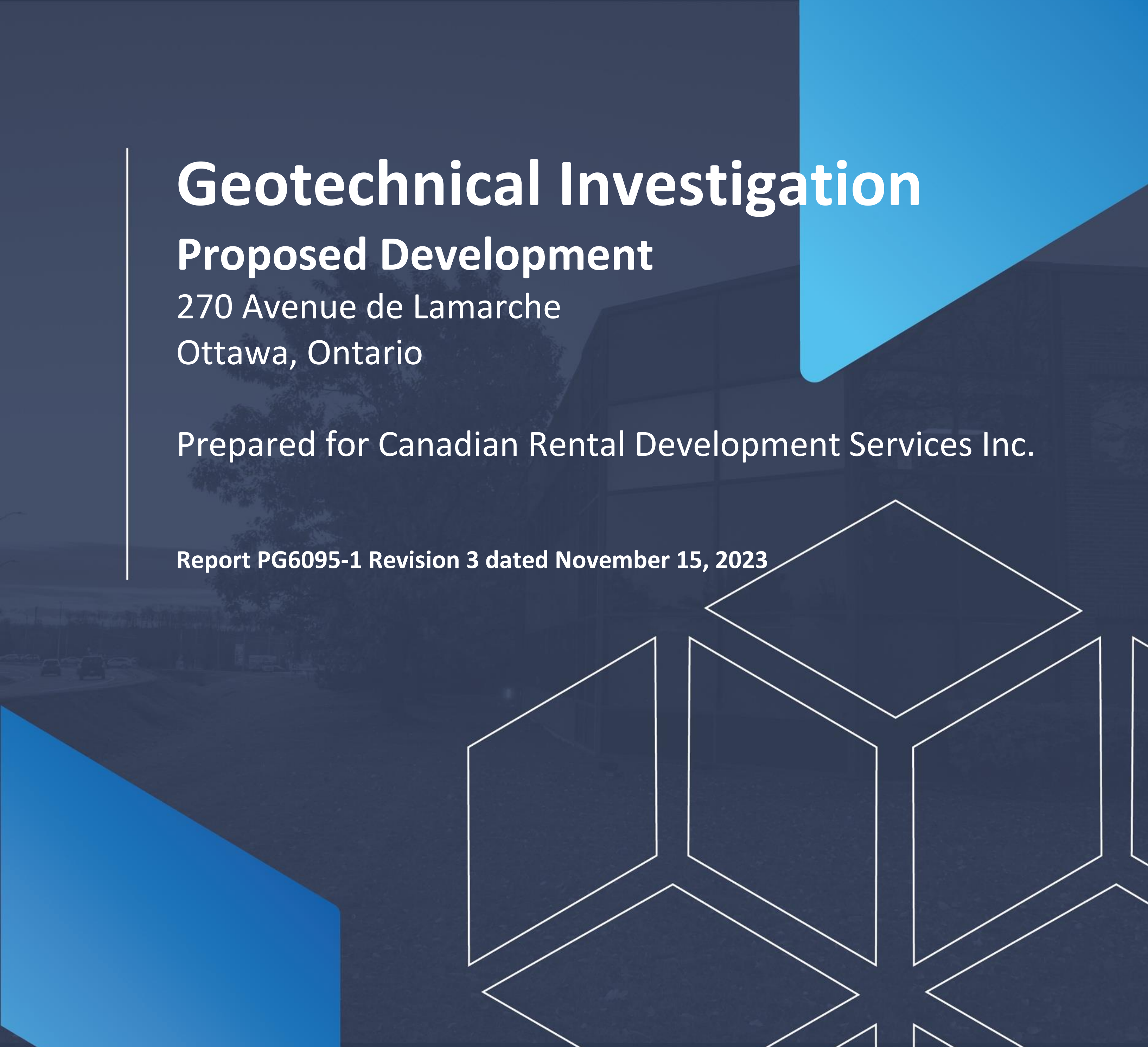

Geotechnical Investigation

Proposed Development

270 Avenue de Lamarche
Ottawa, Ontario

Prepared for Canadian Rental Development Services Inc.

Report PG6095-1 Revision 3 dated November 15, 2023

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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 residential development to be located at 270 Avenue de LaMarche 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 three (3) multi-storey residential buildings, having 6 to 7 floors, and sharing two (2) underground parking levels. Associated access lanes, at-grade parking and 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 between December 14 and 17, 2021 and consisted of advancing a total of 13 boreholes to a maximum depth of 11.8 m below existing ground surface. The test hole locations were determined by the client, taking into consideration underground utilities and site features. A supplemental geotechnical investigation program for bedrock delineation was completed on August 24, 2023. At that time, 19 probe holes were advanced to a maximum depth of 11.6 m below existing grade to delineate the bedrock surface across the subject site. Previous geotechnical investigations were completed by Paterson and others within the subject site and its neighbouring sites in 2016 and 2018. At that time, 1 borehole and 18 probe holes were located within the current project area and were advanced to a maximum depth of 8.63 m or refusal over bedrock surface. The test hole locations are shown on Drawing PG6095-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance drill rig operated by a two-person crew. The probe holes were completed using a track mounted air-track 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 core samples were recovered from eight boreholes drilled during the current investigation (BH 1-21 through BH 8-21) 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 5-21 and BH 13-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. 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.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 3.0 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.

3.2 Field Survey

The test hole locations for the current investigation were selected by the client, 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 PG6095-1 - 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. A total of 3 Atterberg limits tests, 2 grain size distribution analyses, 1 shrinkage test and moisture content testing were completed on selected soil samples.

All test results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limit's Results and Shrinkage Test Results sheets presented in Appendix 1.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject is currently vacant and grass covered. The ground surface across the subject site is generally flat with a slight downward slope toward the south and east. The east portion of the site was observed to be approximately at grade with Avenue de LaMarche.

The subject site is bordered to the north by vacant land, commercial buildings and Innes Road, to the east by vacant land and industrial property and to the south and west by a residential development.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil followed by a very stiff to stiff brown silty clay crust overlying a firm to stiff grey silty clay layer. A layer of compact to very dense glacial till was encountered below the above noted layers at the location of boreholes BH 1-21, BH 2-21, BH 3-21, BH 4-21, BH 8-21, BH 11-21, and BH 12-21. The glacial till deposit was found to consist of compact to dense grey silty clay with sand, gravel and cobbles. Practical refusal to augering was encountered in BH 9-21 through BH 13-21 at approximate depths between 6.0 and 7.4 m below existing ground surface. Bedrock was cored in boreholes BH 1-21 through BH 8-21 at approximate depths between 4.5 and 10.0 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.

Bedrock

Based on available geological mapping, the bedrock in this area consists of limestone and shale of the Lindsay Formation with an overburden drift thickness of 5 to 7 m depth.

Grain Size Distribution and Hydrometer Testing

Two sieve analyses were 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 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-21	SS3	0.0	0.5	53.5	46.0
BH 10-21	SS2	0.0	1.8	39.2	59.0

Atterberg Limit Tests

Three selected silty clay samples were submitted for Atterberg Limit testing. The test results indicate that high plasticity silty clays are anticipated at the subject site. The results are summarized in Table 2 and presented in Appendix 1.

Table 2 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 2-21 SS2	0.8 – 1.4	66	32	34	CH
BH 4-21 SS2	0.8 – 1.4	63	31	32	CH
BH 9-21 SS3	1.5 – 2.1	67	30	37	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index;
CH: Inorganic Clay of High Plasticity MH: Inorganic Silt of High Plasticity

Shrinkage Test

The results of the shrinkage limit test indicate a shrinkage limit of 23.8% and a shrinkage ratio of 1.64.

4.3 Groundwater

The groundwater levels were recorded within the monitoring wells and piezometers installed within the boreholes during the current investigation on December 24, 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 – Current Investigation

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	88.59	1.01	87.58	December 24, 2021
BH 2-21	88.56	0.42	88.14	
BH 3-21	88.81	0.91	87.90	
BH 4-21	88.84	1.24	87.60	
BH 5-21	88.54	1.75	86.79	
BH 6-21	88.55	1.85	86.70	
BH 7-21	88.52	2.06	86.46	
BH 8-21	88.40	1.60	86.80	
BH 9-21	88.81	0.54	88.27	
BH 10-21	88.50	0.76	87.74	
BH 11-21	88.77	1.56	87.21	
BH 12-21	88.46	1.38	87.08	
BH 13-21	88.62	1.82	86.80	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to a geodetic datum.

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 colour and consistency of the recovered soil samples. Based on these observations, it is estimated that long-term groundwater level can be expected between 2 to 3 m below existing ground surface. However, groundwater levels are subject to seasonal fluctuations and therefore could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. 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 undisturbed, compact to dense glacial till, and/or a clean, surface sounded bedrock bearing surface.

☐ Conventional footings placed on vertical, zero entry lean concrete in-filled trenches extended to the underlaying clean, surface-sounded bedrock 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.

For buildings founded directly over the silty clay deposit, a permissible grade raise restriction will be required. A permissible grade raise restriction of **2 m** is recommended for the site.

Where bedrock removal is required, consideration should be given to hoe-ramming or controlled blasting. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

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

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 buildings, all overburden material will be excavated from within the proposed underground parking structure footprint. It is anticipated that bedrock removal will be required for portions of the underground parking structure.

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, or construction debris/remnants should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the 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 proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

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 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 excavated almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipment. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines.

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

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 development depending on the design loading and foundation depth. The following foundation options are recommended:

Bearing Resistance Values (Conventional Shallow Foundation)

Bedrock Medium

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at ULS of **2,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Alternatively, footings placed over zero entry, near vertical trenches extending to bedrock and in-filled with lean concrete (15 MPa) to underside of footing level can be designed using the values provided above. It is recommended that the trench sidewalls extend at least 300 mm beyond the outside face of the 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.

Footings bearing on surface sounded bedrock and designed using the above noted bearing resistance values will be subjected to negligible post-construction total and differential settlements.

Overburden

Isolated shallow footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was incorporated into the above noted bearing resistance values at ULS.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, very stiff to stiff silty clay crust can be designed using the 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**.

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, in the dry, prior to the placement of concrete for footings.

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.

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 an undisturbed soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V, passing through in situ soil of the same or higher capacity as the bearing medium soil.

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.

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.

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**15 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, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. 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 geotechnical engineer, 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 **2,000 kPa**.

Deep Foundation - End Bearing Piles (Driven piles)

A deep foundation method, such as end bearing piles, can be considered where supplemental axial resistance is required for structural design for the proposed development. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable factored pile resistance at ULS values is provided in Table 4. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 4. 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 4 - End Bearing Pile (Driven piles) Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1100	10	32
245	11	1300	10	40
245	13	1500	10	45
324	9	1600	10	49
324	11	1850	10	58
324	13	2100	10	67

Deep Foundation - End Bearing Micro- Caissons

End bearing drilled in placed cased caisson (micropiles) can be used where supplemental axial resistance is required for structural design for the proposed building. The caisson should be installed by drilling a steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the piles. The steel casing should remain in place as part of the pile structure.

Two conditions for micropiles are applicable for this site. The first alternative is a pile installed on the sound bedrock augering through the weathered bedrock (end bearing). The compressive resistance for such piles is directly related to the compressive strength of the bedrock.

The second alternative is a concrete pile socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced. Table 5 provides preliminary micropile design capacities. Finale design will be the responsibility of the specialised foundation contractor.

Table 5 - End Bearing Micro- Caissons Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance
		Factored at ULS (kN)
245	9	1,250
245	11	1,400
245	13	1,550
342	9	2,000
342	11	2,200
342	13	2,400

*Central reinforcement required

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter.

Prior to the commencement of production piles the contractor should be ready to encountered and drill through boulders. One sacrificial test should be conducted on one pile. The test should be conducted in both compression and tension if tension loading is expected on the piles. Otherwise, compression testing only is acceptable. It is recommended to proof test 5% of the production piles under compression.

Buildings founded on piles installed to refusal in the bedrock will have negligible post-construction settlement.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the caissons, where utilized. Unit weights of materials are provided in Table 6.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted above, caissons would be located below the groundwater level, so the submerged, or effective, weight of the caisson will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

Should the caisson uplift resistance capacities be insufficient for the foundation uplift loads, rock anchors should be utilized. This is discussed further in Section 5.7. A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 6 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Friction Angle (φ')	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_a	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22	13.5	40	0.50	0.22	0.36	4.58
OPSS Granular B, Type II (Well Graded Sand-Gravel)	21.5	13.5	36	0.46	0.26	0.41	3.85
In Situ Silty Clay	18	11.2	33	0.40	0.30	0.46	3.45
Glacial Till	22	13.5	35	0.43	0.27	0.42	3.70

Notes: Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.

The earth pressure coefficients provided are for horizontal backfill profile.

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.

Buildings founded on piles installed to refusal in the bedrock will have negligible post-construction settlement.

