth	13.79				
Inferred **GLACIAL TILL**					
End of Borehole					
Practical DCPT refusal at 13.79m depth					

DEPTH (m)	ELEV. (m)
0	80.66
1	79.66
2	78.66
3	77.66
4	76.66
5	75.66
6	74.66
7	73.66
8	72.66
9	71.66
10	70.66
11	69.66
12	68.66
13	67.66



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 11, 2018

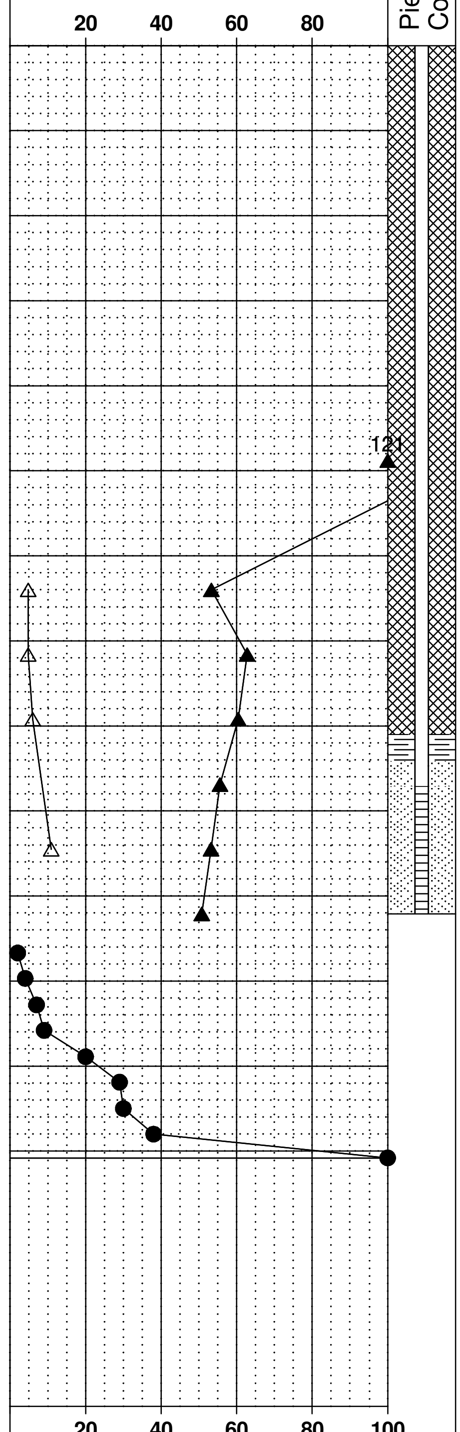
FILE NO. PG4440

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	79.88
						1	78.88
OVERBURDEN						2	77.88
						3	76.88
						4	75.88
4.57						5	74.88
						6	73.88
Very stiff to stiff, grey **SILTY CLAY**						7	72.88
						8	71.88
						9	70.88
10.21						10	69.88
Dynamic Cone Penetration Test commenced at 10.21m depth						11	68.88
						12	67.88
13.08						13	66.88
End of Borehole Practical DCPT refusal at 13.08m depth							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Walkley Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the south wall of proposed building. Geodetic elevation = 81.81m.

