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Geotechnical Investigation
Proposed Multi-Storey Building
1050 Canadian Shield Avenue, Ontario

Prepared For

Canadian Rental Development Services Inc.

March 23, 2022

Report: PG5371-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Canadian Rental Development Services Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1050 Canadian Shield Avenue 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 test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of one (1) multi-storey building with two (2) underground parking levels. Associated access lanes, hardscaped areas, and walkways are also anticipated as part of the proposed development. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out during the period of November 15 to November 17, 2021. The program consisted of advancing eight (8) boreholes (BH 1-21 to BH 8-21) down to a maximum depth of 9.3 m or practical auger refusal. Bedrock coring was completed to 3 m below the bedrock surface in three (3) boreholes (BH 1-21, BH 2-21, and BH 3-21). A previous investigation was also completed by this firm on May 28, 2020. At that time, a total of 20 boreholes were advanced to a maximum depth of 11.2 m below existing ground surface or refusal. The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5371-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of drilling to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

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

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

Rock samples were recovered from three boreholes drilled during the current investigation (BH 1-21, BH 2-21, and BH 3-21) using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Boreholes BH 1-21 and BH 2-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. Typical monitoring well construction details are described below:

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

The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5371-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the test holes were examined in our laboratory to review the results of the field logging. Two (2) samples were submitted for Atterberg Limits testing, one sample for shrinkage, and one (1) sample for grain size distribution testing.

All test results are included in Appendix 1 and further discussed in Subsection 4.2 of the current report.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant with a gravel covered parking area within the central area of the site. The ground surface across the subject site is generally flat and gently slopes down from the northwest portion (at approximate elevation of 102.7m) to the southeast portion (at approximate elevation of 99.0m). Also, the subject site was observed to be surrounded by trees along the southern and western borders of the site, and by a wide grassed area along the northern site border. Based on historical aerial photographs, it appears that the site was undergone topsoil removal and that piles of fill were stored within the subject site during the construction works at the adjacent eastern property in 2008.

The site is bordered by Campeau Drive to the north, Great Lakes Avenue to the west, Canadian Shield Avenue to the south, and by a multi-storey residential building and associated parking and landscaped areas to the east.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations within the northwest portion of the site consists of fill overlying a silty clay deposit. The fill consists of brown silty sand with crushed stone. The silty clay deposit consisted of a top hard to very stiff brown silty clay rust underlain by stiff grey silty clay below 3.8 to 4.4m depth. The silty clay deposit is underlain by compact glacial till and/or bedrock.

Practical refusal to augering or DCPT on bedrock was encountered in boreholes BH 1 through BH 20 and BH 5-21 through BH 8-21 at depths varying between 1.0 to 12.4m. A glacial till layer was encountered below the silty clay layer in BH 8, BH 5-21, BH 7-21, and BH 8-21 at a depth of 8.2 m, 5.9 m, 7.5 m, and 6.7 m, respectively. Also, a layer of brown silty sand was encountered below ground surface in BH 7.

The bedrock was cored in the locations of boreholes BH 1-21, BH 2-21, and BH 3-21 at depths between 2.9 m and 6.3 m below existing ground surface, with an average RQD value ranging from 45 to 100%. This is indicative of a poor to excellent quality bedrock within the footprint of the proposed building. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Bedrock

Based on available geological mapping, refusal to augering/DCPT and rock coring completed in BH 1-21, BH 2-21, and BH 3-21, the bedrock in the subject area consists of Paragneiss Migmatic bedrock of granitic origin, with an overburden thickness of 1 to 10 m depth.

Grain Size Distribution and Hydrometer Test

One sieve analysis was completed to classify selected soil samples according to the Unified Soil Classification System (USCS). The results are summarized in Table 1 and presented in Appendix 1.

Table 1 - Grain Size Distribution and Hydrometer Testing

Test Hole	Sample	Gravel (%)	Sand (%)	Silt and Clay (%)
BH 4-21	SS3	0	0.6	99.4

Atterberg Limit Tests

Two selected silty clay samples were submitted for Atterberg Limit testing. The test results indicate that the silty clay is classified as clay of High Plasticity (CH) in accordance with the Unified Soil Classification System. The results are summarized in Table 2 and presented in Appendix 1.

Table 2 - Summary of Atterberg Limits Test Results

Test Hole	Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
BH 5-21	SS3	52	26	26
BH 8-21	SS3	60	23	37

Shrinkage Test

The shrinkage limit and shrinkage ratio of the tested silty clay sample (BH 7-21 SS3) were found to be 20.06% and 1.792, respectively.

4.3 Groundwater

The groundwater levels were recorded within the monitoring wells and piezometers installed within the boreholes during the current investigations on November 23, 2021. The recorded groundwater levels are presented in Table 3 below and are further noted on the Soil Profile and Test Data sheets in Appendix 1.

Table 3 - Measured Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
*BH 1-21	98.97	3.12	95.85	November 23, 2021
BH 2-21	98.83	3.06	95.77	
BH 3-21	99.48	1.31	98.17	
BH 4-21	99.32	1.31	98.01	
BH 5-21	99.51	1.33	98.18	
BH 6-21	99.87	1.93	97.94	
BH 7-21	99.37	1.44	97.93	
BH 8-21	99.29	N/A	-	

Note: Ground surface elevations at the test hole locations were referenced to a geodetic datum.
**BH 1-21 had a monitoring well installation.*

It is important to note that groundwater readings can be influenced by surface water perched within the borehole backfill material. Long-term groundwater conditions can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that long-term groundwater level can be expected between geodetic elevations 95 m to 97 m. However, groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed multi-storey building. The foundation support system required is dependent on the design building loading and depth of foundation. Several foundation support options are listed below and discussed in the following sub-sections:

- Conventional footings placed on hard to very stiff brown silty clay, stiff grey silty clay, silty sand, glacial till bearing surface and/or a clean, surface sounded bedrock bearing surface.

- Conventional footings placed on vertical, zero entry lean concrete filled trenches extended to the underlying bedrock bearing surface where depths to bedrock are considered feasible for this application.

- End bearing piled foundations that extend down a clean, surface sounded bedrock bearing surface where the depth of bedrock is considered too deep for lean concrete filled trenches.

Based on the current conceptual drawing, it is expected that bedrock removal will be required for portions of the underground parking levels of the proposed structure. Line drilling and controlled blasting could be used where large quantities of bedrock are needed to be removed. The blasting operations should be planned and conducted under the guidance of a professional engineer with experience in blasting operations.

Based on the founding medium consisting mainly of bedrock for the majority of the underground parking structure and superstructure, a permissible grade raise restriction and tree planting restriction are not required from a geotechnical perspective.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, all existing fill material will be excavated from within the proposed building footprint. Bedrock removal will be required for portions of the underground parking levels.

Topsoil and deleterious fill, such as those containing organic materials, or construction debris/remnants should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Bedrock Removal

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

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site.

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

Excavation side slopes in sound bedrock can be 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. The 1 m horizontal ledge set back can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Bedrock Excavation Face Reinforcement

A bedrock stabilization system consisting of a combination of horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs. This system is usually considered where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for pavements. Where the fill is open-graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

5.3 Foundation Design

Several foundation design options are available for the proposed building depending on the design loading and foundation depth. The following foundation options are recommended:

Conventional Shallow Foundation

Using continuously applied loads, footings for the proposed building placed over an undisturbed silty clay, silty sand, glacial till or bedrock bearing surface can be designed using the bearing resistance values presented in Table 4.

Table 4 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Hard to very stiff brown silty clay	150	225
Stiff grey silty clay	125	200
Silty sand	120	200
Glacial till	200	300
Bedrock	-	1500

Note: Strip footings, up to 2 m wide, and pad footings, up to 5 m wide, placed over a silty clay bearing surface can be designed using the above noted bearing resistance values.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces or clean, surface-sounded bedrock bearing surface.

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

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.

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating zero entry, vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**20 MPa** 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, a test hole should be undertaken to assess the water infiltration issues and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. Where strip footings are butting against a vertical bedrock face, the extent of the vertical, in-filled trench can be stopped at the bedrock face. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**.

Deep Foundation - End Bearing Piles

A deep foundation method, such as end bearing piles, can be considered for the central north portion of the proposed structure where the bedrock layer extends to a depth well below the design underside of footing elevation. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 5. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 5. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles will also be required after at least 48 hours have elapsed since initial driving.

Table 5 - End Bearing Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	975	1460	10	36
245	12	1100	1650	10	42
245	13	1175	1760	10	45

Settlement

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to native soil and engineered fill bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium. In unfractured bedrock, a plane with a slope of 1H:6V can be used.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. 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 attached to the present letter report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5371-1 - Test Hole Location Plan, attached to the present letter report. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were located at 2, 3 and 15 m away from the first and last geophone and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the foundation of the building. 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 average overburden shear wave velocity is **236 m/s**, while the bedrock shear wave velocity is **2,517 m/s**. Provided the building will be founded on bedrock surface, the overburden shear wave velocity does not need to be considered for the calculation of V_{s30}.

Based on this, the Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

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

$$V_{s30=} \frac{30\,m}{\left(\frac{30\,m}{2{,}517\,m/s}\right)}$$

$$V_{s30=}\ 2{,}510\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed building is **2,510 m/s** provided the footings are placed directly on bedrock surface. Therefore, for the anticipated underside of footing elevation, a **Site Class A** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native soils and bedrock surface will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

Where silty sand or glacial till subgrade is encountered below the basement slab, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings). It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

If the floor slab is constructed in the areas of shallow bedrock, it is recommended that a minimum 300 mm thick layer (native soil plus crushed stone layer) be present between the floor slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

5.6 Basement Wall

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

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

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 site area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Design

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil and possibly bedrock. The proposed pavement structures are presented in Tables 6 and 7.

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

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, bedrock or concrete fill.	

Table 7 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

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

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

5.8 Hydraulic Conductivity

Paterson completed a set of slug testing in the installed monitoring wells at the subject site as part of the geotechnical investigation in order to establish site-specific hydraulic conductivity values for the encountered bedrock.

The results of the slug testing are summarized in Table 8 below.