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012 (OBC 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 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG6095-1 - 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 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 were 15, 3 and 2 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. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, 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 the test results, the average overburden seismic shear wave velocity was found to be **275 m/s** and the bedrock shear wave velocity was **2,975 m/s**. The V_{s30} was calculated using the standard equation for average shear wave velocity from the Ontario Building as presented below.

Site Class for Footings within 3 m of Bedrock Surface

For conventional footings within 3 m of bedrock surface, the V_{s30} was calculated as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{275\ m/s} + \frac{27\ m}{2{,}975\ m/s}\right)}$$

$$V_{s30=}\ 1{,}501\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **1,501 m/s** for conventional footings founding within 3 m of the bedrock surface. Therefore, a **Site Class A** is applicable for the design of proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. Soils underlying the subject site are not susceptible to liquefaction.

Site Class for Footings Greater than 3 m Above Bedrock Surface

For conventional footings with more than 3 m of softer material between the rock and underside of footing, the V_{s30} was calculated as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{4\ m}{275\ m/s} + \frac{26\ m}{2{,}975\ m/s}\right)}$$

$$V_{s30=}\ 1{,}288\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **1,288 m/s** for conventional footings founded more than 3 m above the bedrock surface. Therefore, a **Site Class C** is applicable for the design of the proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. 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 existing soil and bedrock surface, which is reviewed and approved by Paterson personnel at the time of construction, will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone. Alternatively, excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. Pipe spacing requirements should be determined at the time of excavation when the groundwater infiltration can be better assessed.

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/m3.

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.

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 0.30 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 Rock Anchor Design

The geotechnical design of grouted rock anchors in limestone bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the individual anchor load capacity.

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

Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and a free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 75 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 47** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively.

Recommended Grouted Rock Anchor Lengths

Parameters used to calculate grouted rock anchor lengths are provided in Table 7.

Table 7 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	47 MPa m=0.128 and s=0.00009
Unconfined compressive strength - Limestone	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 8. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 8 - 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	1.9	1.9	3.8	450
	2.6	2.0	4.6	600
	3.2	2.2	5.4	750
	4.0	2.5	6.5	900
150	1.0	1.8	2.8	450
	1.3	2.0	3.3	600
	1.6	2.3	3.9	750
	2.0	2.5	4.5	900

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.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 9 below. The flexible pavement structure presented in Table 10 should be used for at grade access lanes and heavy loading parking areas.

Table 9 - Recommended Rigid Pavement Structure - Lower Parking Level

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

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 10 – Recommended Asphalt 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.

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

The following recommendations may be considered for the architectural design of the buildings foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future buildings (and prior to tender) to review and provide supplemental information for the buildings foundation drainage system design.

Supplemental details, review of architectural design drawings and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structures. It is expected that the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

❑ A waterproofing membrane should be placed against the shoring system between underside of footings and 2 m below existing ground surface. The height of the waterproofing layer should be confirmed on a per-building basis, however, is expected to vary between 2 and 3 m below existing ground surface. Where the membrane will extend below the bedrock surface, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the bedrock surface. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.

❑ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

❑ The bedrock face, where located within a buildings excavation, is recommended to be grinded to provide a smooth-surface for the installation of the waterproofing layer. Large cavities should be reviewed by Paterson as the excavation progresses to assess the requirement to in-fill cavities suitably to facilitate the installation of the waterproofing layer.

❑ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall.

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer's specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or 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.

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall and as depicted in Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-In Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material.

This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Foundation Raft Slab Construction Joints

Where a raft slab is being considered, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab.

Finalized Drainage and Waterproofing Design

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious 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 be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

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 unfavorable 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 11 – 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 above the cover material if the excavation and filling operations are carried out in dry weather conditions. 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 low to 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.

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

Based on observations, the groundwater level is anticipated at a 2 to 3 m depth. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the existing 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 neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring 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.

6.7 Corrosion Potential and Sulphate

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

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 of the proposed underground structure, no tree planting restrictions are required for the proposed buildings at the subject site.

7.0 Recommendations

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

☐ Review preliminary and detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction, if applicable.

☐ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

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:

☐ Review and inspection of the installation of the foundation drainage systems.

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

☐ Observation of driving and re-striking of all pile foundations.

☐ 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 by the geotechnical consultant.

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 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
Nov.15, 2023
D. J. GILBERT
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PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TEST HOLE LOGS BY OTHERS

ANALYTICAL TEST RESULTS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 270 LaMarche Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

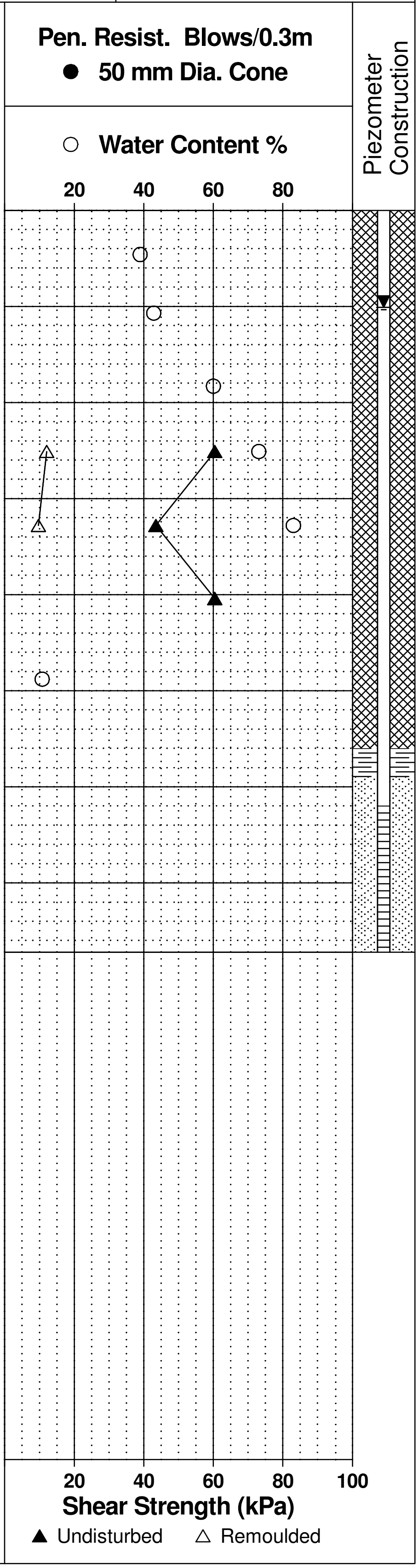
BORINGS BY CME-55 Low Clearance Drill

DATE December 14, 2021

FILE NO. PG6095

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	88.59
TOPSOIL 0.28						
	AU	1				
	SS	2	100	13	1	87.59
	SS	3	100	8		
Very stiff, brown **SILTY CLAY**					2	86.59
- firm to stiff and grey by 2.9m depth					3	85.59
4.42					4	84.59
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders 4.93	SS	4	67	50+		
					5	83.59
	RC	1	100	46		
BEDROCK: Poor to good quality, grey limestone					6	82.59
	RC	2	100	75	7	81.59
7.72						
End of Borehole (GWL @ 1.01m - Dec. 24, 2021)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 270 LaMarche Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

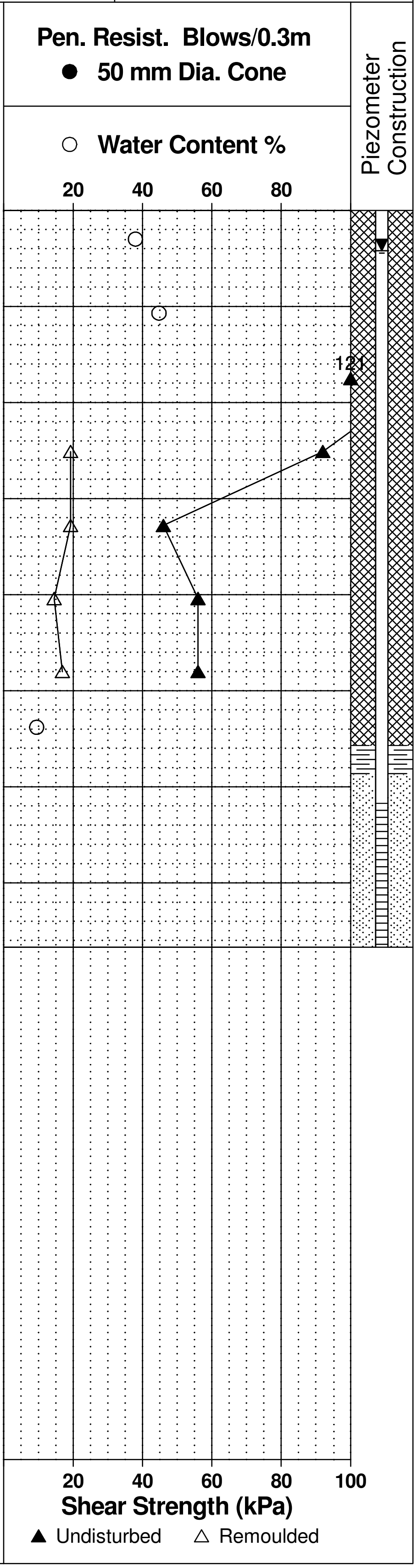
BORINGS BY CME-55 Low Clearance Drill

DATE December 14, 2021

FILE NO. **PG6095**

HOLE NO. **BH 2-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface							
TOPSOIL 0.25		AU	1			0	88.56
		SS	2	100	12	1	87.56
						2	86.56
Very stiff to stiff, brown **SILTY CLAY**							
- firm to stiff and grey by 3.0m depth						3	85.56
						4	84.56
5.18						5	83.56
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders 5.49		SS	3	0	50+		
		RC	1	100	88	6	82.56
BEDROCK: Good to excellent quality, grey limestone		RC	2	100	100	7	81.56
7.67							
End of Borehole							
(GWL @ 0.42m - Dec. 24, 2021)							



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

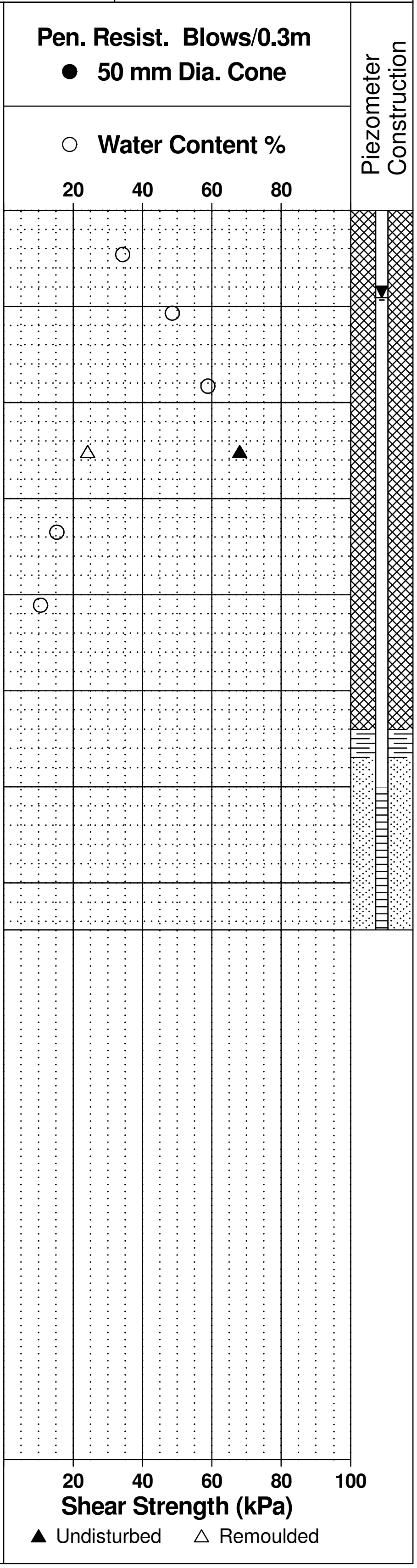
BORINGS BY CME-55 Low Clearance Drill

DATE December 14, 2021

FILE NO. PG6095

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	88.81
TOPSOIL 0.30						
Very stiff to stiff, brown **SILTY CLAY**	AU	1				
	SS	2	100	12	1	87.81
	SS	3	100	8	2	86.81
3.05						
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	SS	4	67	41	3	85.81
	SS	5	42	22	4	84.81
4.47	SS	6	0	50+		
BEDROCK: Excellent to fair quality, grey limestone	RC	1	100	93	5	83.81
					6	82.81
	RC	2	100	67	7	81.81
7.49						
End of Borehole						
(GWL @ 0.91m - Dec. 24, 2021)						