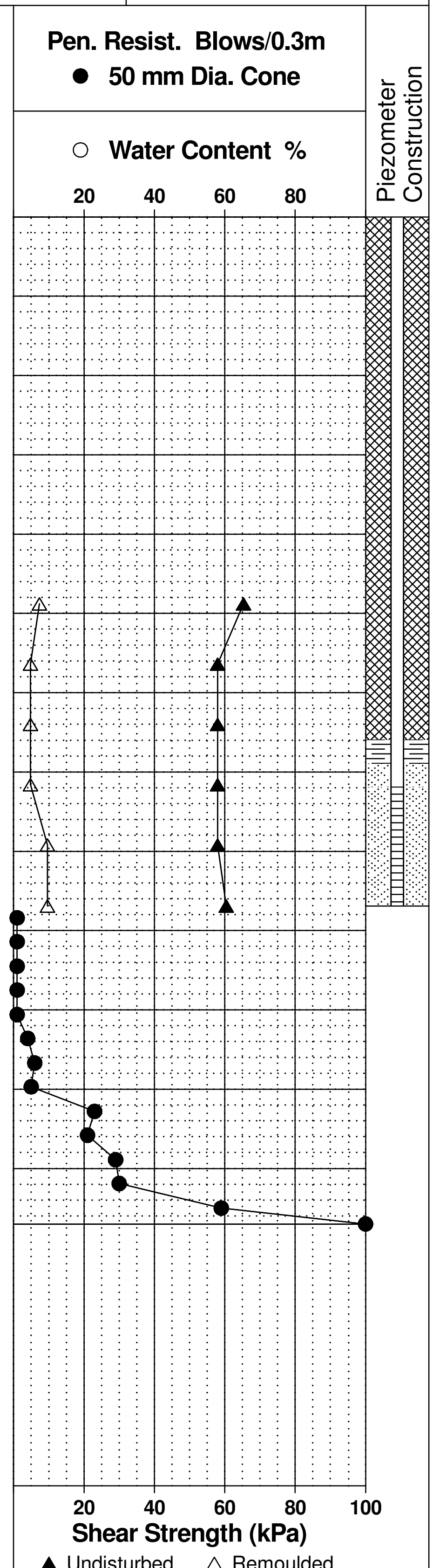
REMARKS

BORINGS BY CME 55 Power Auger **DATE** December 11, 2018

FILE NO. PG4440

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	79.65
						1	78.65
OVERBURDEN						2	77.65
						3	76.65
4.57						4	75.65
						5	74.65
						6	73.65
Stiff, grey **SILTY CLAY**						7	72.65
8.69						8	71.65
Dynamic Cone Penetration Test commenced at 8.69m depth						9	70.65
						10	69.65
						11	68.65
12.70						12	67.65
End of Borehole Practical DCPT refusal at 12.70m depth							



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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

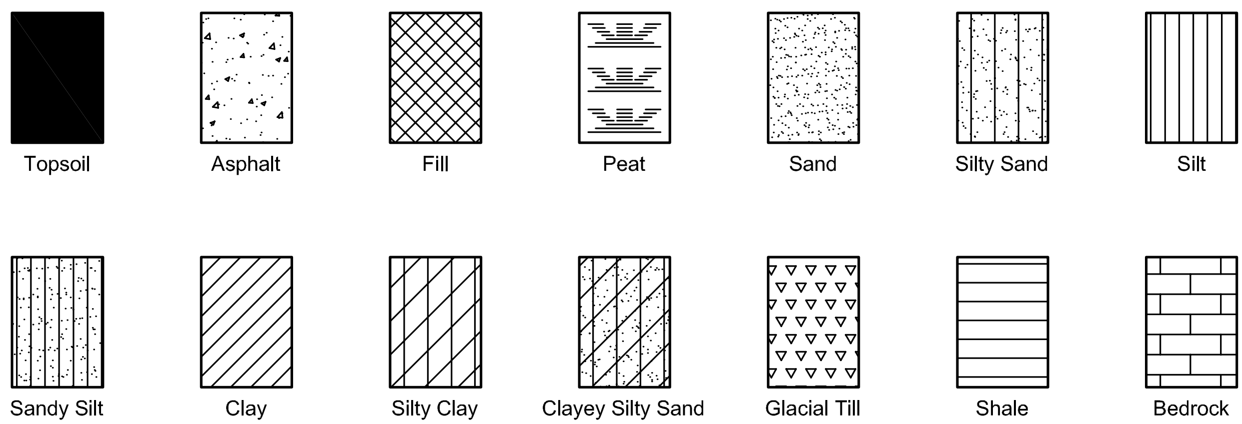
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