Table 8 – Measured Hydraulic Conductivity

BH I.D.	Depth Interval (m)	Hydraulic Conductivity (m/sec)
BH 1-21	5 - 6	3.38×10^{-8}
BH 2-21	5 - 6	1.27×10^{-4} to 1.54×10^{-4}

It should be noted that the measured hydraulic conductivity values were observed to vary widely between the two borehole locations, which can be attributed to a higher fracturing frequency in the bedrock at the location of BH 2-21.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the information provided, it is expected that a portion of the proposed building foundation walls located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which breaches the primary ground infiltration control system.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level and the following is suggested for preliminary design purposes:

- Place a suitable waterproofing membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- Pour foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 6-9 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Underfloor Drainage

Underfloor drainage is required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

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

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

The footings located along parking garage entrance may require protection against frost action depending on 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

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will not be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, temporary shoring of the overburden material will be needed.

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

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

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

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

Table 9 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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

Permit to Take Water

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

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

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day/building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the anticipated groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighboring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighboring structures.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

6.8 Landscaping Considerations

Tree Planting Restrictions

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution and hydrometer testing were also completed on selected soil samples. The results of our testing are presented in Subsection 4.2 and in Appendix 1.

Based on the results of our review, and on the anticipated founding depth no tree planting restrictions are required for the proposed building at the subject site.

7.0 Recommendations

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

- Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction.
- Review the bedrock stabilization and excavation requirements.
- Review the implementation of the water suppression system.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Sampling and testing of the concrete and fill materials used.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Canadian Rental Development Services 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.

LICENSED PROFESSIONAL ENGINEER
March 23, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Maha Saleh, P.Eng (Prov)

David J. Gilbert, P.Eng

Report Distribution:

☐ Canadian Rental Development Services Inc. (3 copies)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE TESTING RESULTS

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

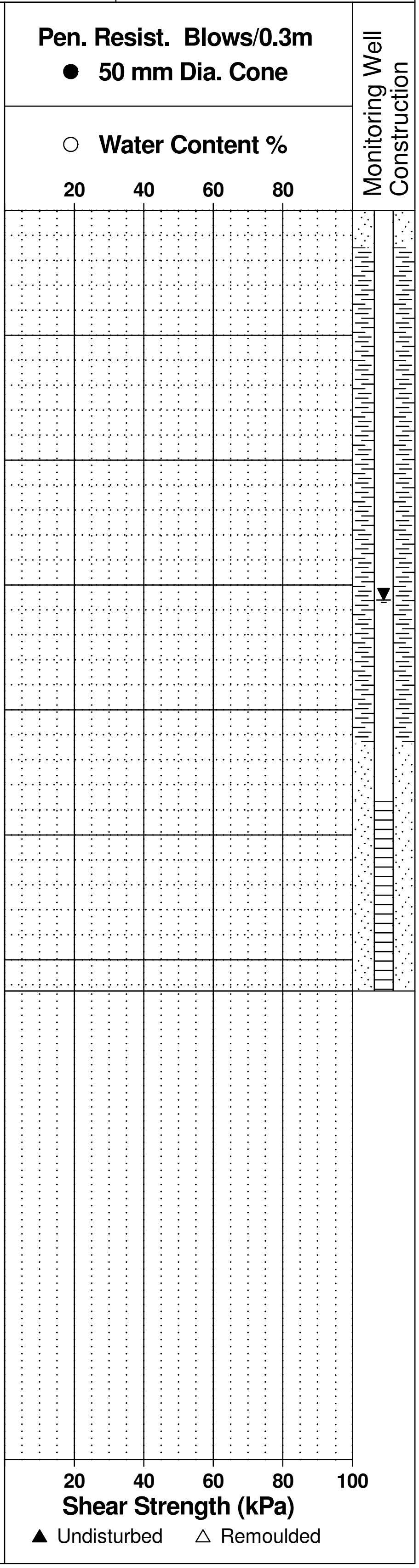
DATE November 15, 2021

FILE NO. PG5371

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brownsilty sand with gravel, cobbles and boulders, some fragmented rock, trace clay	AU	1		
	SS	2	8	9
	SS	3	8	8
	SS	4	4	12
2.90				
BEDROCK: Fair to excellent quality, dark grey gneiss	RC	1	100	52
	RC	2	100	100
	RC	3	100	100
6.25				
End of Borehole (GWL @ 3.12m - Nov. 23, 2021)				

DEPTH (m)	ELEV. (m)
0	98.97
1	97.97
2	96.97
3	95.97
4	94.97
5	93.97
6	92.97



patersongroup Consulting Engineers
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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE November 16, 2021

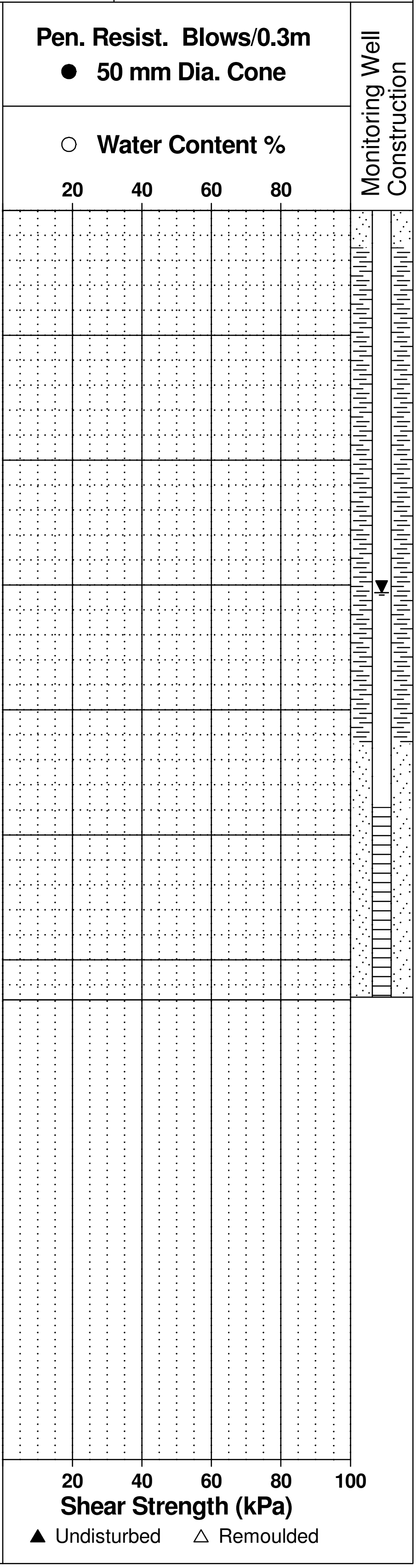
FILE NO. PG5371

HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, cobbles, fragmented rock, trace clay - red by 3.0m depth	3.05
BEDROCK: Poor to good quality, grey to red gneiss - 12mm thick mud seam at 3.4m depth	6.32
End of Borehole (GWL @ 3.06m - Nov. 23, 2021)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	25
SS	3	58	30
SS	4	75	35
SS	5	0	50+
RC	1	100	45
RC	2	100	95
RC	3	100	66

DEPTH (m)	ELEV. (m)
0	98.83
1	97.83
2	96.83
3	95.83
4	94.83
5	93.83
6	92.83



SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

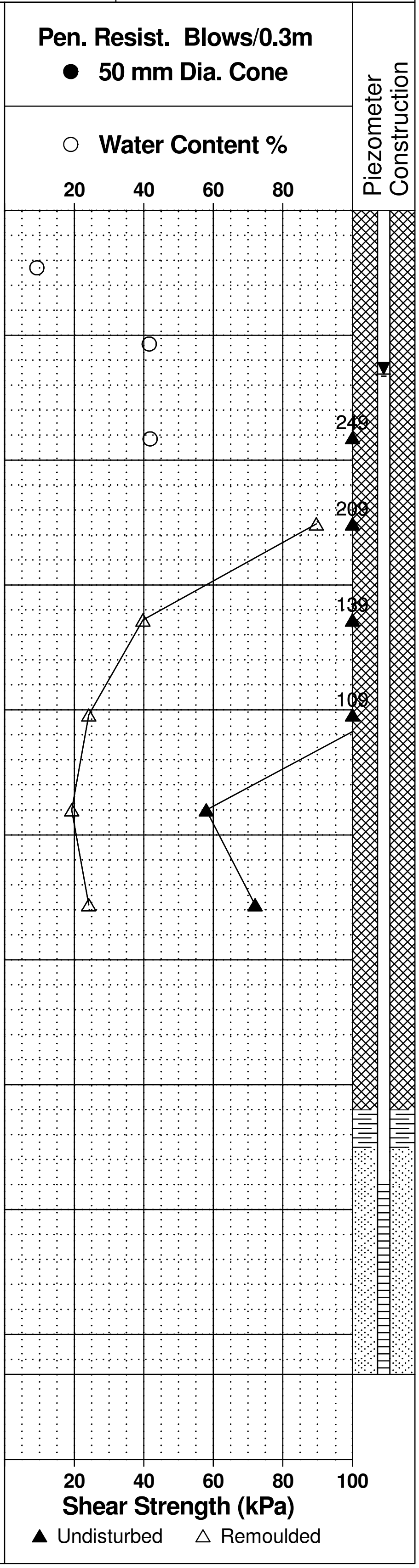
BORINGS BY Track-Mount Power Auger

DATE November 16, 2021

FILE NO. PG5371

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.48
TOPSOIL 0.05							
FILL: Brown silty sand with gravel, cobbles and fragmented rock, trace clay 0.60		AU	1				
		SS	2	83	8	1	98.48
		SS	3	100	P	2	97.48
Hard to very stiff, brown **SILTY CLAY**						3	96.48
						4	95.48
- stiff and grey by 4.4m depth						5	94.48
6.25		SS	4	0	50+	6	93.48
BEDROCK: Excellent quality, dark grey gneiss		RC	1	100	100	7	92.48
		RC	2	100	100	8	91.48
9.32		RC	3	100	100	9	90.48
End of Borehole							
(GWL @ 1.31m - Nov. 23, 2021)							



SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

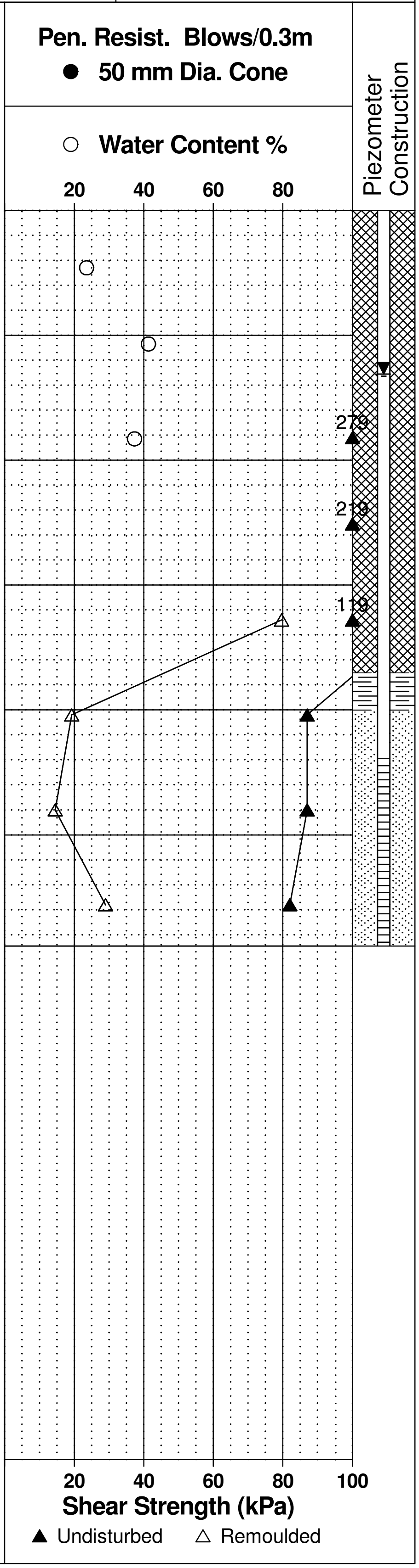
BORINGS BY Track-Mount Power Auger

DATE November 17, 2021

FILE NO. PG5371

HOLE NO. BH 4-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.32
TOPSOIL 0.05						
FILL: Brown silty clay, trace gravel and cobbles 0.60	AU	1				
	SS	2	83	8	1	98.32
	SS	3	83	P		
					2	97.32
Hard to very stiff, brown **SILTY CLAY**						
					3	96.32
- stiff and grey by 3.8m depth					4	95.32
					5	94.32
5.89						
End of Borehole						
(GWL @ 1.31m - Nov. 23, 2021)						



SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE November 17, 2021

FILE NO. PG5371

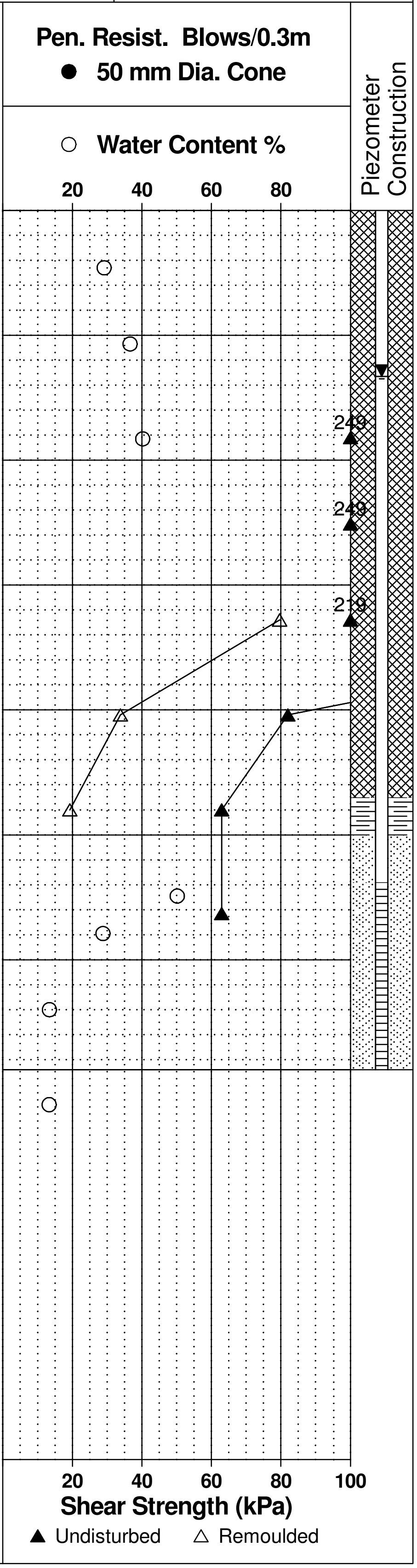
HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.51
FILL: Brown silty clay, some gravel, trace organics		AU	1				
0.66							
		SS	2	83	9	1	98.51
		SS	3	83	P	2	97.51
Hard to very stiff, brown **SILTY CLAY**						3	96.51
- stiff and grey by 3.8m depth						4	95.51
						5	94.51
		SS	4	100	P		
5.94						6	93.51
GLACIAL TILL: Stiff, grey silty clay with sand, gravel and cobbles		SS	5	83	2		
6.88		SS	6	100	50+		

End of Borehole

Practical refusal to augering at 6.88m depth

(GWL @ 1.33m - Nov. 23, 2021)



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

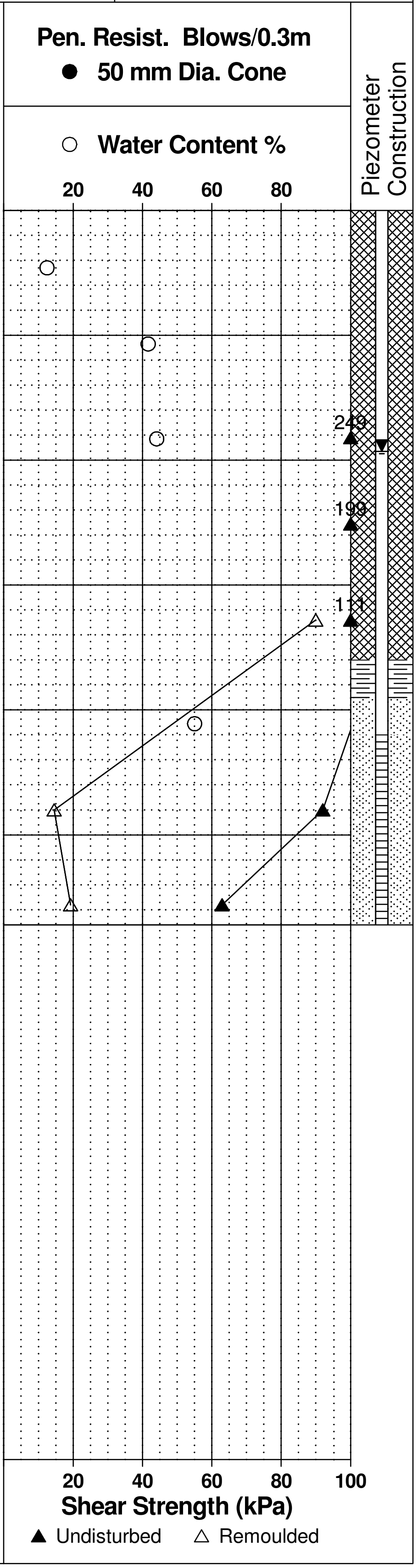
BORINGS BY Track-Mount Power Auger

DATE November 17, 2021

FILE NO. PG5371

HOLE NO. BH 6-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.87
FILL: Brown silty clay, trace sand, gravel and cobbles		AU	1				
0.56							
		SS	2	75	8	1	98.87
		SS	3	83	P	2	97.87
Hard to very stiff, brown **SILTY CLAY**						3	96.87
- stiff and grey by 3.8m depth		SS	4	100	2	4	95.87
						5	94.87
5.72							
End of Borehole Practical refusal to augering at 5.72m depth (GWL @ 1.93m - Nov. 23, 2021)							



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic

REMARKS

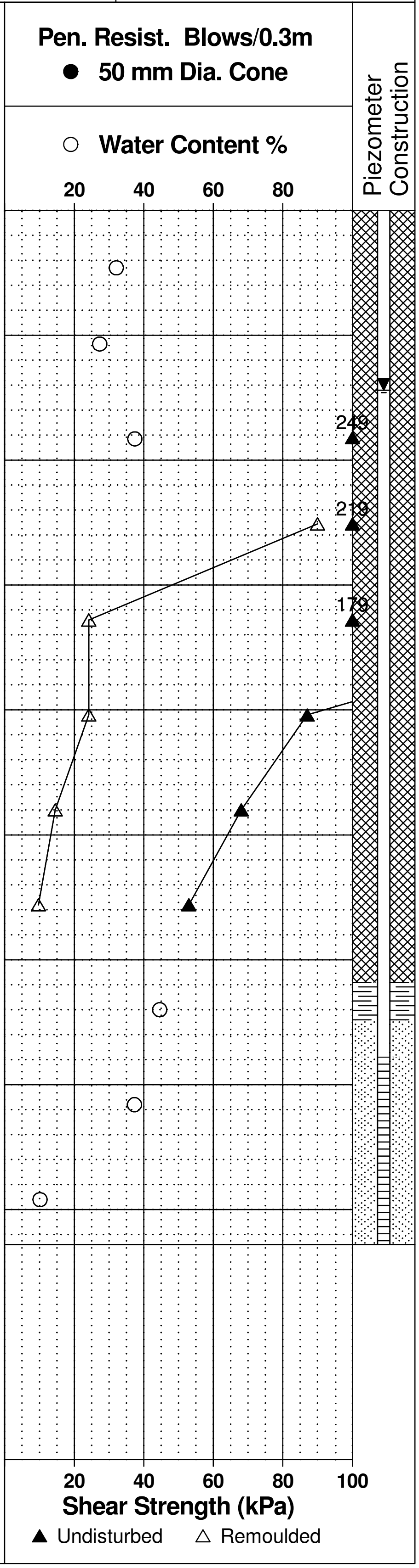
BORINGS BY Track-Mount Power Auger

DATE November 17, 2021

FILE NO. PG5371

HOLE NO. BH 7-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.37
FILL: Gravel and crushed stone 0.15						
	AU	1				
	SS	2	92		1	98.37
	SS	3	83	P		
					2	97.37
Hard to very stiff, brown **SILTY CLAY**						
					3	96.37
- stiff and grey by 3.8m depth					4	95.37
					5	94.37
					6	93.37
	SS	4	83	3		
					7	92.37
	SS	5	83	2		
7.54						
GLACIAL TILL: Very stiff, grey silty clay with sand, gravel, cobbles and boulders	SS	6	75	5	8	91.37
8.28						
End of Borehole Practical refusal to augering at 8.28m depth. (GWL @ 1.44m - Nov. 23, 2021)						