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SOIL PROFILE AND TEST DATA

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REMARKS

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DATE December 14, 2021

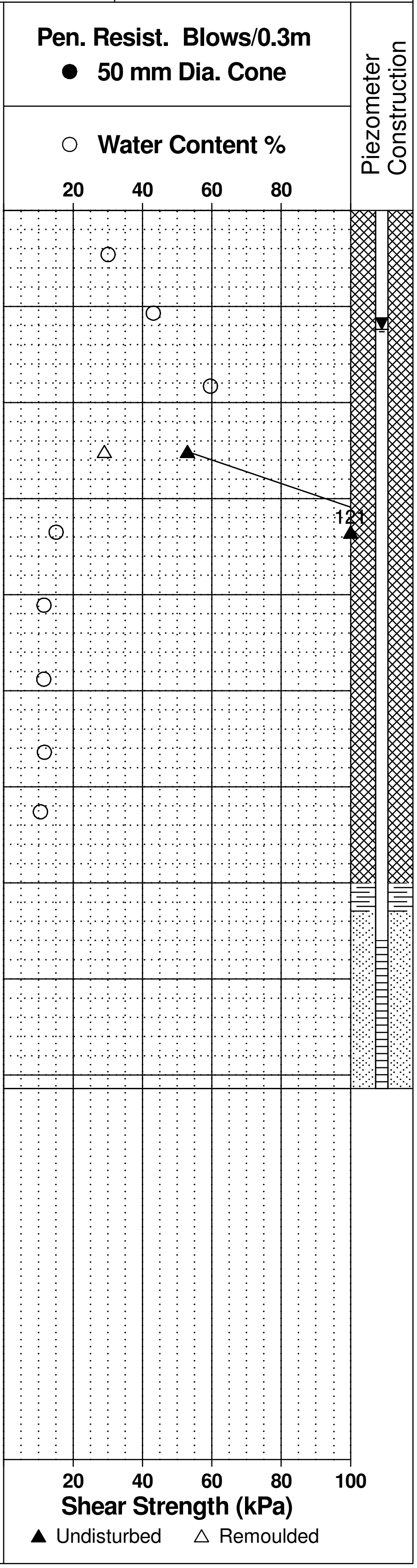
FILE NO. **PG6095**

HOLE NO. **BH 4-21**

SOIL DESCRIPTION	Bottom depth (m)
Ground Surface	
TOPSOIL	0.28
Stiff to very stiff, brown **SILTY CLAY** - grey by 2.9m depth	3.20
GLACIAL TILL: Grey silty clay with sand, trace gravel - clay content decreasing with depth	5.60
GLACIAL TILL: Compact, grey silty sand with clay, gravel, cobbles and boulders	6.43
BEDROCK: Fair to excellent quality, grey limestone - 12mm thick mud seam at 7.0m depth	9.14
End of Borehole (GWL @ 1.24m - Dec. 24, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	12
SS	3	100	6
SS	4		P
SS	5	25	6
SS	6	100	4
SS	7	75	12
SS	8	46	50+
RC	1	100	56
RC	2	100	100

DEPTH (m)	ELEV. (m)
0	88.84
1	87.84
2	86.84
3	85.84
4	84.84
5	83.84
6	82.84
7	81.84
8	80.84
9	79.84



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 15, 2021

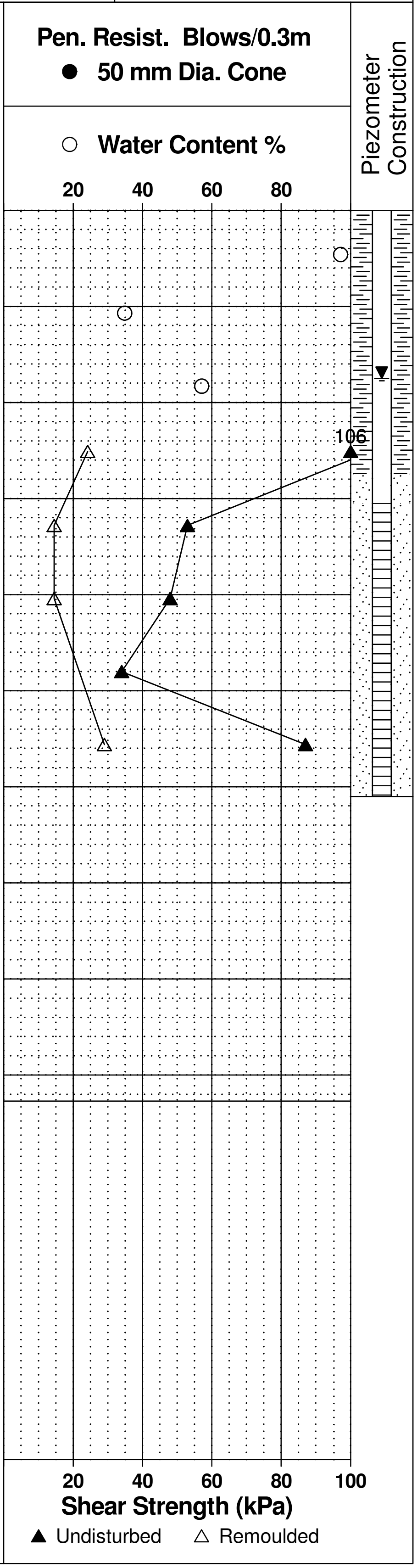
FILE NO. PG6095

HOLE NO. BH 5-21

SOIL DESCRIPTION	Depth of base (m)
Ground Surface	
TOPSOIL	0.25
Very stiff, brown **SILTY CLAY** - stiff to firm and grey by 2.9m depth	6.22
BEDROCK: Poor to good quality, grey limestone	9.27
End of Borehole (GWL @ 1.75m - Dec. 24, 2021)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	10
SS	3	100	8
SS	4	0	50+
RC	1	100	32
RC	2	100	75

DEPTH (m)	ELEV. (m)
0	88.54
1	87.54
2	86.54
3	85.54
4	84.54
5	83.54
6	82.54
7	81.54
8	80.54
9	79.54



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SOIL PROFILE AND TEST DATA

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REMARKS

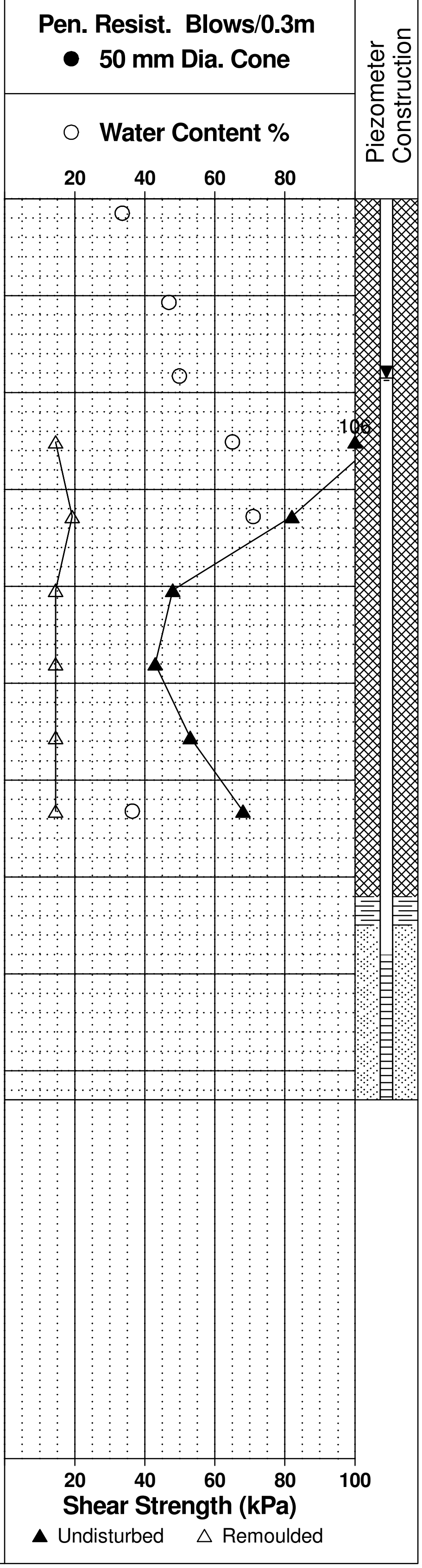
BORINGS BY CME-55 Low Clearance Drill

DATE December 15, 2021

FILE NO. **PG6095**

HOLE NO. **BH 6-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	88.55
TOPSOIL 0.25		AU	1				
		SS	2	50	11	1	87.55
		SS	3	50	7	2	86.55
Very stiff to stiff, brown **SILTY CLAY**						3	85.55
- firm to stiff and grey by 3.5m depth						4	84.55
						5	83.55
						6	82.55
- trace sand and gravel by 6.4m depth 6.65							
		RC	1	100	100	7	81.55
BEDROCK: Excellent to poor quality, grey limestone						8	80.55
		RC	2	100	43	9	79.55
9.30							
End of Borehole							
(GWL @ 1.85m - Dec. 24, 2021)							



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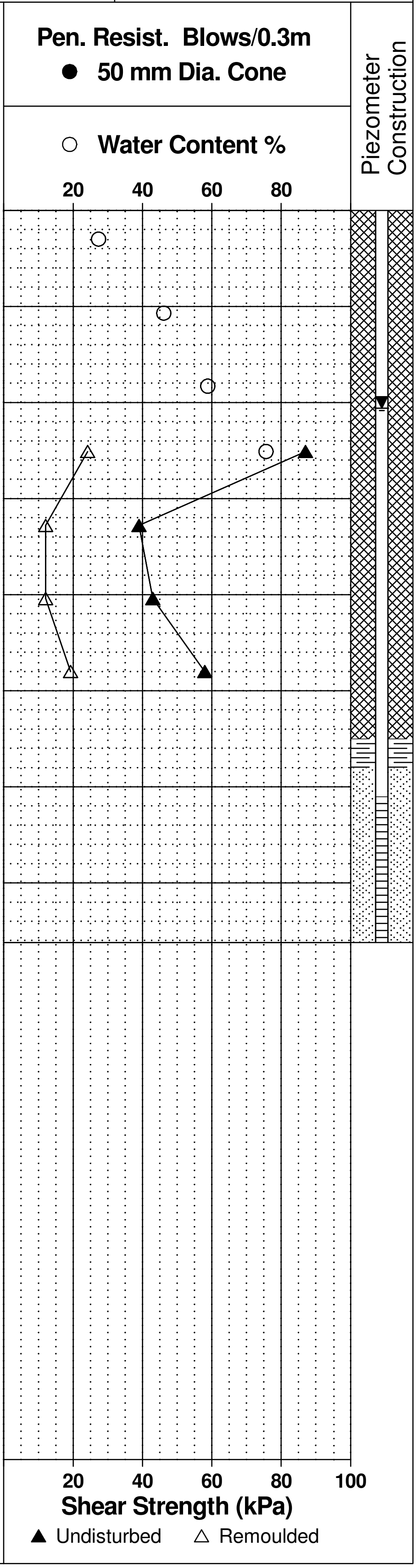
DATE December 15, 2021

FILE NO. PG6095

HOLE NO. BH 7-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	88.52
TOPSOIL	0.30	AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	67	8	1	87.52
		SS	3	100	6	2	86.52
- firm to stiff and grey by 2.9m depth						3	85.52
						4	84.52
	5.33					5	83.52
BEDROCK: Good to excellent quality, grey limestone		RC	1	100	80	6	82.52
		RC	2	100	100	7	81.52
	7.62						
End of Borehole							
(GWL @ 2.06m - Dec. 24, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 270 LaMarche Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