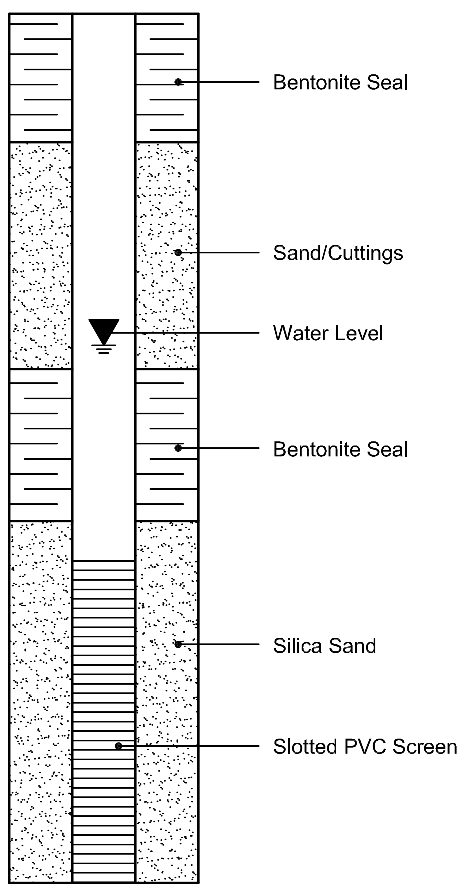
SYMBOLS AND TERMS (continued)

STRATA PLOT

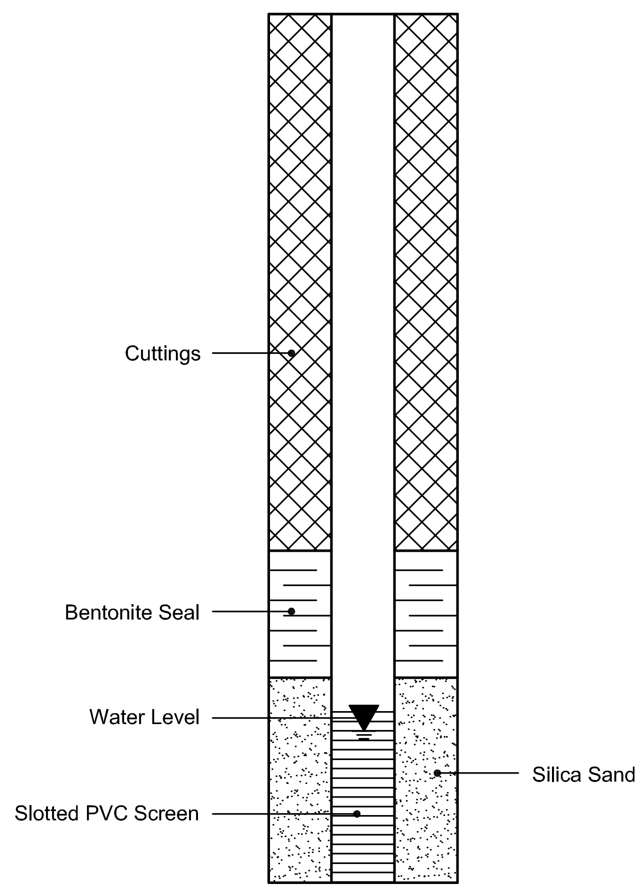


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1811279

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 23604

Report Date: 21-Mar-2018

Order Date: 14-Mar-2018

Project Description: PG4440

	Client ID:	BH4 PSV1	-	-	-
	Sample Date:	13-Mar-18	-	-	-
	Sample ID:	1811279-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.56	-	-	-
Resistivity	0.10 Ohm.m	26.3	-	-	-
Anions					
Chloride	5 ug/g dry	116	-	-	-
Sulphate	5 ug/g dry	82	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4440-1 – TEST HOLE LOCATION PLAN