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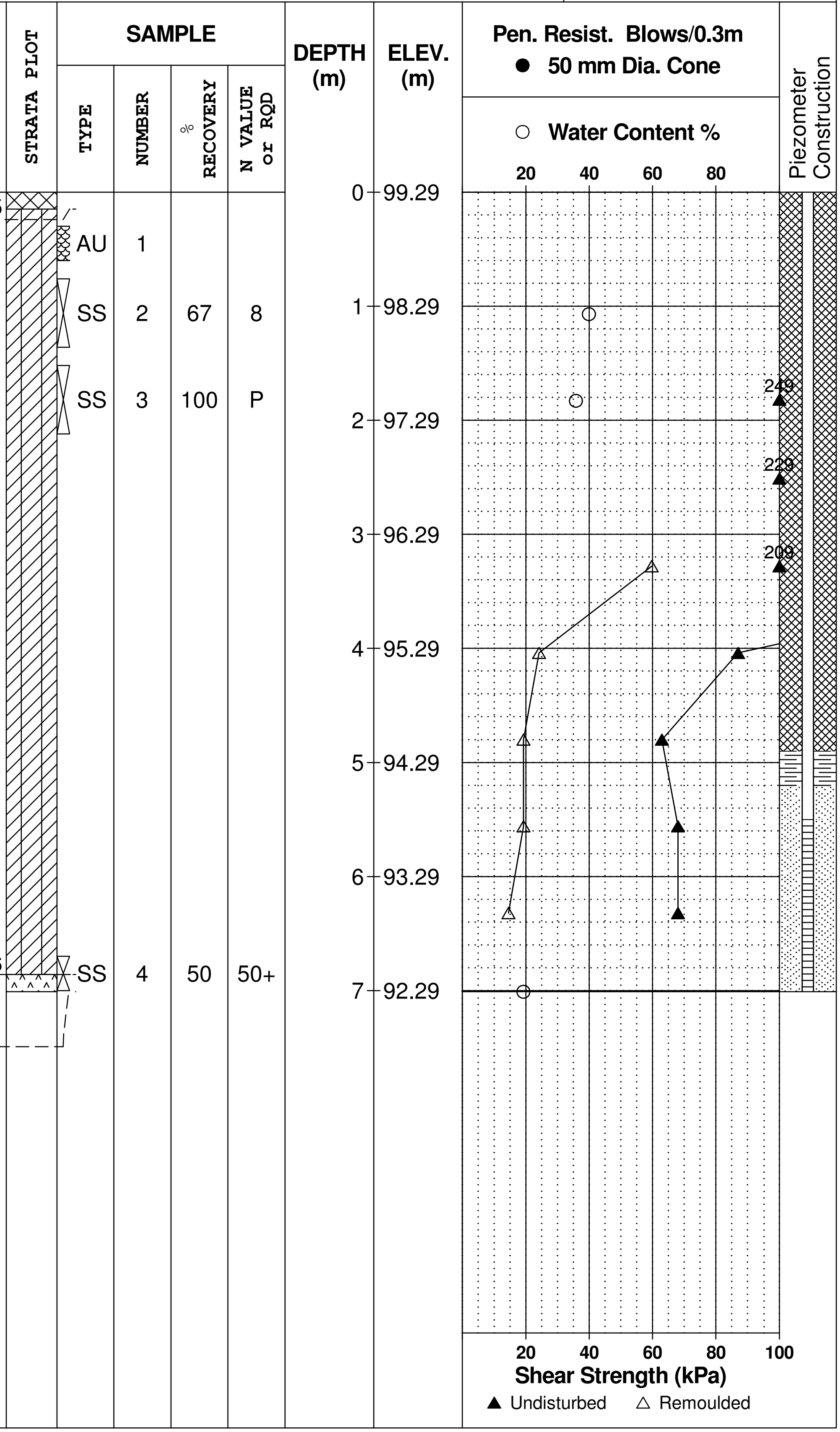
SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Canadian Shield Way
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE November 17, 2021
FILE NO. PG5371
HOLE NO. BH 8-21

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Gravel and crushed stone	0.15				
		AU	1		
		SS	2	67	8
		SS	3	100	P
Hard to very stiff, brown **SILTY CLAY**					
- stiff and grey by 3.8m depth					
	6.86	SS	4	50	50+
GLACIAL TILL: Stiff,grey silty clay with sand, gravel, cobbles and boulders	7.01				
End of Borehole					
Practical refusal to augering at 7.01m depth					
(Piezomter destroyed - Nov. 23, 2021)					

DEPTH (m)	ELEV. (m)
0	99.29
1	98.29
2	97.29
3	96.29
4	95.29
5	94.29
6	93.29
7	92.29



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SOIL PROFILE AND TEST DATA

Bedrock Surface Delineation Investigation
1050 Canadian Shield Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH 1

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	102.65		
Brown **SILTY CLAY** with gravel	▨					1	101.65		
1.80									
End of Borehole Practical refusal to augering at 1.80m depth									

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

Bedrock Surface Delineation Investigation
1050 Canadian Shield Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.65
Brown to grey **SILTY CLAY** with sand, trace gravel	▨					1	101.65
						2	100.65
						3	99.65
						4	98.65
						5	97.65
						6	96.65
						7	95.65
						8	94.65
						9	93.65
						10	92.65
11.23						11	91.65
End of Borehole Practical refusal to augering at 11.23m depth							

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Bedrock Surface Delineation Investigation
1050 Canadian Shield Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

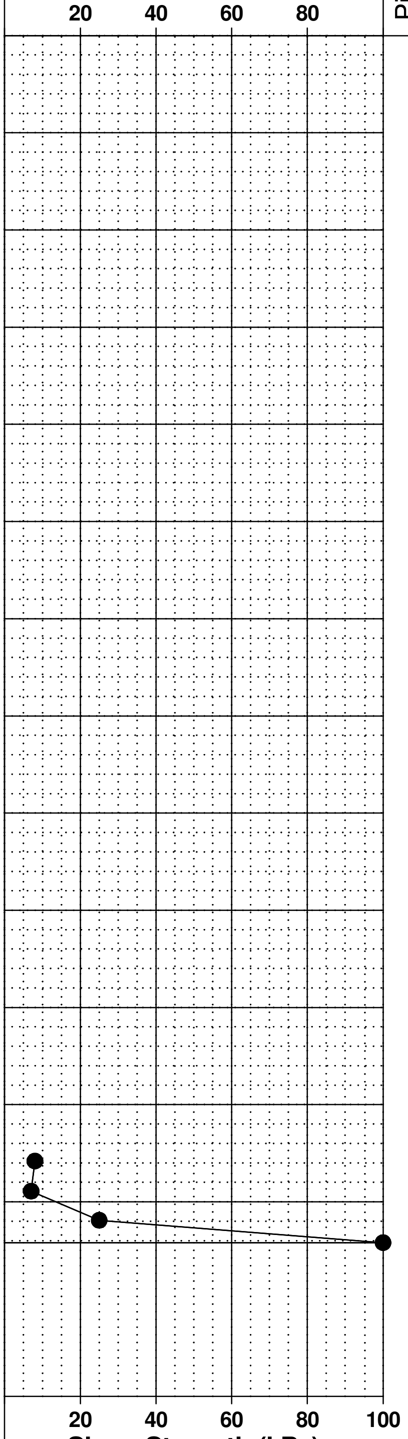
HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.33
						1	101.33
						2	100.33
						3	99.33
Brown to grey **SILTY CLAY,** some sand	▨					4	98.33
						5	97.33
						6	96.33
						7	95.33
						8	94.33
9.14						9	93.33
Dynamic Cone Penetration Test commenced at 9.14m depth. Cone pushed to 11.3m depth.						10	92.33
						11	91.33
						12	90.33
12.42							
End of Borehole Practical DCPT refusal at 12.42m depth							

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

○ **Water Content %**

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	101.93		
Brown **SILTY CLAY,** some sand and gravel	▨					1	100.93		
1.98									
End of Borehole Practical refusal to augering at 1.98m depth									

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

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SOIL PROFILE AND TEST DATA

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FILE NO. PG5371

REMARKS

HOLE NO. BH 5

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	101.78		
Brown **SILTY CLAY**	▨					1	100.78		
	▨					2	99.78		
	▨					3	98.78		
3.89									
End of Borehole Practical refusal to augering at 3.89m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE									
FILL: Brown silty sand with crushed stone	▨					0	100.88		
	▨					1	99.88		
	▨					2	98.88		
	▨					3	97.88		
3.71									
End of Borehole Practical refusal to augering at 3.71m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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FILE NO. PG5371

REMARKS

HOLE NO. BH 7

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 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	99.18		
Brown **SILTY SAND,** some gravel						1	98.18		
						2	97.18		
						3	96.18		
3.48									
End of Borehole Practical refusal to augering at 3.48m depth									

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

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REMARKS

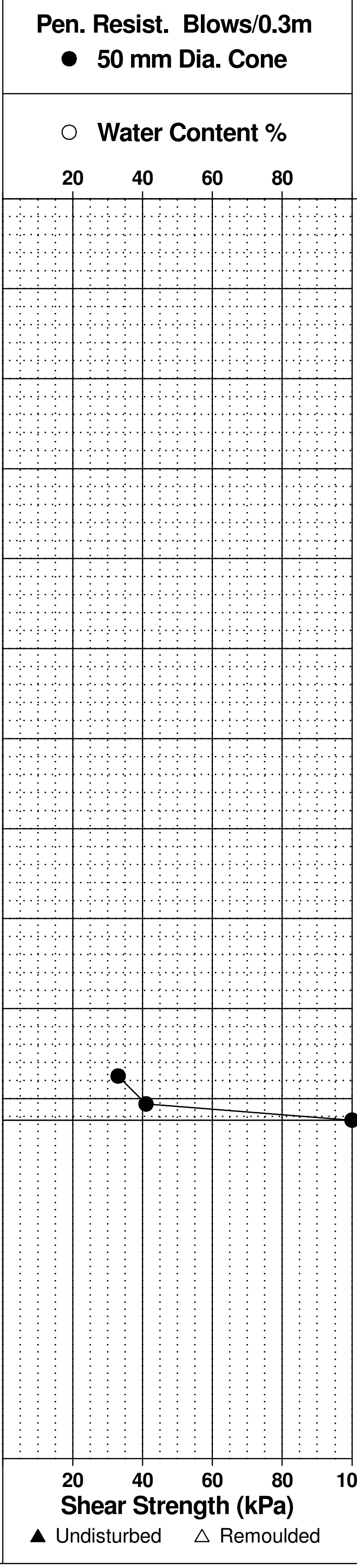
BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.36
						1	98.36
						2	97.36
						3	96.36
Brown to grey **SILTY CLAY**						4	95.36
						5	94.36
						6	93.36
						7	92.36
8.23						8	91.36
GLACIAL TILL 9.14						9	90.36
Dynamic Cone Penetration Test commenced at 9.14m depth. Inferred **GLACIAL TILL** 10.24						10	89.36
End of Borehole Practical DCPT refusal at 10.24m depth							