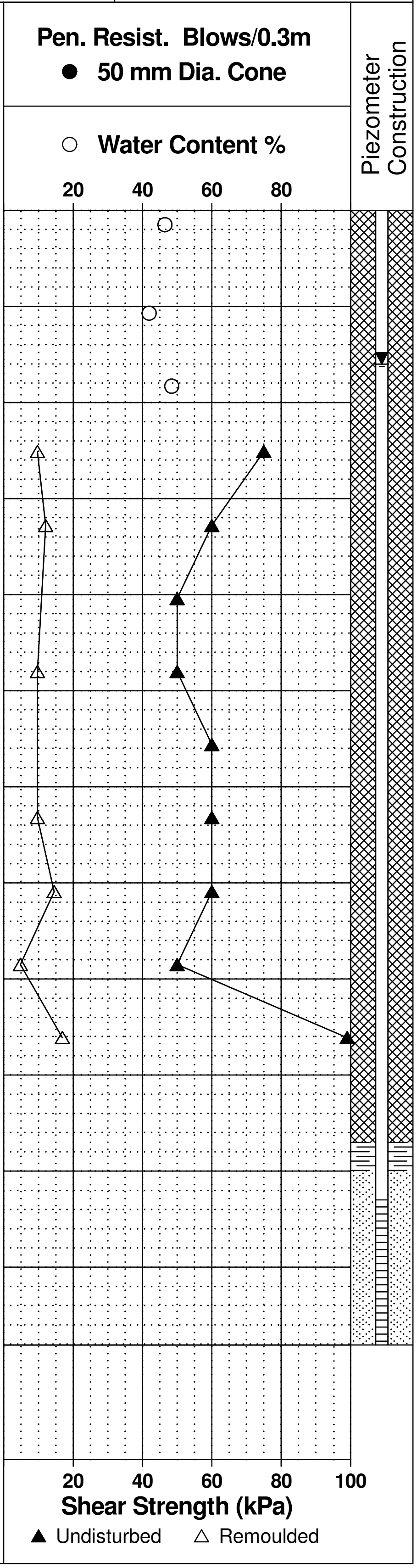
BORINGS BY CME-55 Low Clearance Drill

DATE December 15, 2021

FILE NO. PG6095

HOLE NO. BH 8-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	88.40
TOPSOIL 0.30	AU	1				
	SS	2	83	8	1	87.40
	SS	3	79	5	2	86.40
Stiff, brown **SILTY CLAY** - grey by 2.9m depth					3	85.40
					4	84.40
					5	83.40
					6	82.40
					7	81.40
					8	80.40
8.69						
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders					9	79.40
10.03	RC	1	100	100	10	78.40
BEDROCK: Excellent to poor quality, grey limestone					11	77.40
11.81	RC	2	100	45		
End of Borehole (GWL @ 1.60m - Dec. 24, 2021)						



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REMARKS

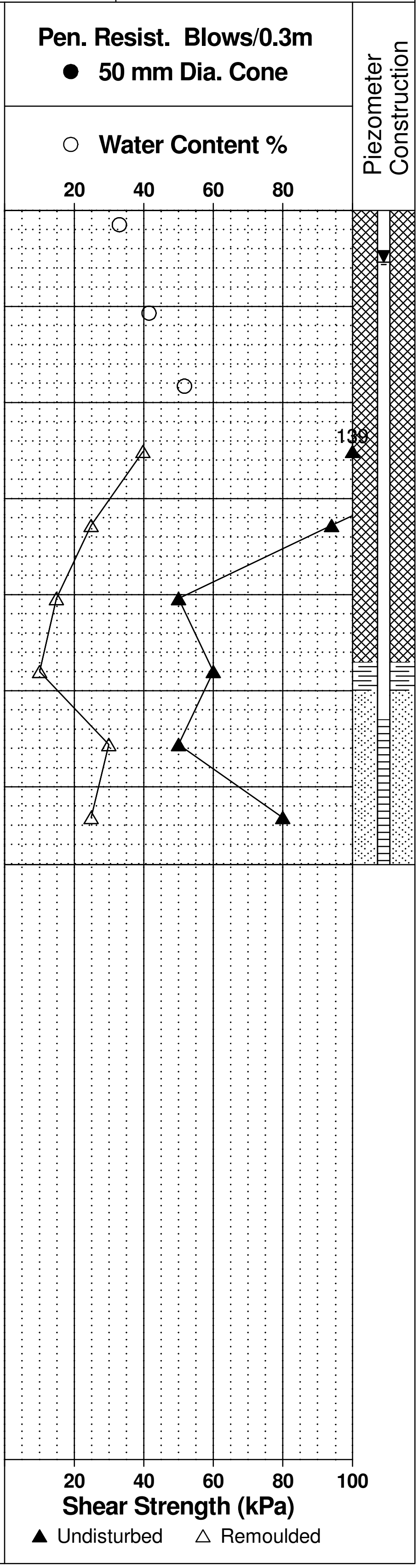
BORINGS BY CME-55 Low Clearance Drill

DATE December 16, 2021

FILE NO. **PG6095**

HOLE NO. **BH 9-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	88.81
TOPSOIL 0.30		AU	1				
		SS	2	100	11	1	87.81
		SS	3	100	10	2	86.81
Very stiff to stiff, brown **SILTY CLAY**						3	85.81
- grey by 3.7m depth						4	84.81
						5	83.81
6.81						6	82.81
End of Borehole Practical refusal to augering at 6.81m depth (GWL @ 0.54m - Dec. 24, 2021)							



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 15, 2021

FILE NO. PG6095

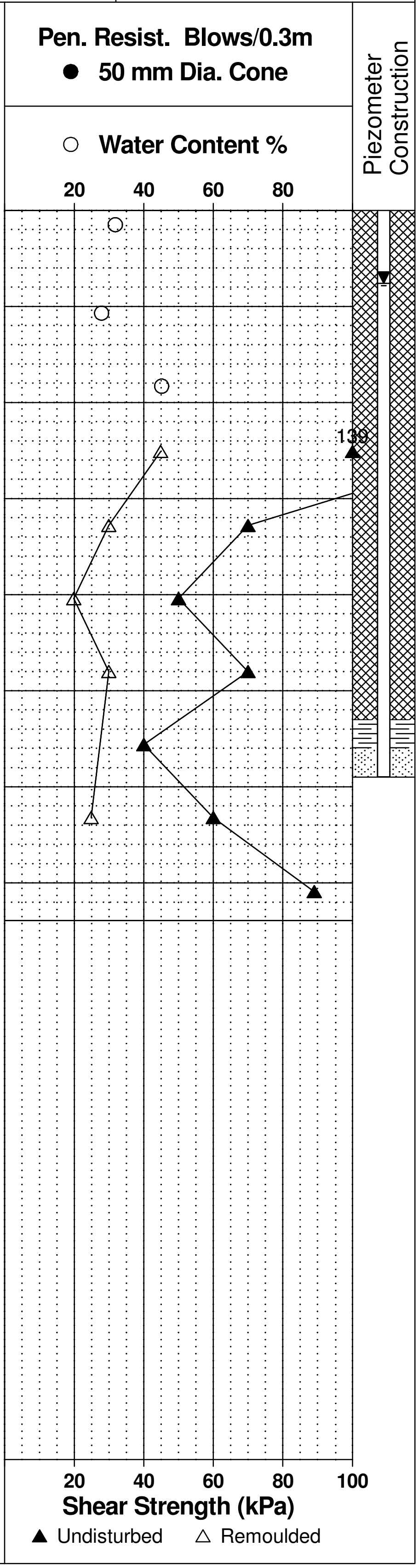
HOLE NO. BH10-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface							
TOPSOIL 0.30		AU	1			0	88.50
		SS	2	100	16	1	87.50
		SS	3	100	15		
						2	86.50
Very stiff to stiff, brown **SILTYCLAY**						3	85.50
- grey by 3.7m depth						4	84.50
						5	83.50
						6	82.50
						7	81.50
7.39							

End of Borehole

Practical refusal to augering at 7.39m depth

(GWL @ 0.76m - Dec. 24, 2021)



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 16, 2021

FILE NO. **PG6095**

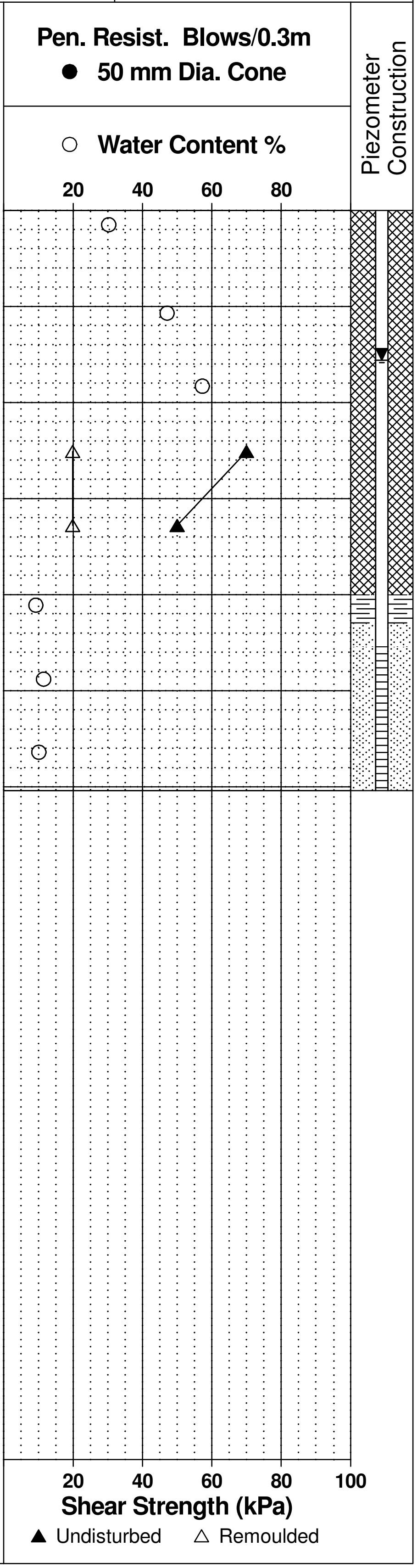
HOLE NO. **BH11-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	88.77
TOPSOIL 0.25		AU	1				
Stiff, brown **SILTY CLAY**		SS	2	100	13	1	87.77
		SS	3	100	4	2	86.77
- firm and grey by 2.9m depth						3	85.77
3.66							
GLACIAL TILL: Very stiff, grey silty clay with sand, gravel, cobbles and boulders		SS	4	25	20	4	84.77
		SS	5	42	12	5	83.77
		SS	6	50	9	6	82.77
6.04							

End of Borehole

Practical refusal to augering at 6.04m depth

(GWL @ 1.56m - Dec. 24, 2021)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 270 LaMarche Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 16, 2021

FILE NO. PG6095

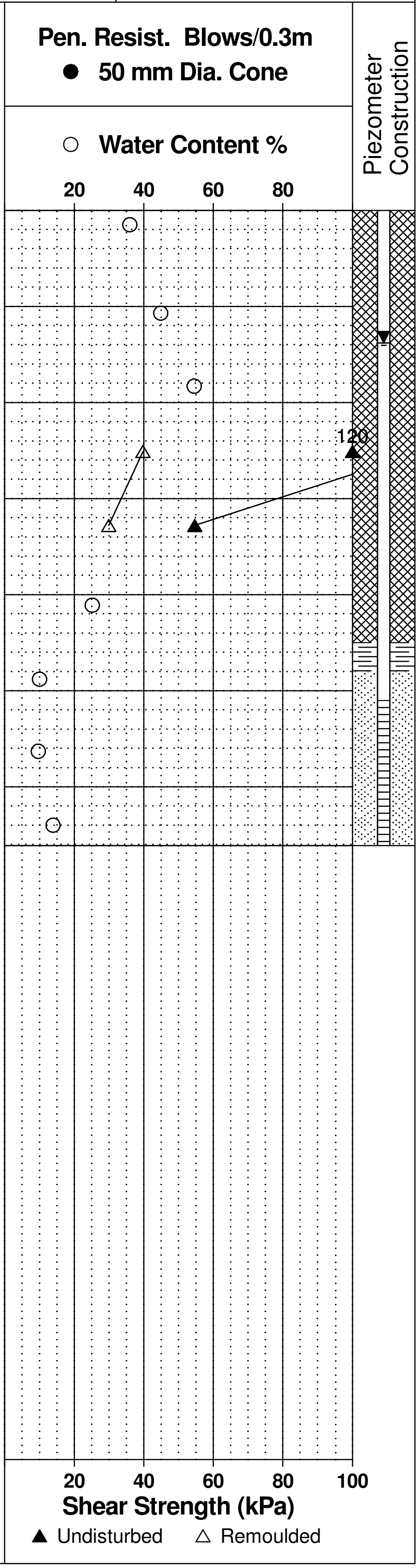
HOLE NO. BH12-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	88.46
TOPSOIL 0.15	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	100	18	1	87.46
	SS	3	100	8	2	86.46
- grey by 2.9m depth					3	85.46
4.11	SS	4	17	1	4	84.46
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	SS	5	67	5	5	83.46
	SS	6	42	5	6	82.46
6.61	SS	7		4		

End of Borehole

Practical refusal to augering at 6.61m depth

(GWL @ 1.38m - Dec. 24, 2021)



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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 17, 2021

FILE NO. PG6095

HOLE NO. BH13-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	88.62
		AU	1				
		SS	2	83	23	1	87.62
		SS	3	100	11		
						2	86.62
Very stiff to stiff, brown **SILTY CLAY**						3	85.62
- grey by 3.7m depth						4	84.62
						5	83.62
- firm to stiff by 5.2m depth						6	82.62
6.78							