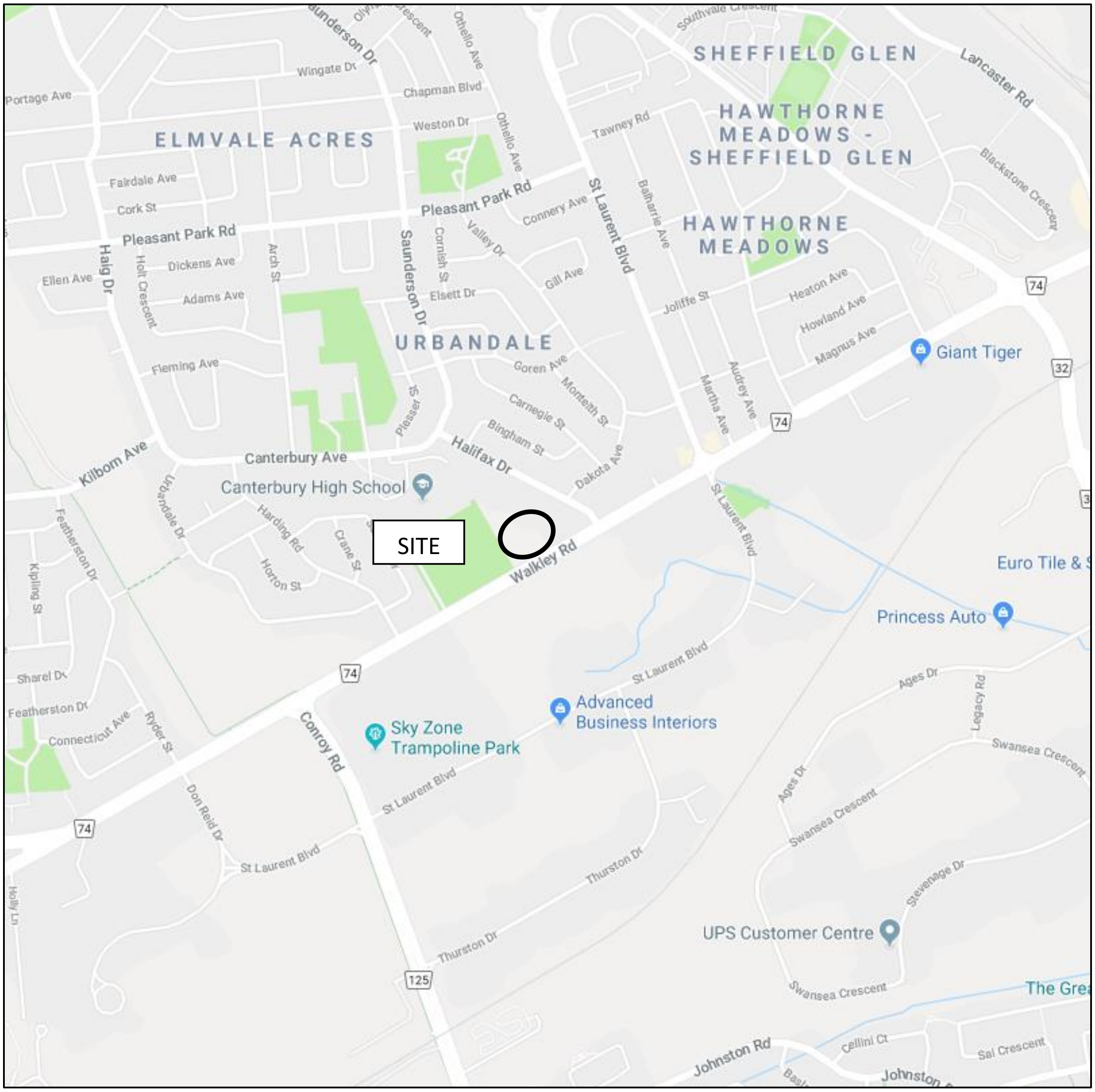


FIGURE 1

KEY PLAN

PATERSON GROUP

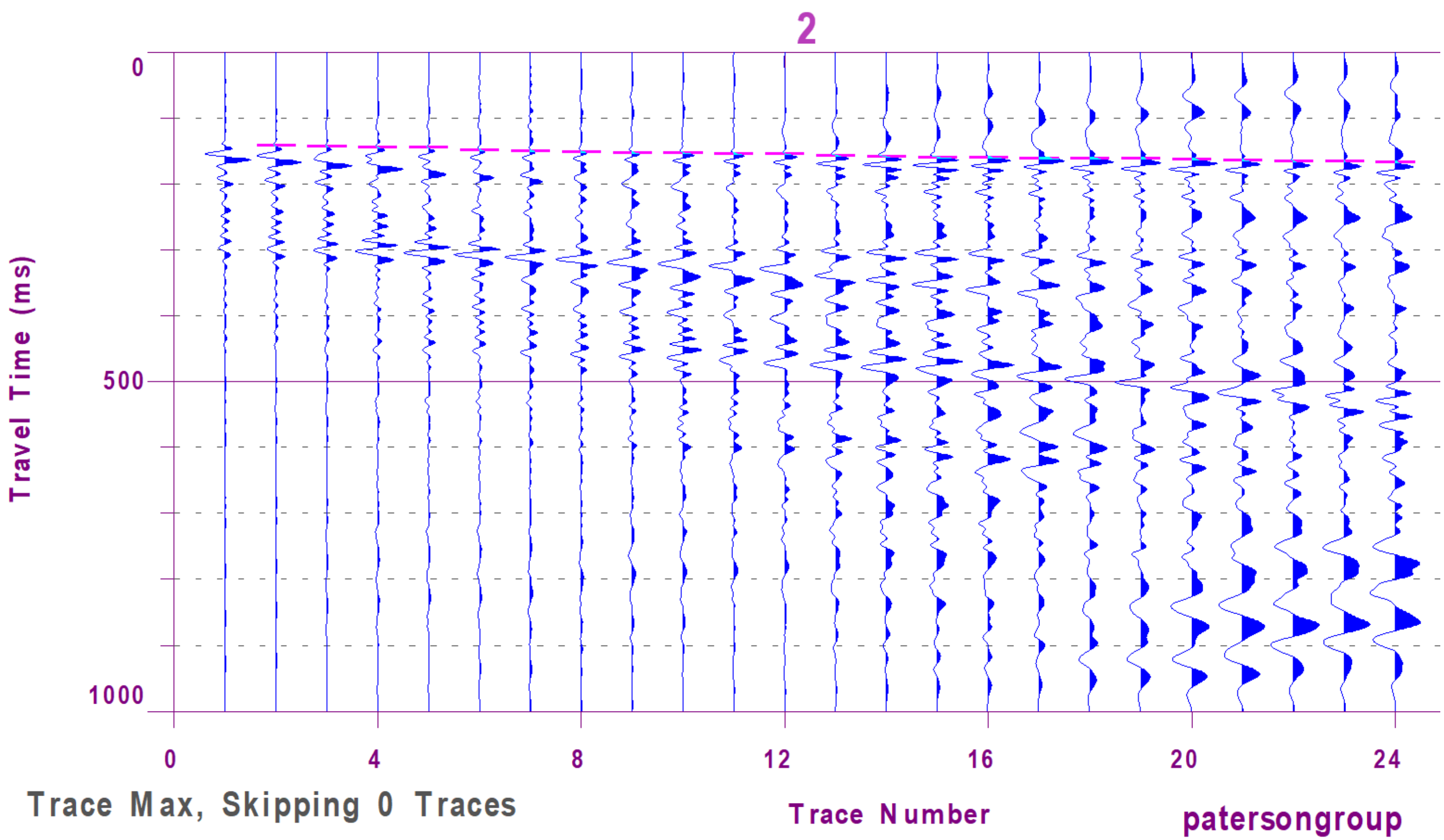


Figure 2 – Shear Wave Velocity Profile at Shot Location -18 m

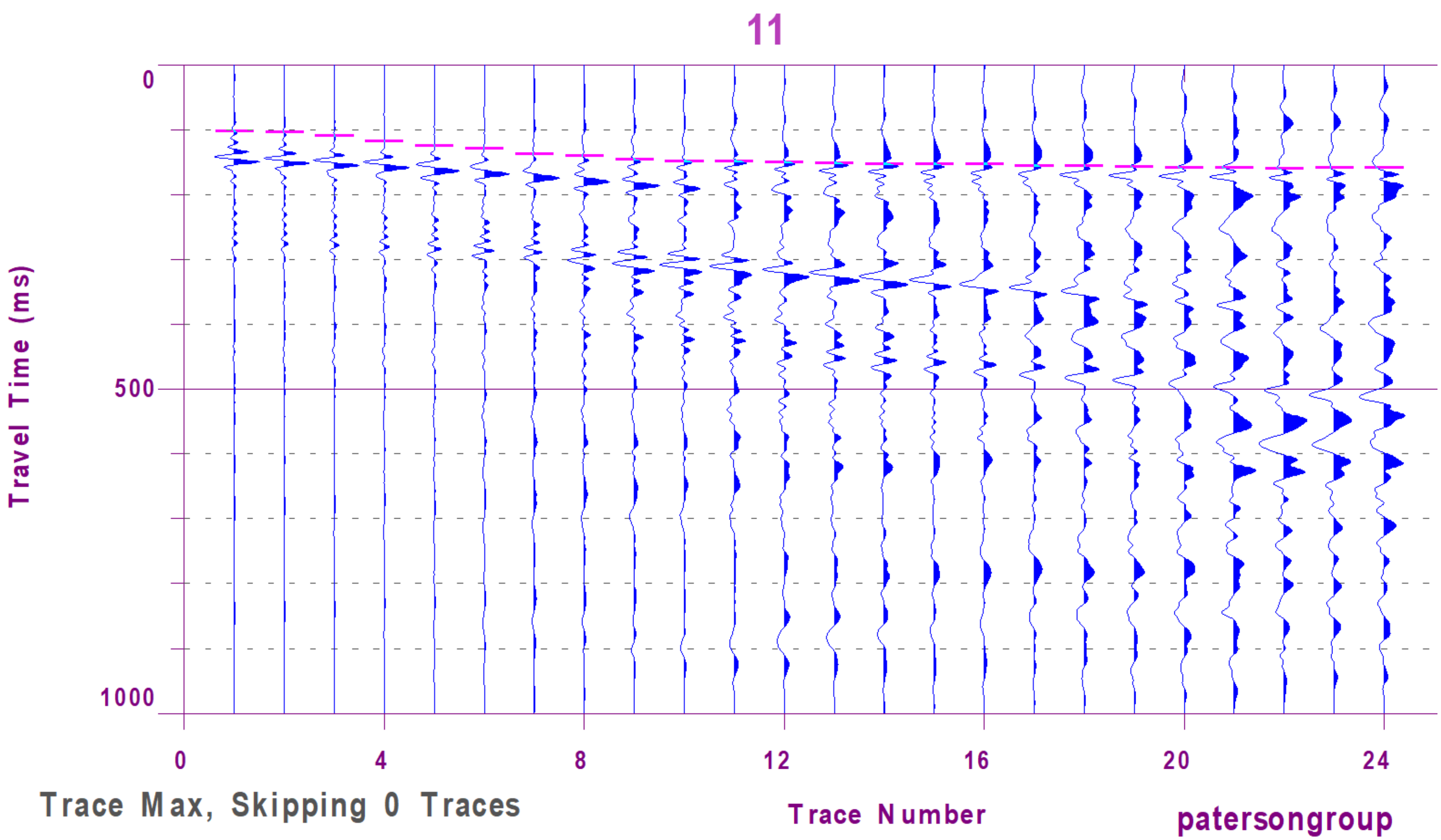