Piezometer Construction

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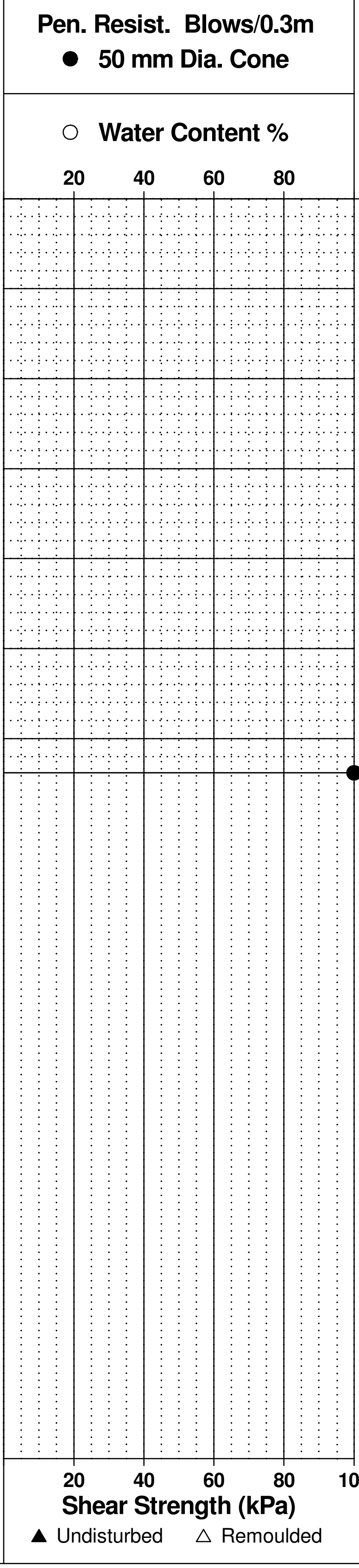
BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

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	99.60
FILL: Crushed stone 0.60							
						1	98.60
						2	97.60
Brown to grey **SILTY CLAY**						3	96.60
						4	95.60
						5	94.60
6.10						6	93.60
Dynamic Cone Penetration Test commenced at 6.10m depth 6.38							
End of Borehole Practical DCPT refusal at 6.38m depth							



Piezometer Construction

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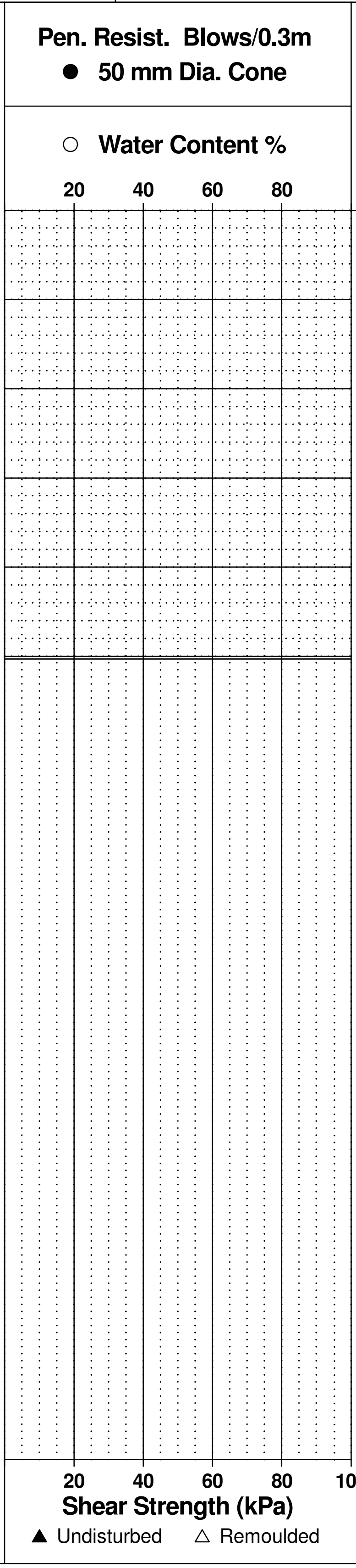
BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.55
FILL: Brow silty sand with gravel	▨						
0.60						1	99.55
Brown to grey **SILTY CLAY**	▨					2	98.55
						3	97.55
						4	96.55
5.03						5	95.55
End of Borehole Practical refusal to augering at 5.03m depth							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Bedrock Surface Delineation Investigation
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FILE NO. **PG5371**

REMARKS

HOLE NO. **BH11**

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	100.15		
						1	99.15		
Brown to grey **SILTY CLAY**	▨					2	98.15		
						3	97.15		
						4	96.15		
5.33						5	95.15		
End of Borehole Practical refusal to augering at 5.33m depth									

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

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Ottawa, Ontario

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH12

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									
FILL: Brown silty clay, some gravel and crushed stone 0.60	☒					0	99.66		
	▨					1	98.66		
	▨					2	97.66		
	▨					3	96.66		
Grey **SILTY CLAY**	▨					4	95.66		
	▨					5	94.66		
	▨					6	93.66		
7.60	▨					7	92.66		
End of Borehole Practical refusal to augering at 7.60m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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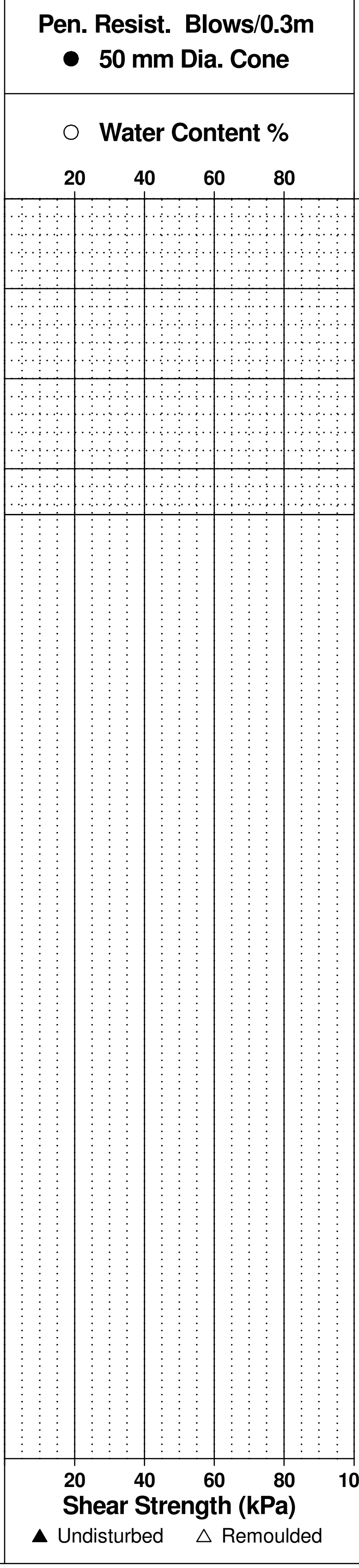
BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH13

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
	▨					0	99.20	
FILL: Crushed stone over brown silty clay with gravel and crushed stone	▨					1	98.20	
	▨					2	97.20	
3.51	▨					3	96.20	
End of Borehole Practical refusal to augering at 3.51m depth								



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SOIL PROFILE AND TEST DATA

Bedrock Surface Delineation Investigation
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FILE NO. PG5371

REMARKS

HOLE NO. BH14

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	98.81		
FILL: Brown silty sand with crushed stone	▨					1	97.81		
1.50									
End of Borehole Practical refusal to augering at 1.50m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. PG5371

HOLE NO. BH15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	98.93		
FILL: Brown silty sand with clay and crushed stone	▨					1	97.93		
1.04									
End of Borehole Practical refusal to augering at 1.04m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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FILE NO. PG5371

REMARKS

HOLE NO. BH16

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	98.90		
FILL: Brown silty sand with crushed stone	▨					1	97.90		
	▨					2	96.90		
3.15	▨					3	95.90		
End of Borehole Practical refusal to augering at 3.15m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

FILE NO. PG5371

HOLE NO. BH17

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	99.05		
FILL: Brown silty sand with crushed stone	▨					1	98.05		
	▨					2	97.05		
2.77									
End of Borehole Practical refusal to augering at 2.77m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

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REMARKS

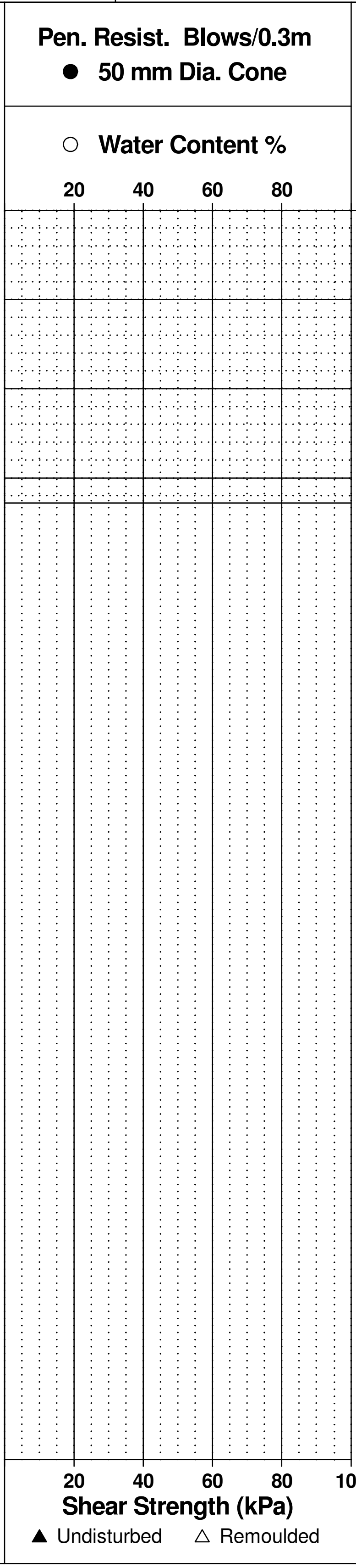
BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