End of Borehole

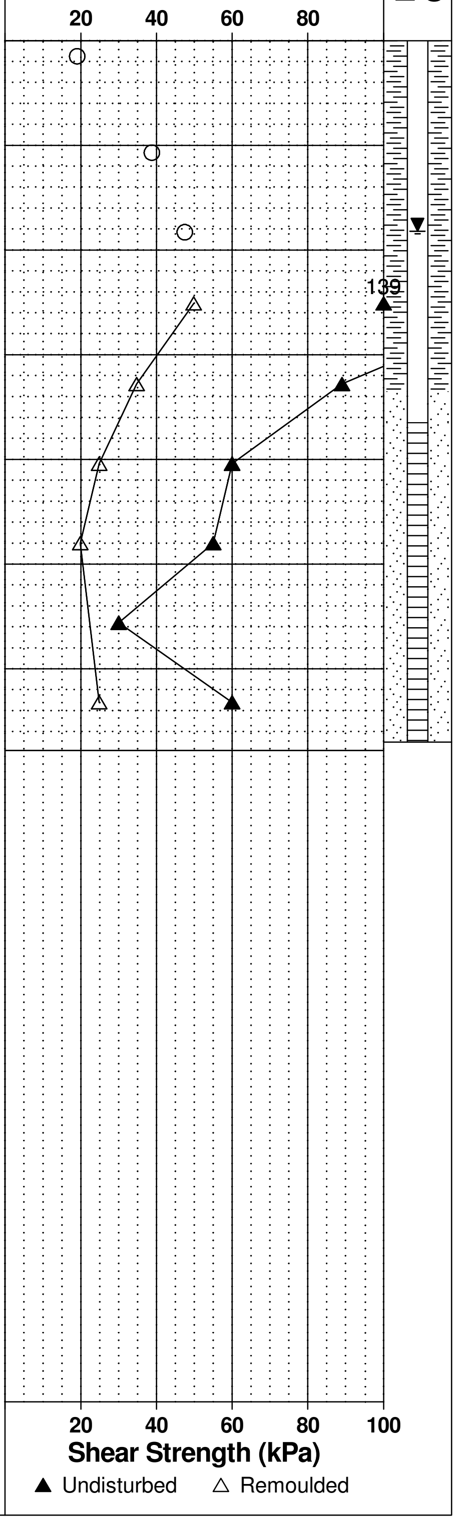
Practical refusal to augering at 6.78m depth

(GWL @ 1.82m - Dec. 24, 2021)

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

○ **Water Content %**

Piezometer Construction



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

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)

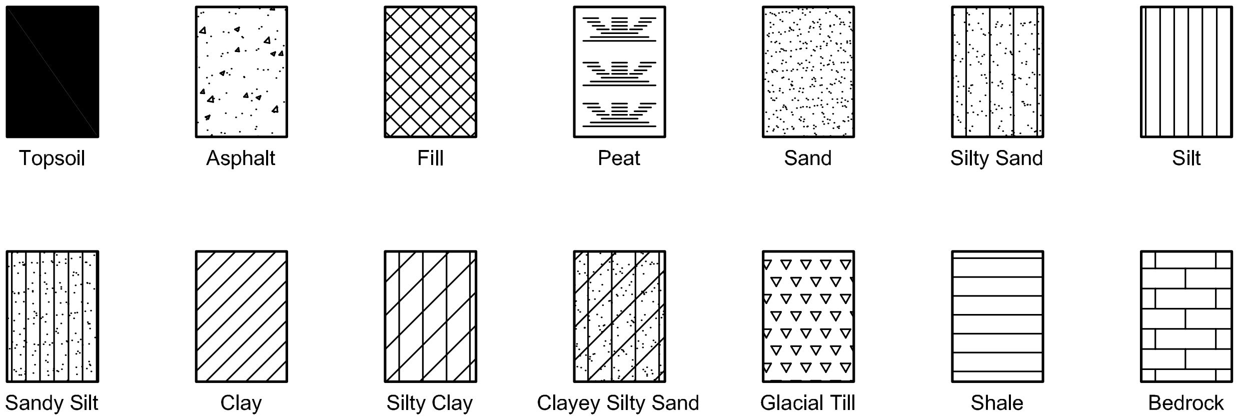
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

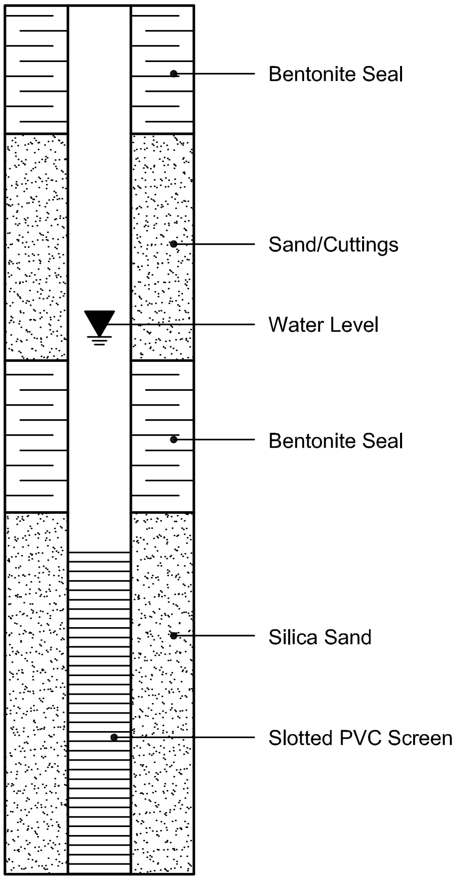
SYMBOLS AND TERMS (continued)

STRATA PLOT

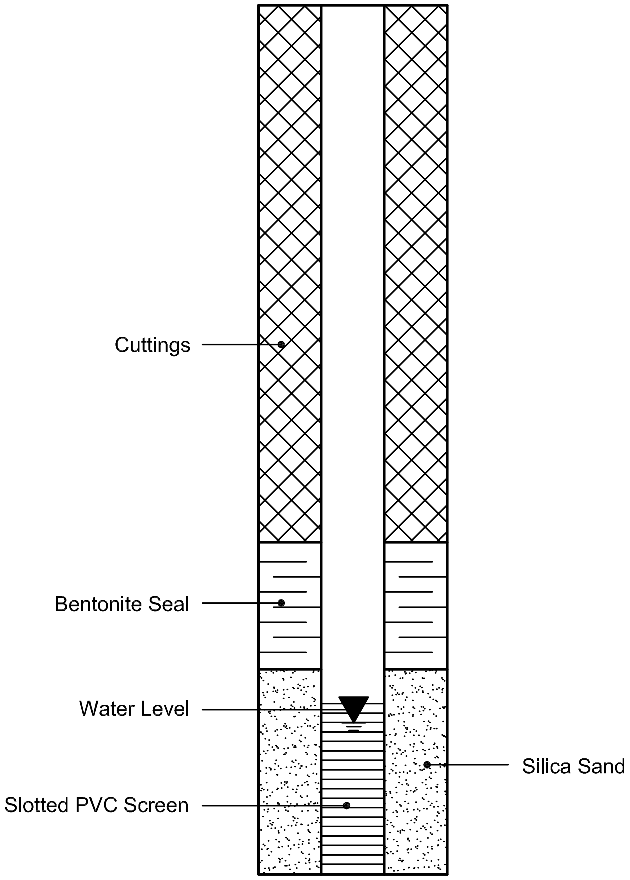


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 1660030

RECORD OF BOREHOLE: 16-6

SHEET 1 OF 1

LOCATION: N 5034371.8 ;E 381143.3

BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

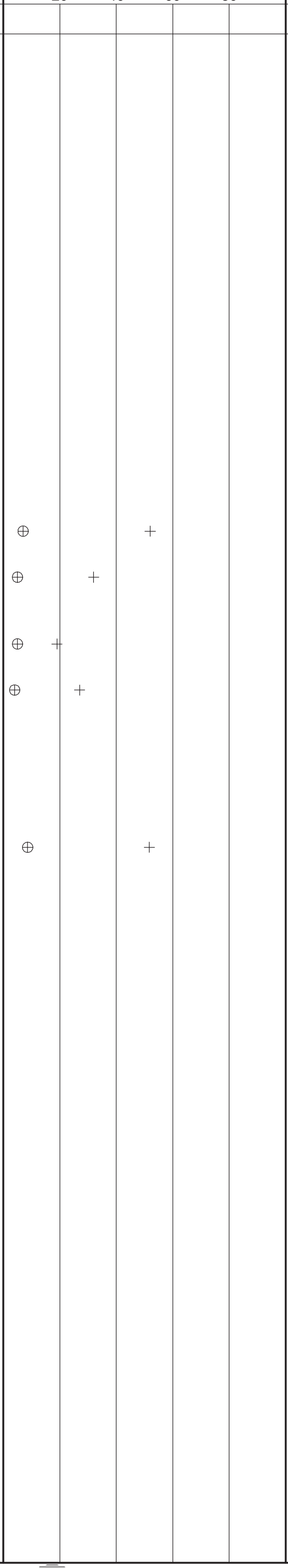
BORING METHOD: Power Auger
200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	88.72
TOPSOIL - (ML) sandy SILT; brown	0.00
(CI/CH) SILTY CLAY to CLAY; grey brown with red mottling (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	88.52 0.20
(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, soft to stiff	85.37 3.35
End of Borehole Auger Refusal	83.19 5.53

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	SS	7
2	SS	6
3	SS	5
4	SS	1
5	SS	PH

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○



HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE 1 : 50

LOGGED: DWM

CHECKED: WAM

MIS-BHS 001 1660030-GEOTECH.GPJ GAL-MIS.GDT 12/16/16 JEM

Certificate of Analysis

Client: Paterson Group Consulting Engineers

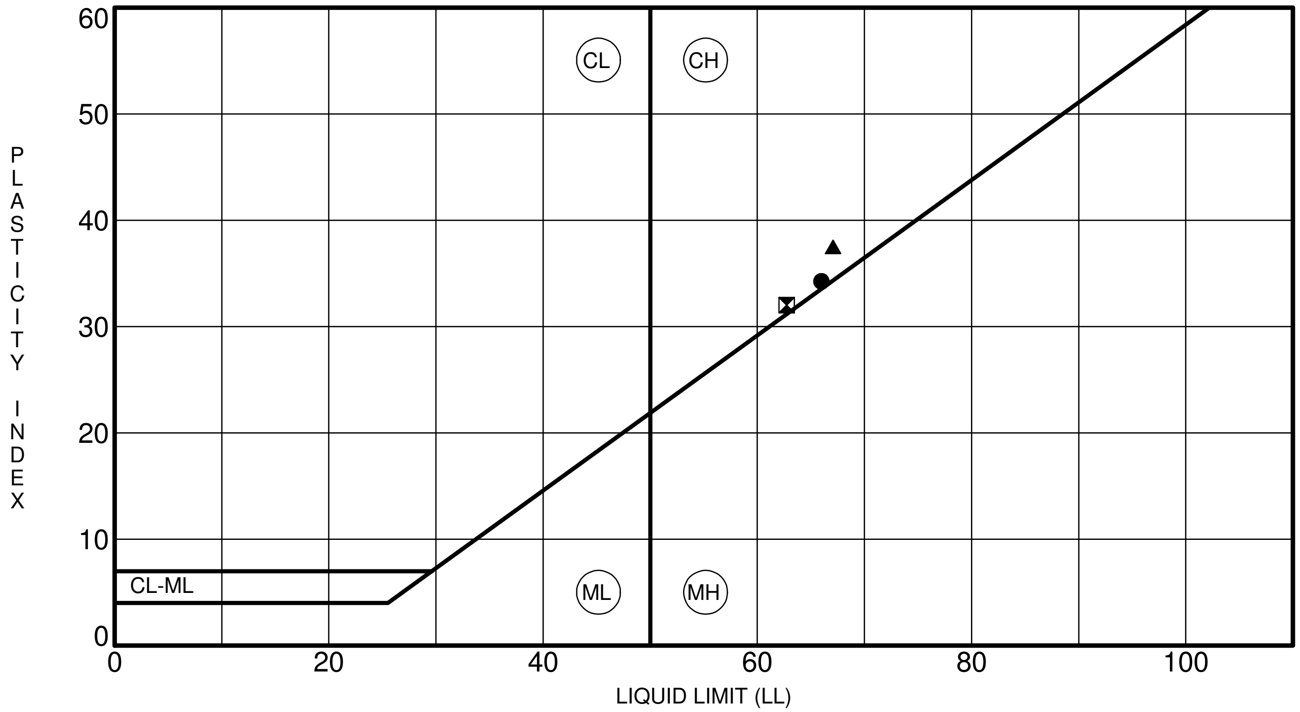
Client PO: 33408

Report Date: 04-Jan-2022

Order Date: 23-Dec-2021

Project Description: PG6095

	Client ID:	BH9-21 SS2	-	-	-
	Sample Date:	16-Dec-21 09:00	-	-	-
	Sample ID:	2152466-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	71.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.41	-	-	-
Resistivity	0.10 Ohm.m	137	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	9	-	-	-



	Specimen Identification	LL	PL	PI	Fines	Classification
●	BH 2-21 SS2	66	32	34		CH - Inorganic clays of high plasticity
⧗	BH 4-21 SS2	63	31	32		CH - Inorganic clays of high plasticity
▲	BH 9-21 SS3	67	30	37		CH - Inorganic clays of high plasticity