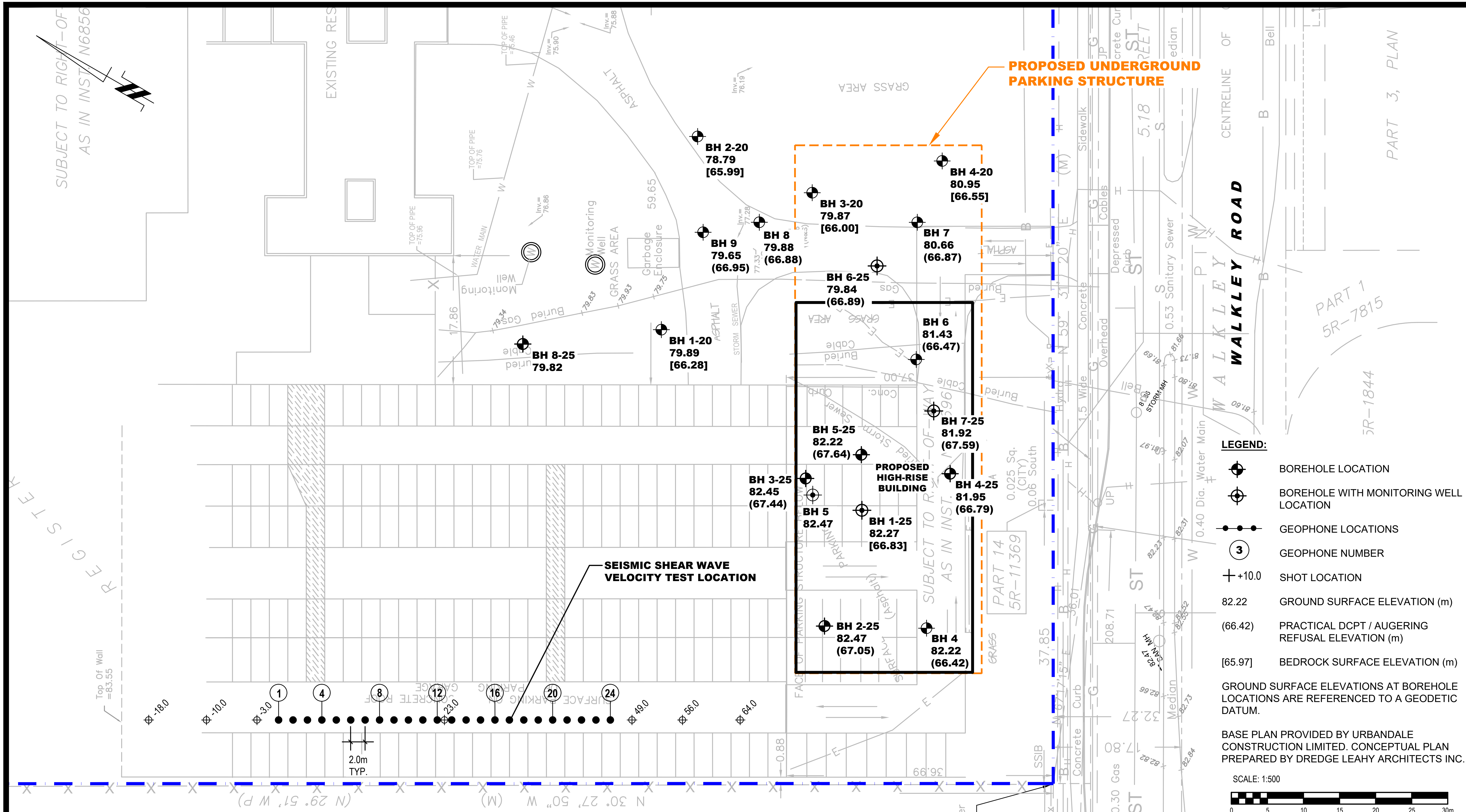


Figure 3 – Shear Wave Velocity Profile at Shot Location -10 m



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
5	BH 1-25 TO 8-25 ADDED, UPDATED CONCEPTUAL SITE PLAN	06/03/2025	NFRV
4	UPDATED CONCEPTUAL SITE PLAN	08/10/2024	DP
3	UPDATED CONCEPTUAL SITE PLAN	18/01/2021	VD
2	BH 1-20 TO BH 4-20 ADDED TO PLAN	11/12/2020	VD
1	NEW BOREHOLES ADDED TO PLAN	17/12/2018	NC

LS GP INC.

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDING
2145 WALKLEY ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 03/2025
Drawn by: NFRV	Report No.: PG4440-1
Checked by: NFRV	Dwg. No.: PG4440-1
Approved by: DP	Revision No.: 5

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