FILE NO. PG5371

HOLE NO. BH18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0	99.62	
FILL: Brown silty sand with crushed stone	▨					1	98.62	
						2	97.62	
3.28						3	96.62	
End of Borehole Practical refusal to augering at 3.28m depth								



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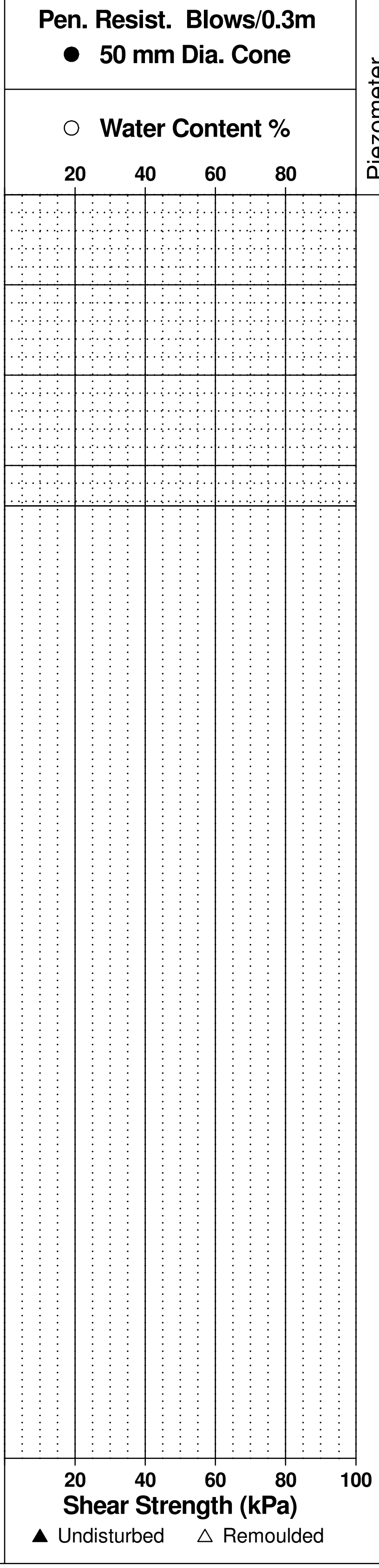
BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

FILE NO. PG5371

HOLE NO. BH19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	99.98
						1	98.98
FILL: Brown silty sand with crushed stone	▨					2	97.98
						3	96.98
3.45							
End of Borehole Practical refusal to augering at 3.45m depth							



Piezometer Construction

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DATE May 29, 2020

FILE NO. PG5371

HOLE NO. BH20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	100.48		
						1	99.48		
FILL: Brown silty sand with crushed stone	▨								
						2	98.48		
2.80									
End of Borehole Practical refusal to augering at 2.80m depth									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

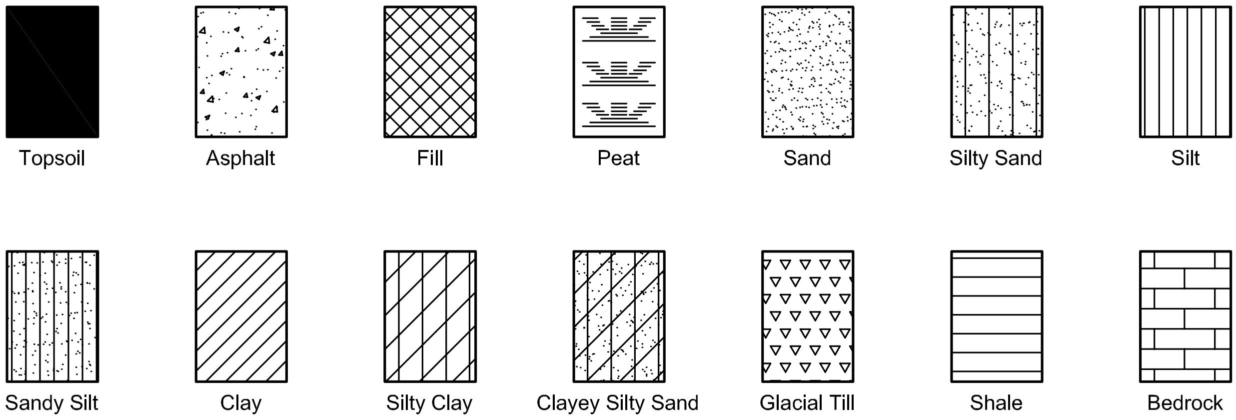
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

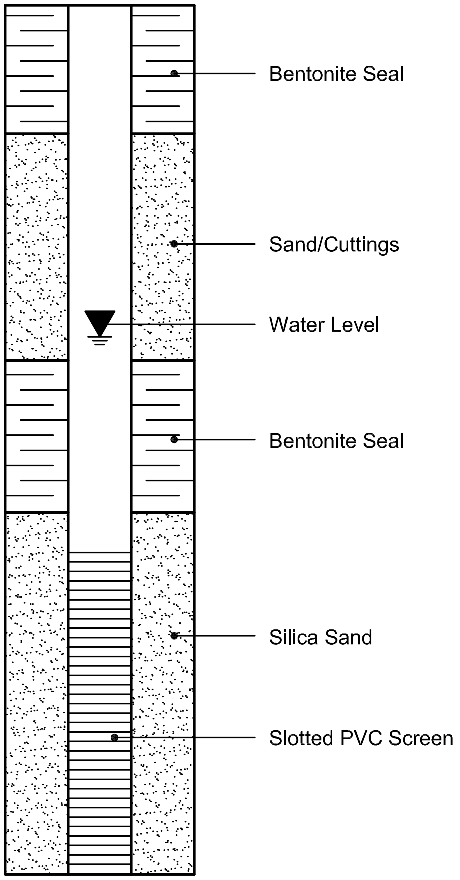
SYMBOLS AND TERMS (continued)

STRATA PLOT

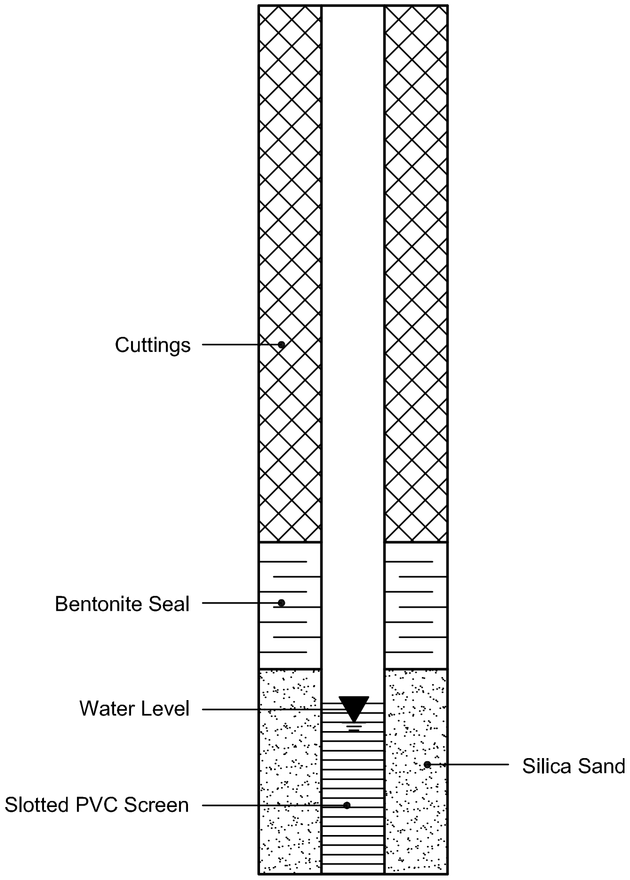


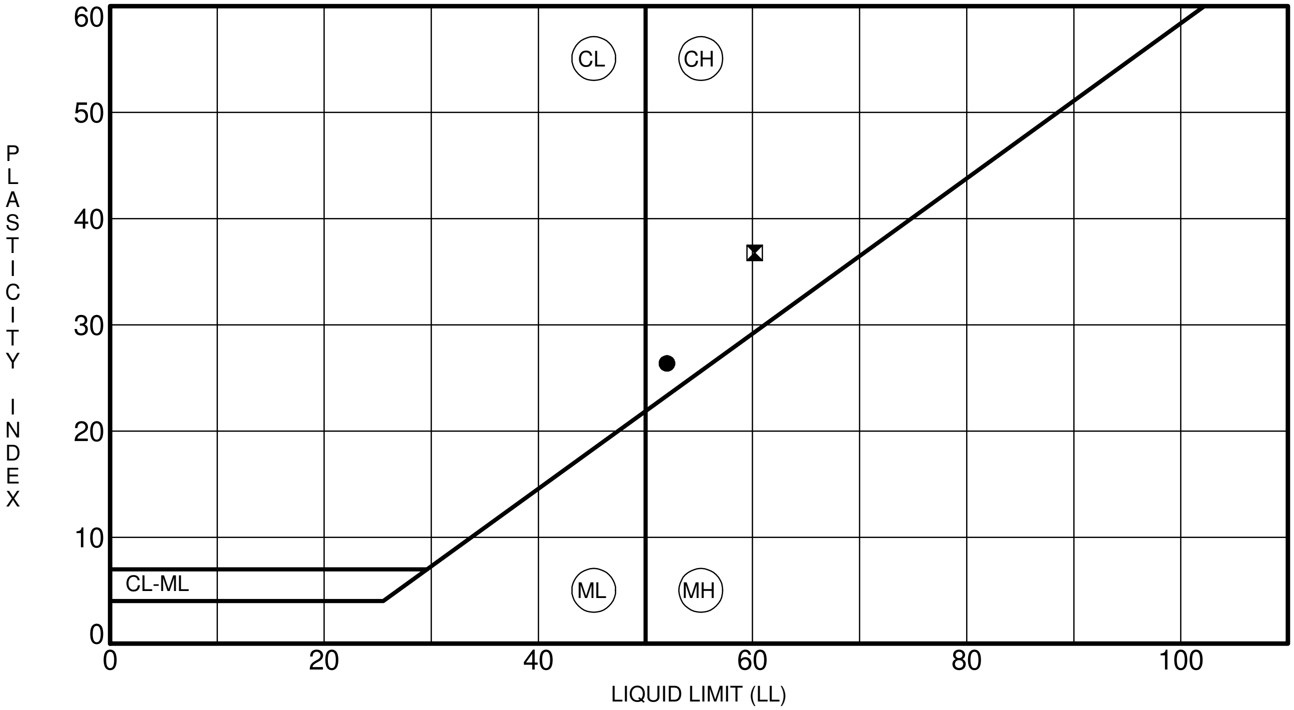
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 5-21**	**SS3**	**52**	**26**	**26**		**CH - Inorganic clays of high plasticity**
⊠	**BH 8-21**	**SS3**	**60**	**23**	**37**		**CH - Inorganic clays of high plasticity**