CLIENT **Lepine Corporation**

PROJECT **Geotechnical Investigation - Proposed Development - 270 LaMarche Avenue**

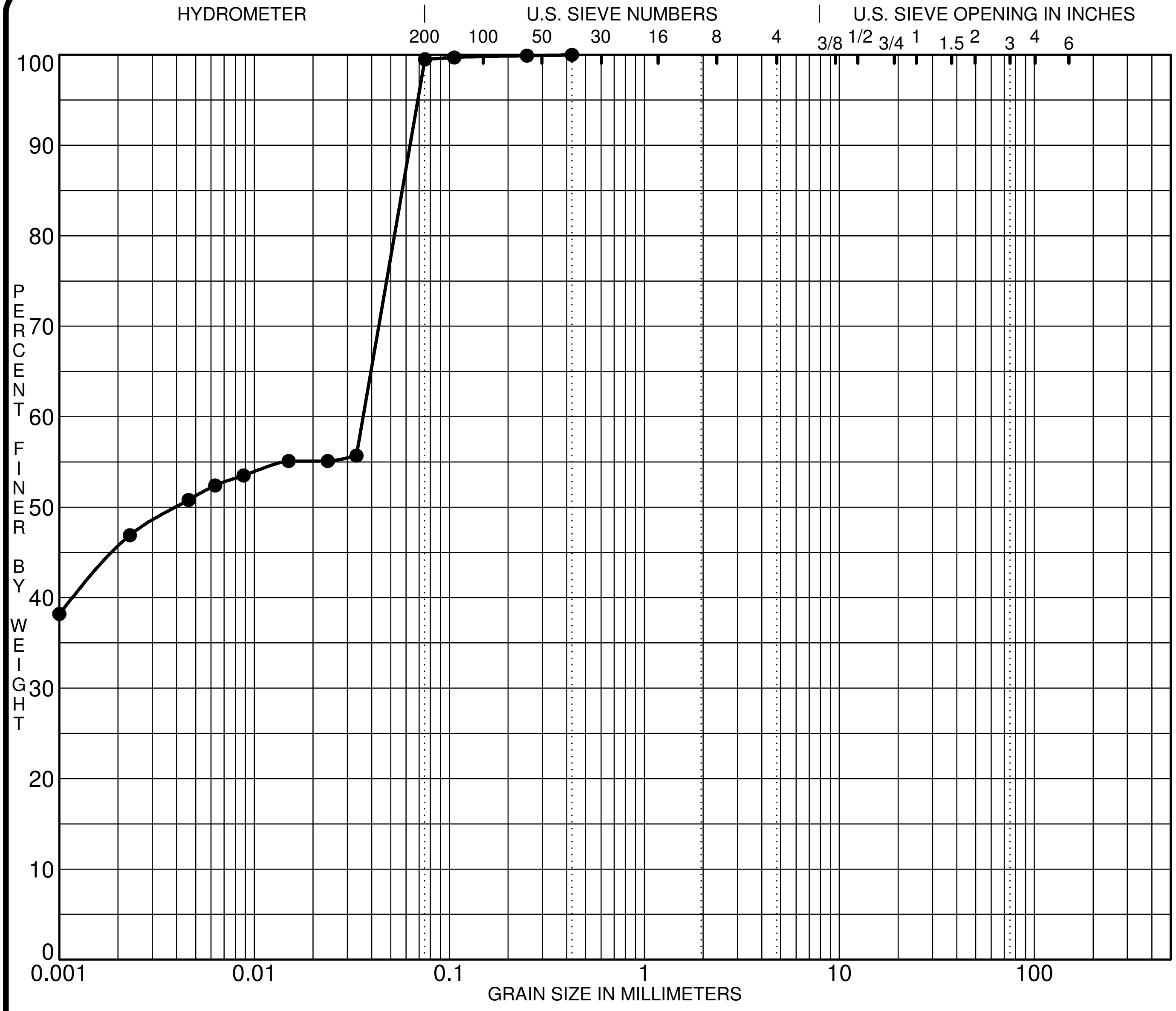
FILE NO. **PG6095**

DATE **16 Dec 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-21**	**SS3**	**CH - Inorganic clays of high plasticity**						
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-21**	**SS3**	**0.43**	**0.04**			**0.0**	**0.5**	**99.5**	
☒										
▲										
★										

CLIENT **Lepine Corporation**

FILE NO. **PG6095**

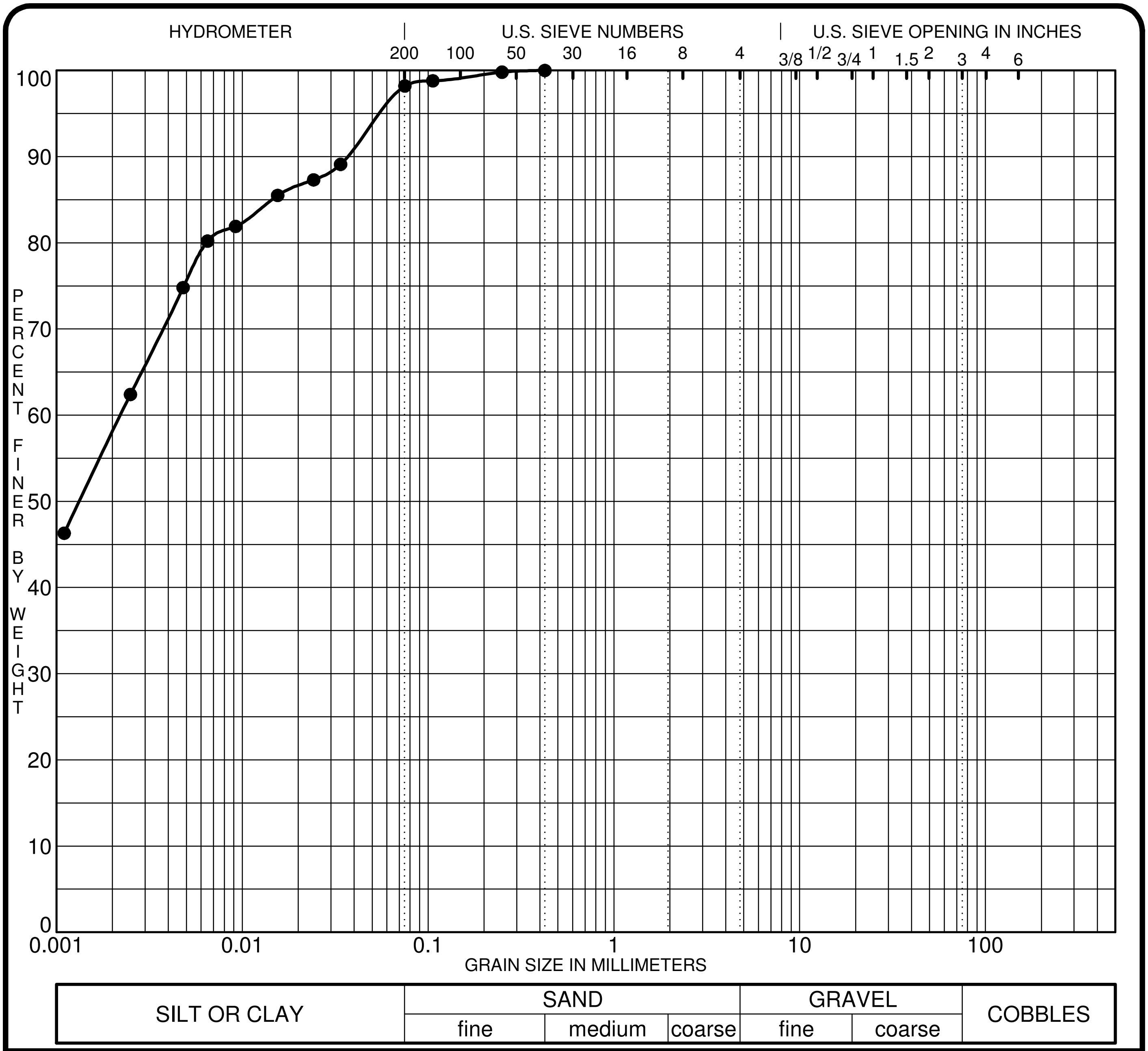
PROJECT **Geotechnical Investigation - Proposed Development - 270 LaMarche Avenue**

DATE **14 Dec 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH10-21**	**SS2**	**CH - Inorganic clays of high plasticity**						
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH10-21**	**SS2**	**0.43**	**0.00**			**0.0**	**1.8**	**98.2**	
☒										
▲										
★										

CLIENT **Lepine Corporation**

FILE NO. **PG6095**

PROJECT **Geotechnical Investigation - Proposed**
Development - 270 LaMarche Avenue

DATE **15 Dec 21**

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

GRAIN SIZE DISTRIBUTION

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – PODIUM DECK TRANSITION

DRAWING PG6095-1 – TEST HOLE LOCATION PLAN

DRAWING PG6095-2 – BEDROCK CONTOUR PLAN

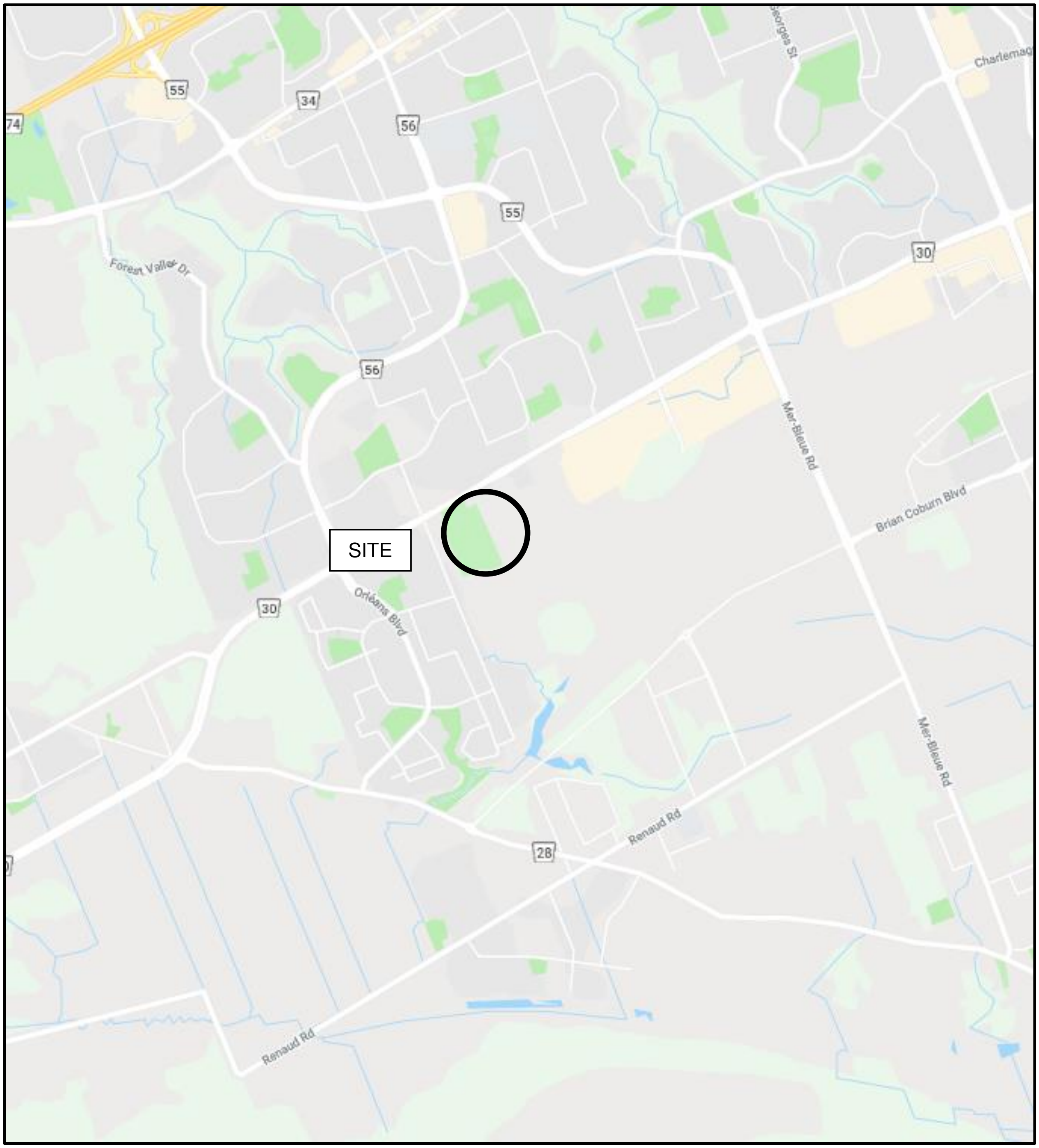
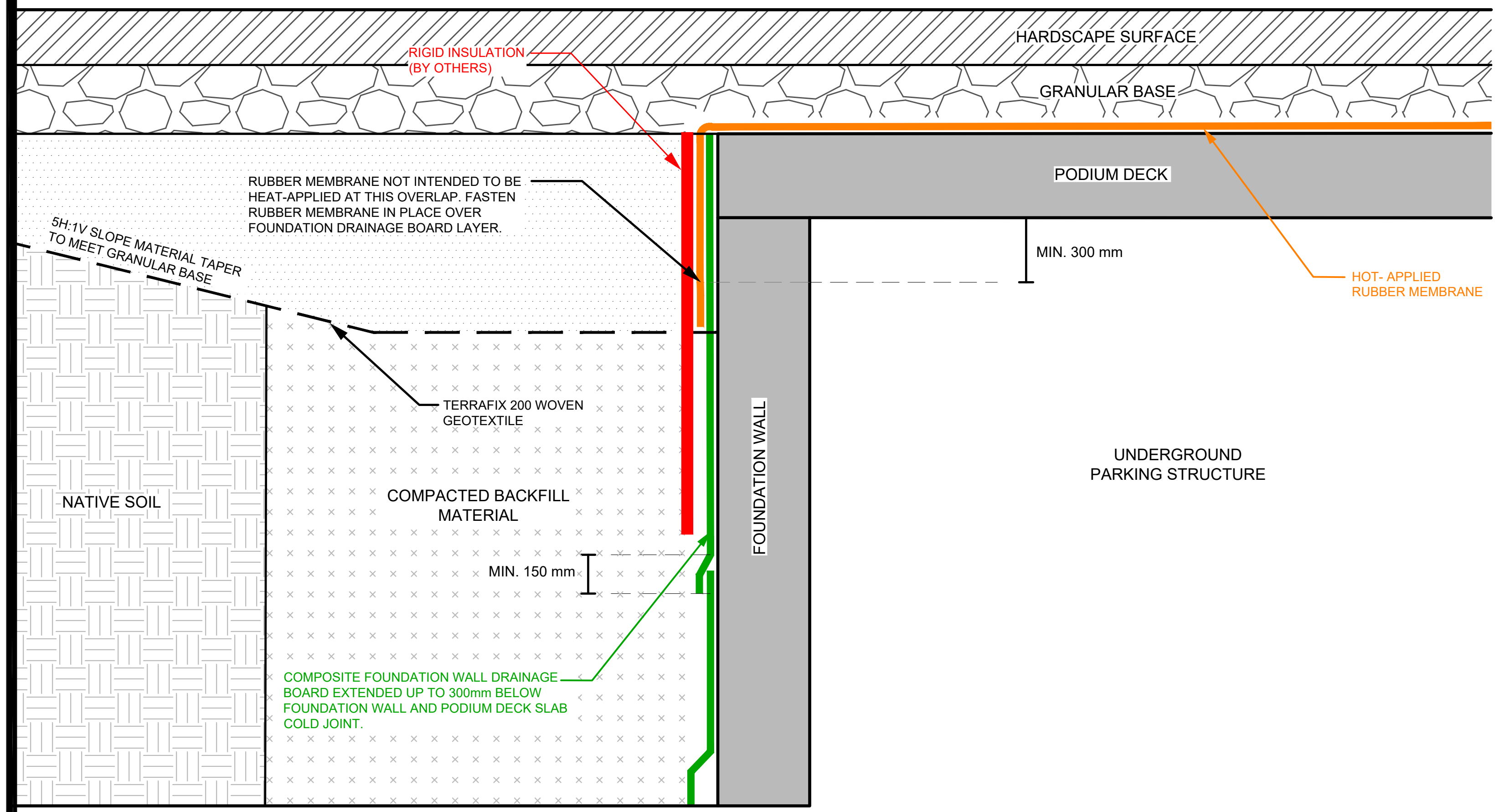


FIGURE 1

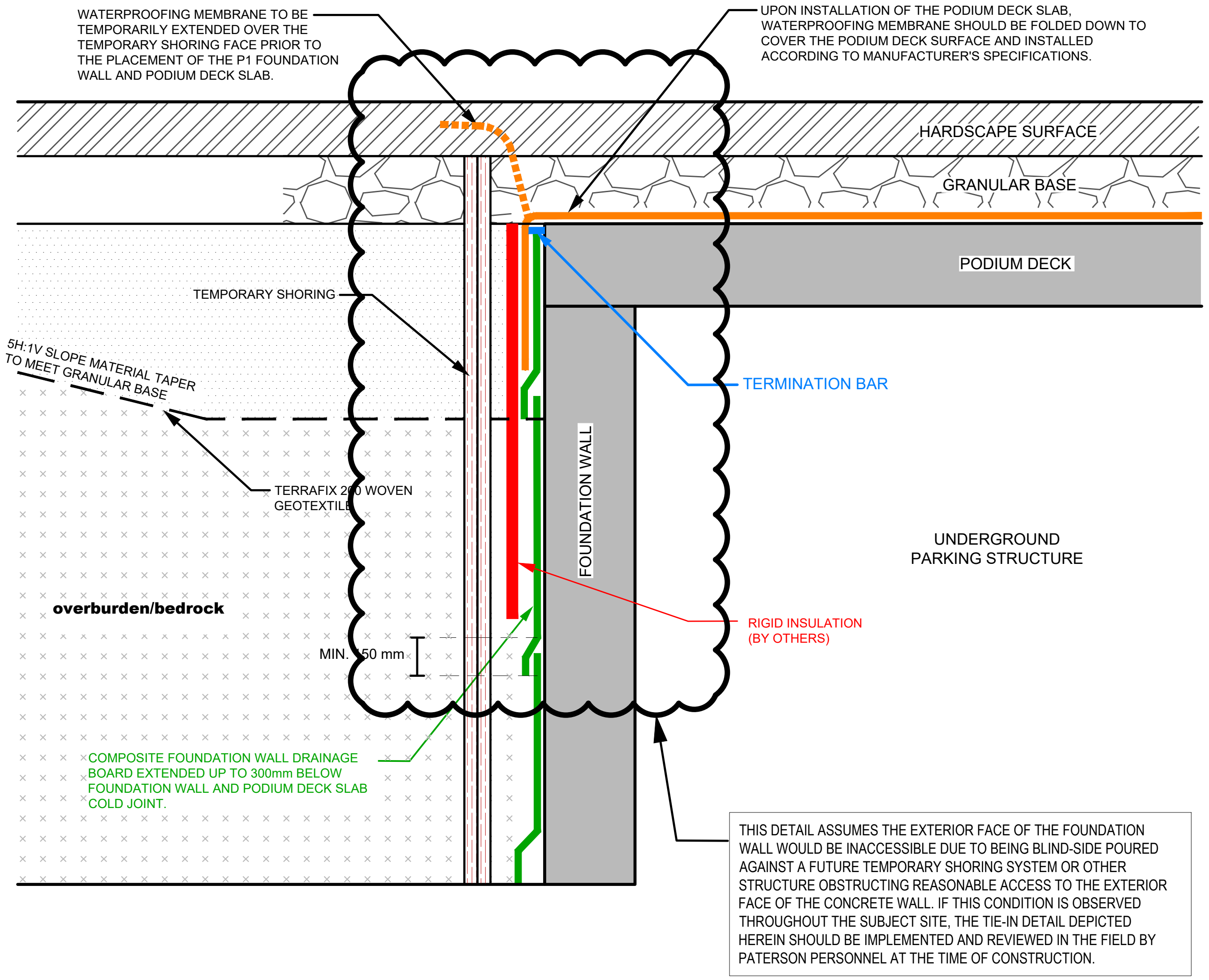
KEY PLAN

PATERSON
GROUP

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



THIS DETAIL ASSUMES THE EXTERIOR FACE OF THE FOUNDATION WALL WOULD BE INACCESSIBLE DUE TO BEING BLIND-SIDE POURED AGAINST A FUTURE TEMPORARY SHORING SYSTEM OR OTHER STRUCTURE OBSTRUCTING REASONABLE ACCESS TO THE EXTERIOR FACE OF THE CONCRETE WALL. IF THIS CONDITION IS OBSERVED THROUGHOUT THE SUBJECT SITE, THE TIE-IN DETAIL DEPICTED HEREIN SHOULD BE IMPLEMENTED AND REVIEWED IN THE FIELD BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

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

NO.	REVISIONS	DATE	INITIAL
0			

CANADIAN RENTAL DEVELOPMENT SERVICES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI STOREY BUILDINGS
OTTAWA, 270 LAMARCHE AVENUE ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: N.T.S
Date: 09/2023
Drawn by: RCG
Report No.: PG6095-1
Checked by: ZA
Dwg. No.: FIGURE 4
Approved by: MS
Revision No.: 0

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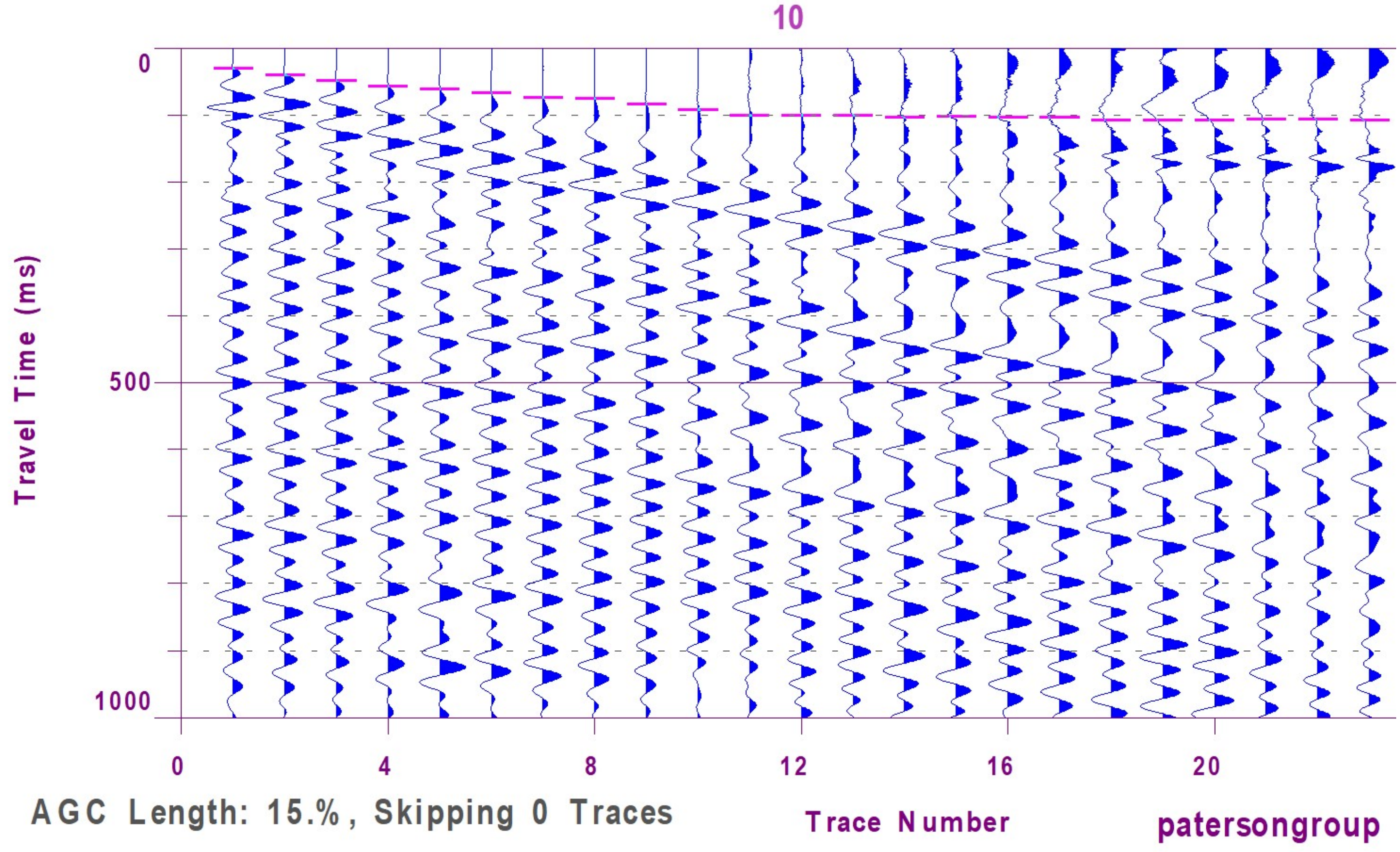