CLIENT **Canadian Rental Development Services Inc.**

PROJECT **Supplemental Geotechnical Investigation - Prop.**
Multi-Storey Building - 1050 Canadian Shield Way

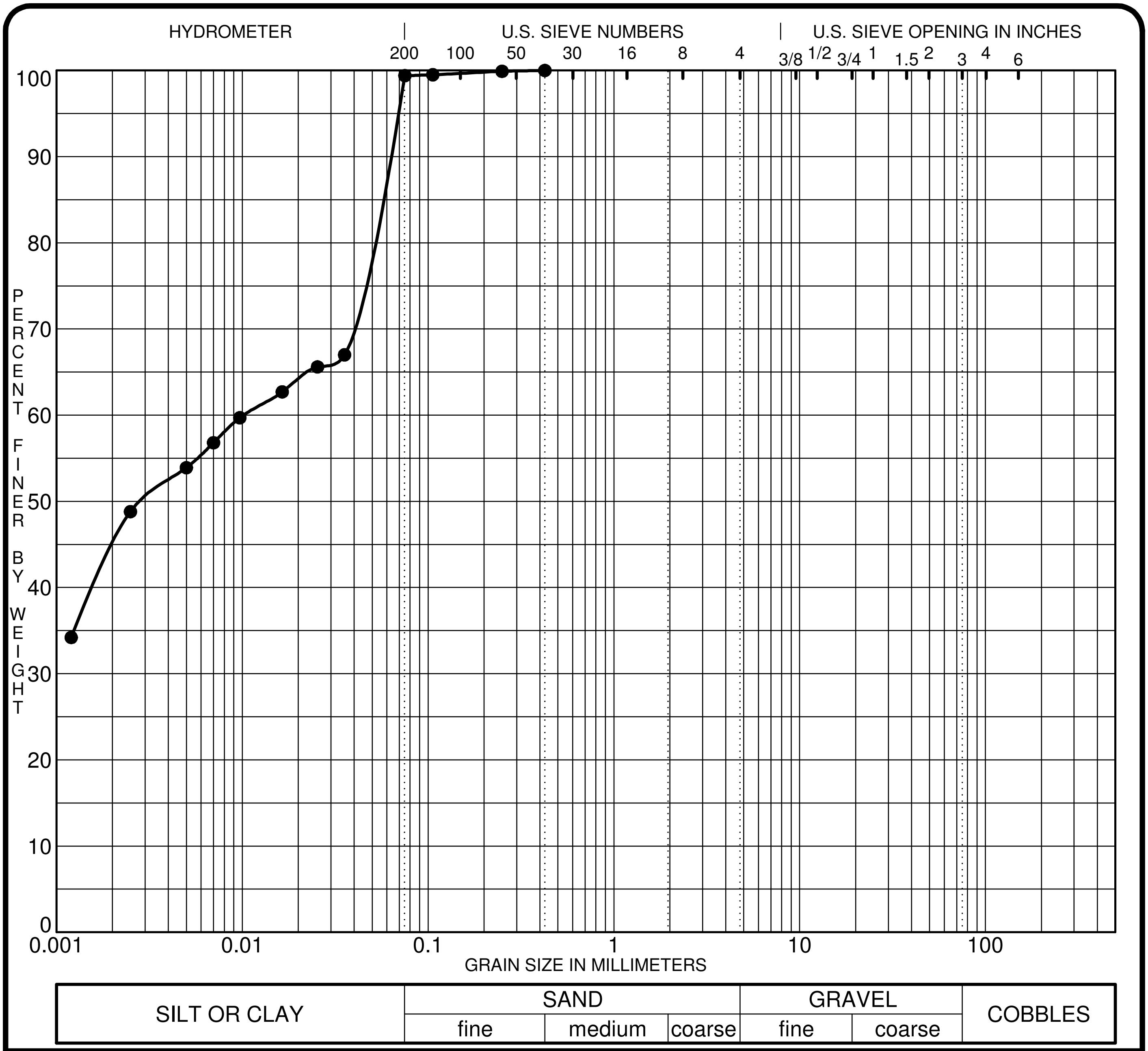
FILE NO. **PG5371**

DATE **17 Nov 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-21**	**SS3**	**CH - Inorganic clays of high plasticity**						
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-21**	**SS3**	**0.43**	**0.01**			**0.0**	**0.6**	**99.4**	
⊠										
▲										
★										

CLIENT **Canadian Rental Development Services Inc.**

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GRAIN SIZE DISTRIBUTION

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5371-1 – TEST HOLE LOCATION PLAN