Figure 2 – Shear Wave Velocity Profile at Shot Location -2 m

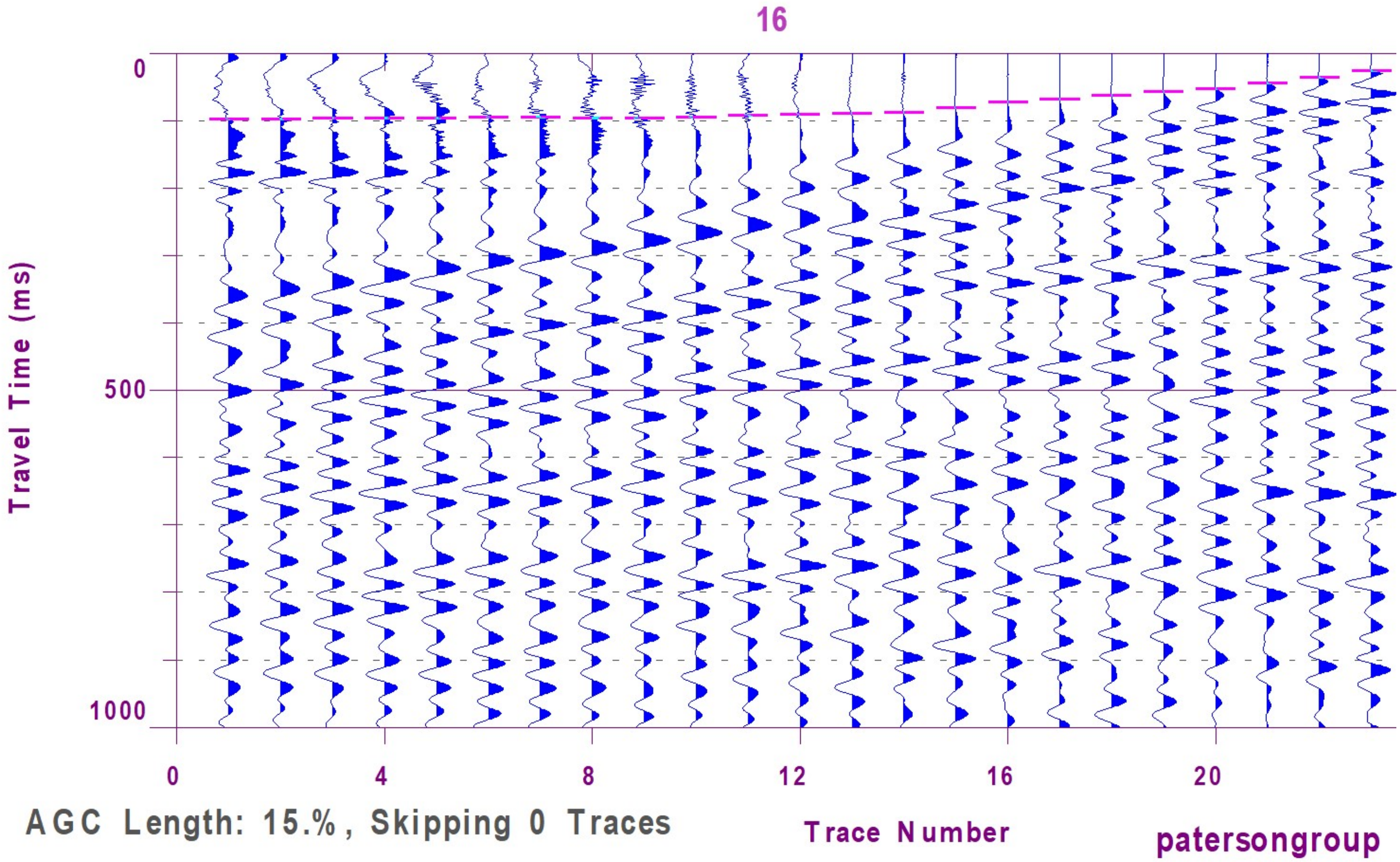
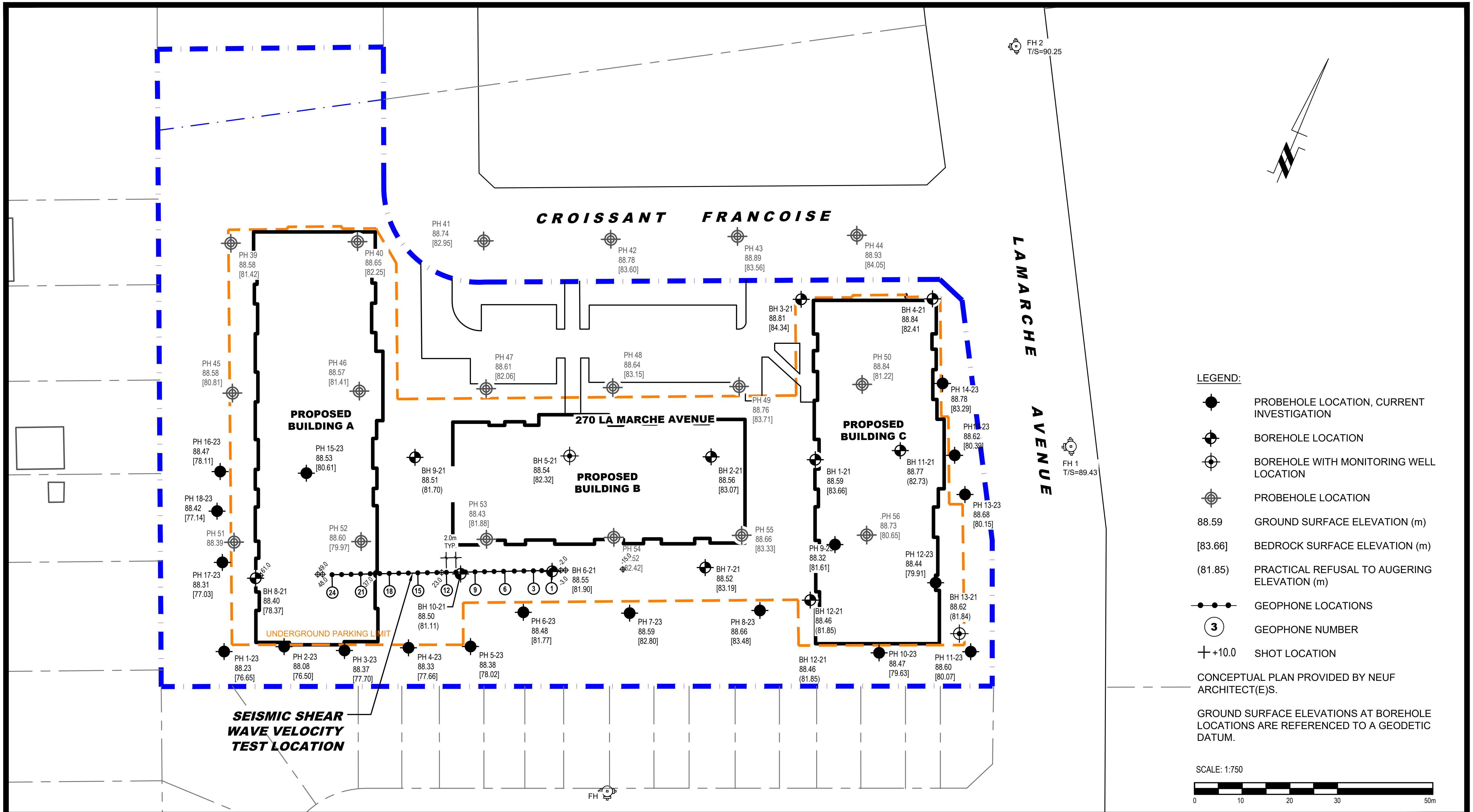


Figure 3 – Shear Wave Velocity Profile at Shot Location 49 m



LEGEND:

●	PROBEHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	PROBEHOLE LOCATION
88.59	GROUND SURFACE ELEVATION (m)
[83.66]	BEDROCK SURFACE ELEVATION (m)
(81.85)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
•—•—•	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECT(E)S.

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

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PATERSON GROUP

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NO.	REVISIONS	DATE	INITIAL
1	SEISMIC SHEAR VELOCITY TEST LOCATION AND 2023 PROBEHOLES ADDED TO PLAN	30/08/2023	MS

CANADIAN RENTAL DEVELOPMENT SERVICES INC.
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
270 LAMARCHE AVENUE

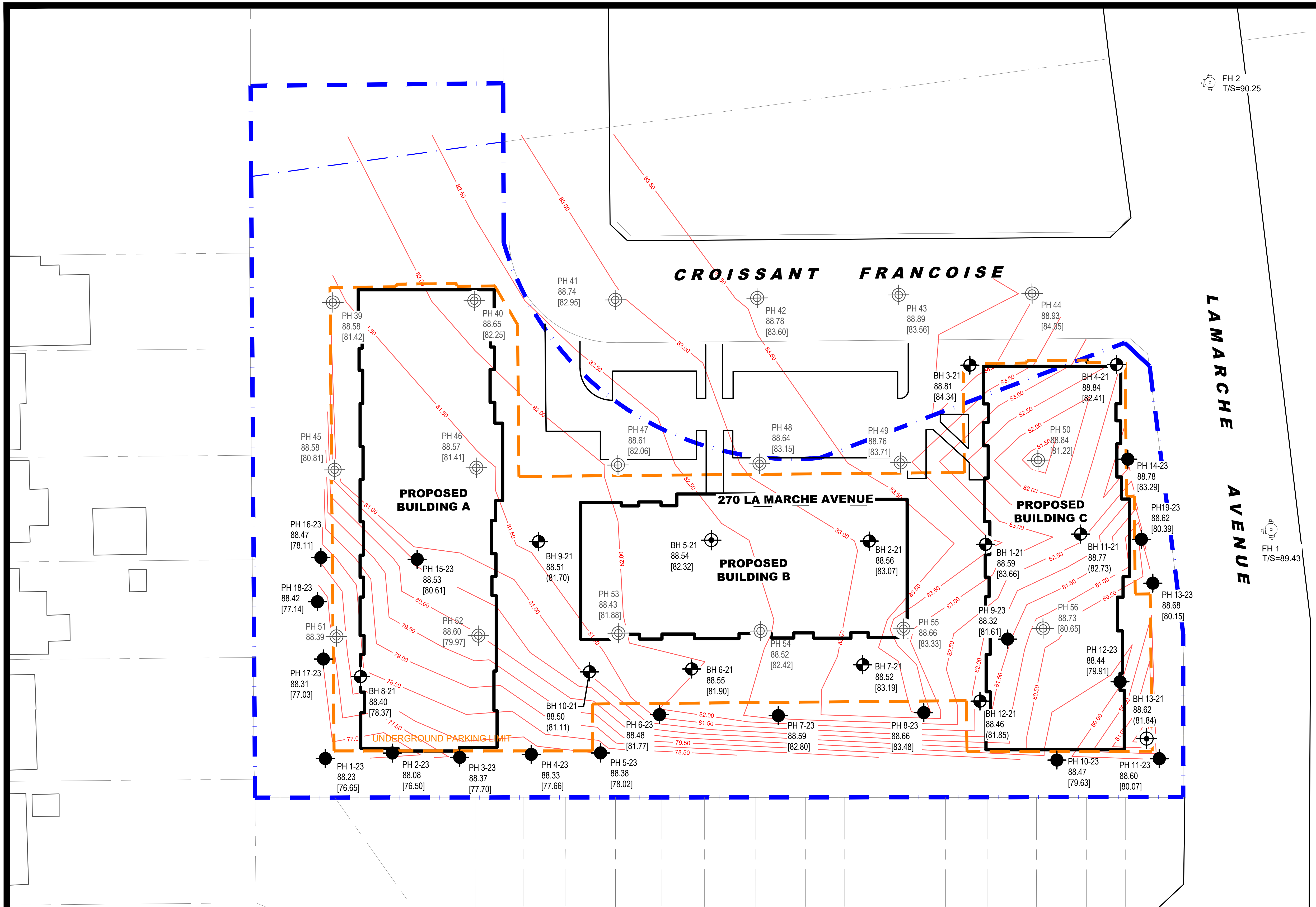
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 08/2023
Drawn by: JM	Report No.: PG6095-1
Checked by: OC	Dwg. No.: PG6095-1
Approved by: DJG	Revision No.: 1

11x17



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
●	PROBEHOLE LOCATION, CURRENT INVESTIGATION
⊕	PROBEHOLE LOCATION
88.59	GROUND SURFACE ELEVATION (m)
[83.66]	BEDROCK SURFACE ELEVATION (m)
(81.85)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
—	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECT(E)S.

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

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7J9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CANADIAN RENTAL DEVELOPMENT SERVICES INC.
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
270 LAMARCHE AVENUE
OTTAWA, ONTARIO

Title:

BEDROCK SURFACE CONTOUR PLAN

Scale: 1:750
Date: 01/2022
Drawn by: JM
Report No.: PG6095-1
Checked by: OC
Dwg. No.: PG6095-2
Approved by: DJG
Revision No.:

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