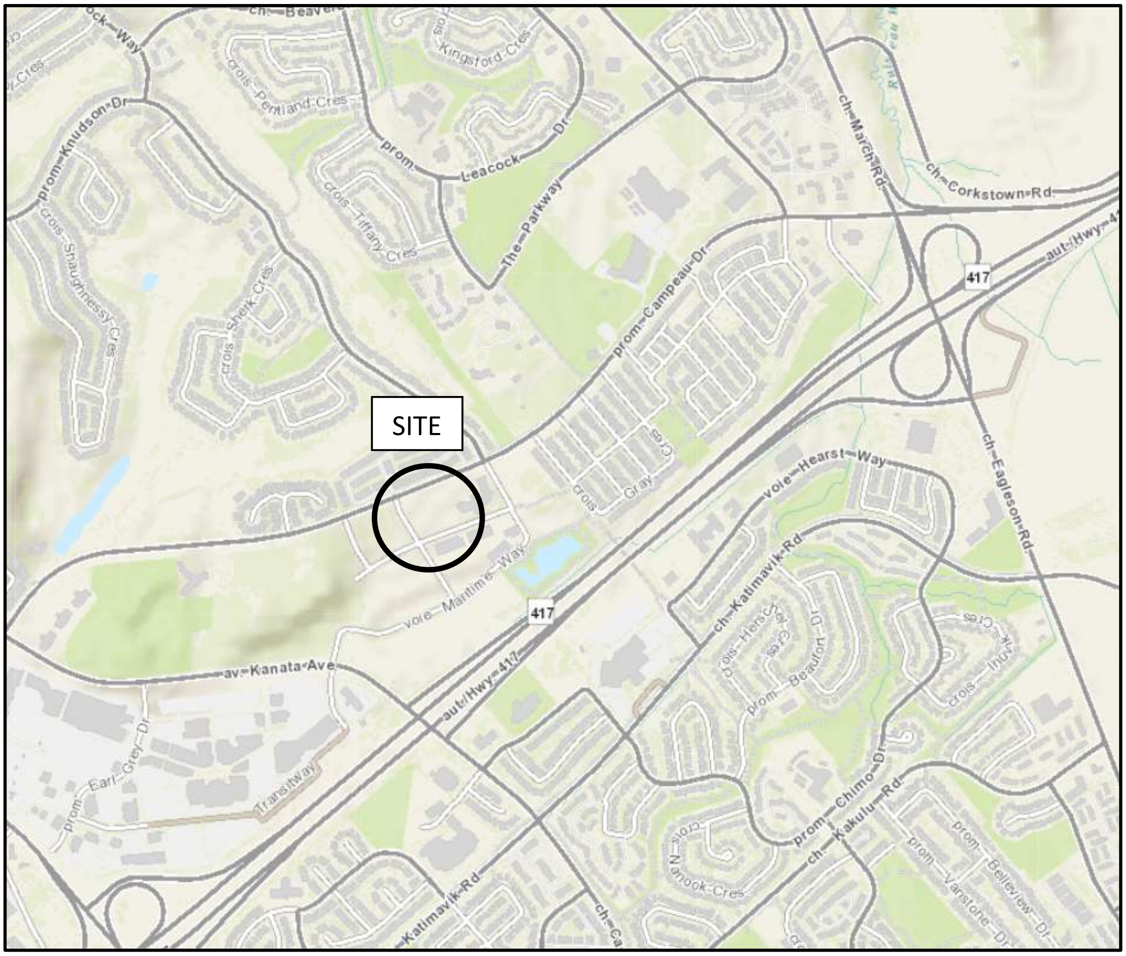


FIGURE 1

KEY PLAN

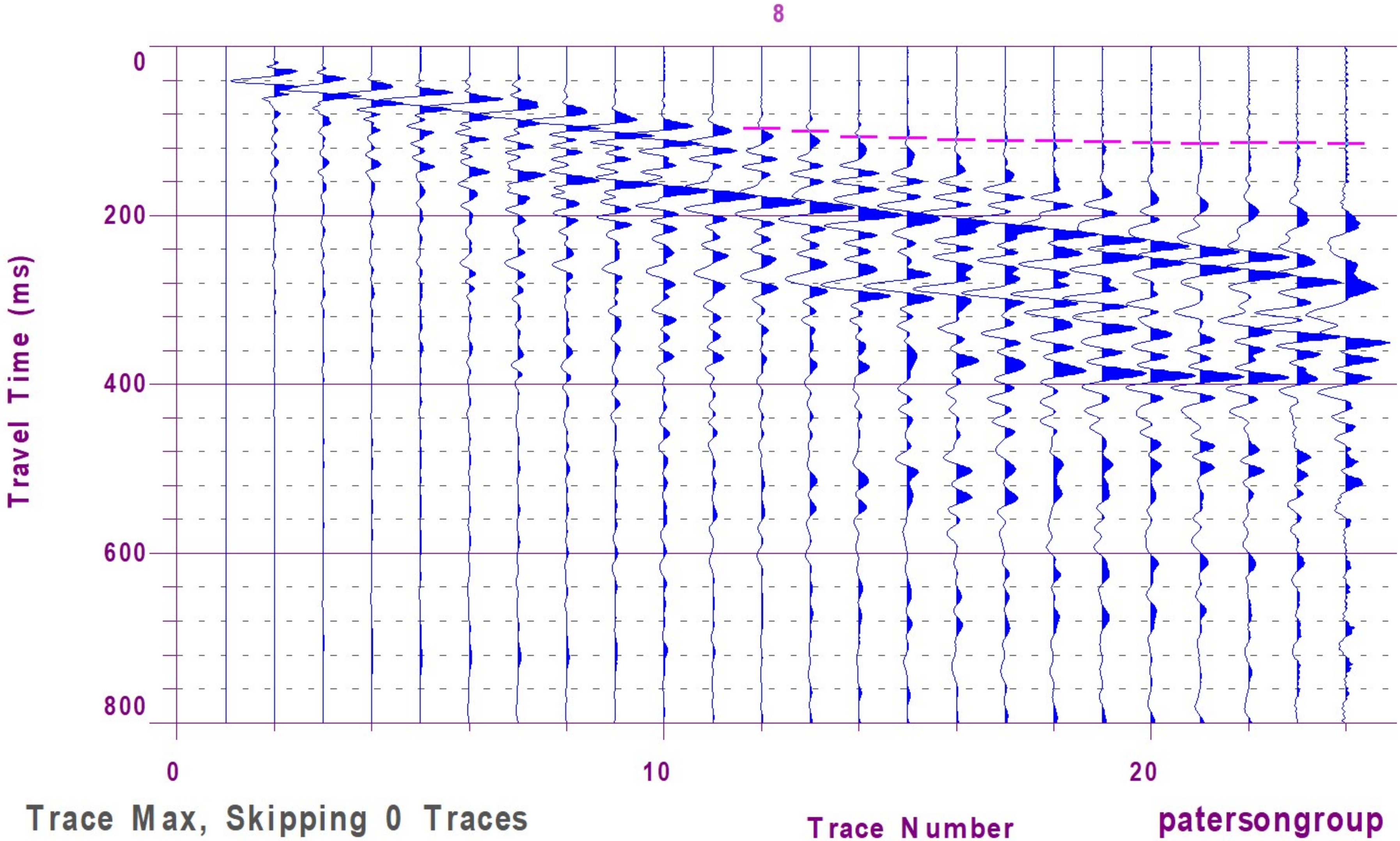


Figure 2 – Shear Wave Velocity Profile at Shot Location -2 m

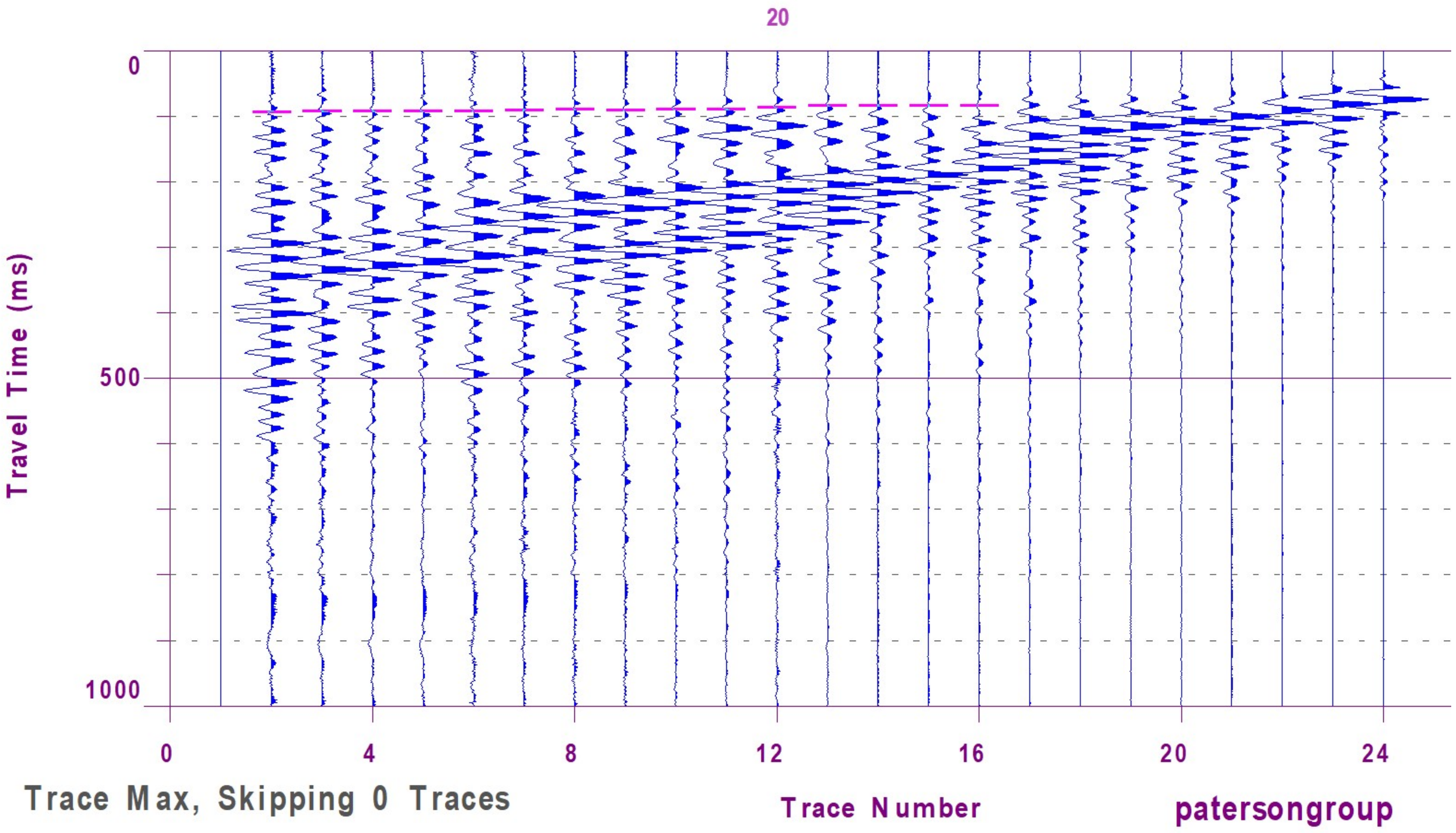
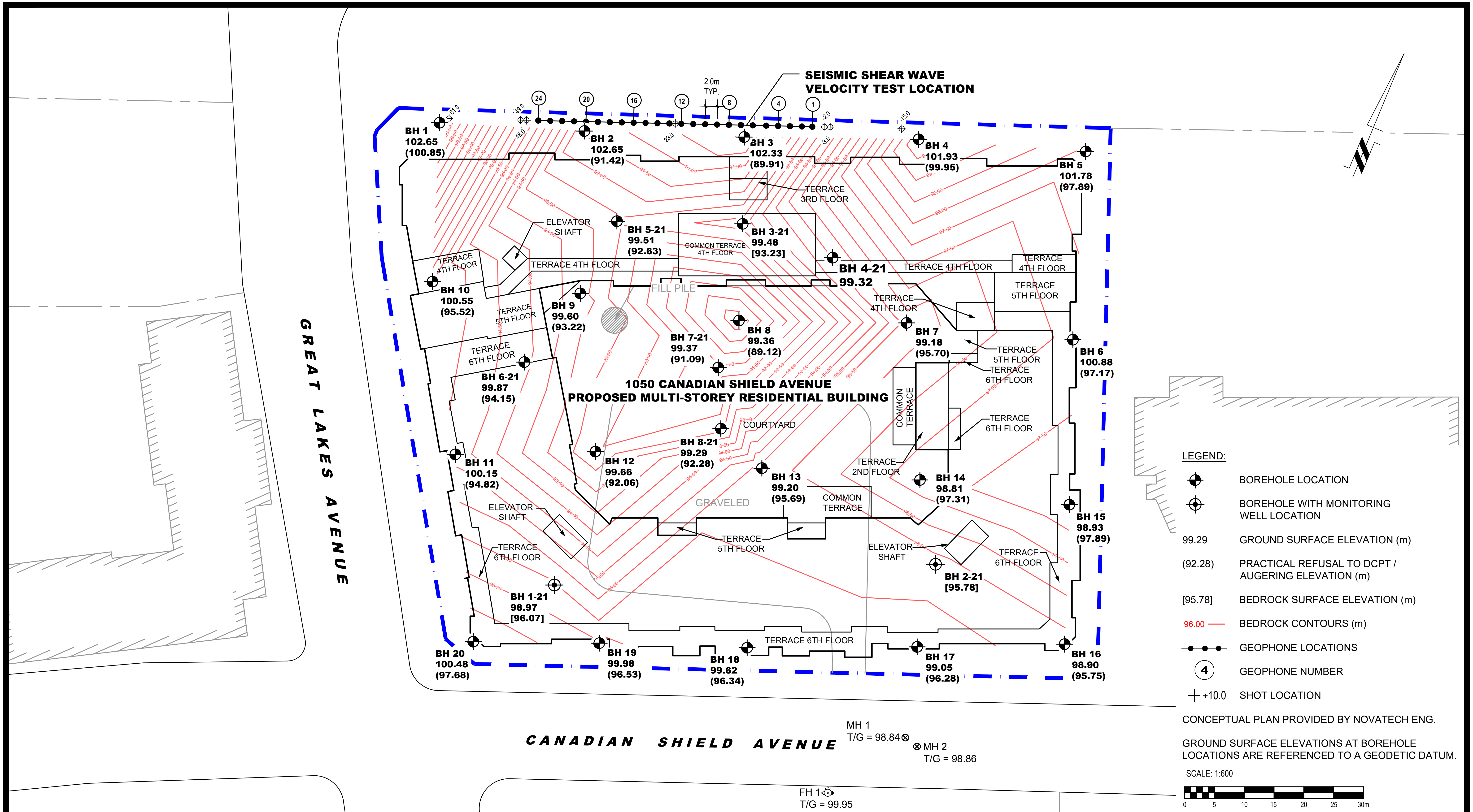


Figure 3 – Shear Wave Velocity Profile at Shot Location 61 m



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NO.	REVISIONS	DATE	INITIAL
2	BH 1-21 - BH 8-21 ADDED TO PLAN UPDATED CONCEPTUAL PLAN ADDED TO DRAWING	11/16/2021	MS
1	SEISMIC SHEAR WAVE VELOCITY TEST LOCATION ADDED	12/07/2021	FC

CANADIAN RENTAL DEVELOPMENT SERVICES

BEDROCK SURFACE DELINEATION INVESTIGATION
1050 CANADIAN SHIELD AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 06/2020
Drawn by: YA
Report No.: PG5371-1
Checked by: KP
Dwg. No.: PG5371-1
Approved by: SD
Revision No.: 2

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