Geotechnical Investigation

Proposed Institutional Building

451 Smyth road
Ottawa, Ontario

Prepared for PCL Constructors Inc.

Report PG6804 -1 dated November 1, 2023

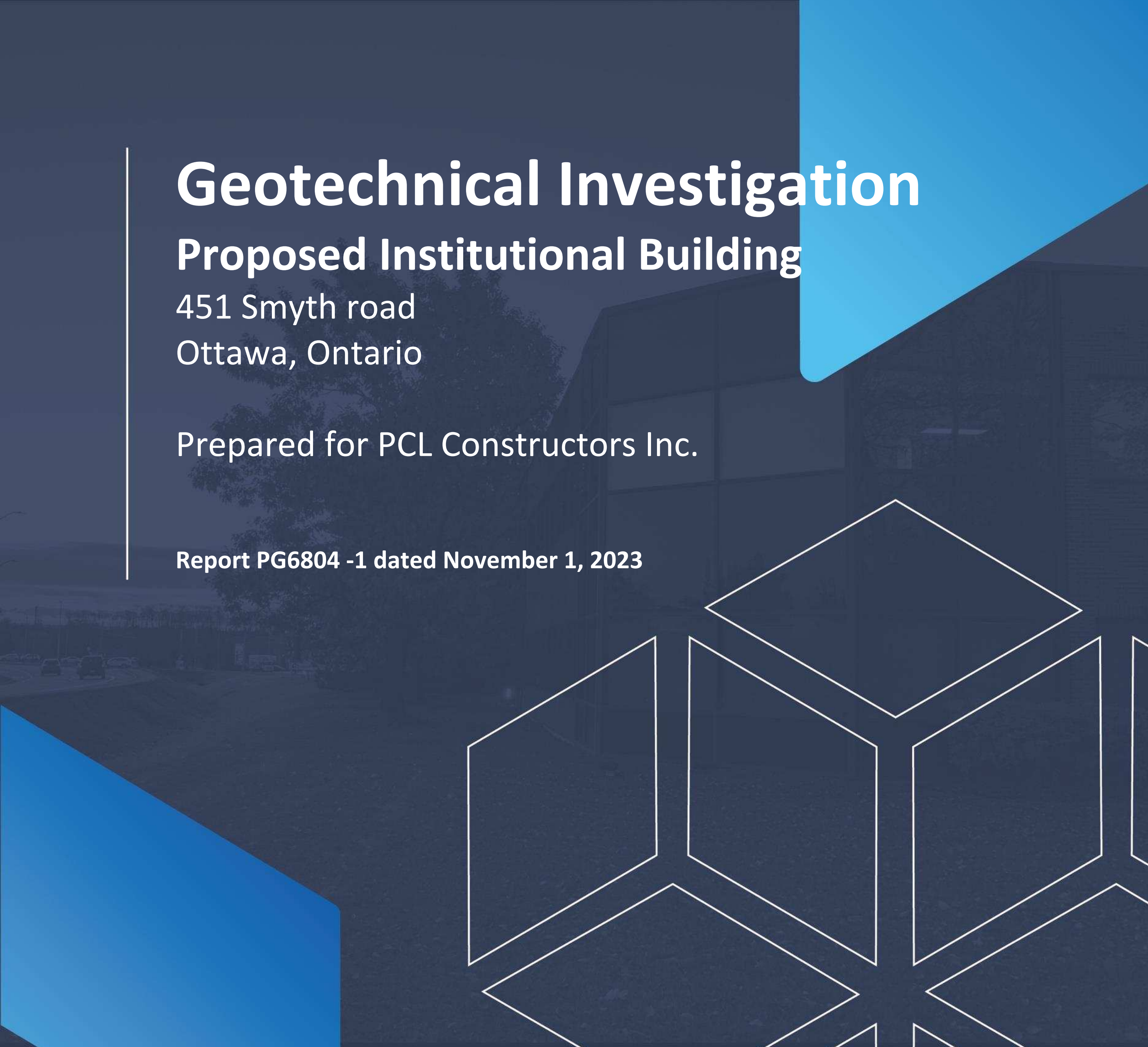


Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Grain Size Analysis Results Unconfined Compressive Strength Testing Results Aggregate Physical Properties and Gradation Analytical Testing Results Soil Profile and Test Data Sheets by Others
Appendix 2	Figure 1 – Key Plan Figures 2 & 3 – Seismic Shear Wave Velocity Profiles Drawing PG6804-1 – Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by PCL to conduct a geotechnical investigation for the proposed Advanced Medical Research Centre (AMRC) Multi-Storey institutional building to be located at 451 Smyth Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsurface and groundwater conditions by means of boreholes and existing soils information.
- ☐ 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. The report contains Paterson's findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Project

The project is understood to be the future Advanced Medical Research Centre (AMRC) of the University of Ottawa which would consist of a seven-storey building with a partial level of underground basement. The building will feature a spacious atrium in between the existing Roger Guidon building. It is further understood that the East entrance would offer convenient lay-by and drop-off facilities, while a parking lot would be available at the West entrance. The subject site is presently occupied by a parking lot for the Roger Guindon campus building.

It is anticipated that the site will be municipally serviced by water, storm, and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on August 31 and September 01, 2023, and consisted of a total of five (5) boreholes sampled to a maximum depth of 9.1 m below the existing grade throughout the subject site. The borehole locations were determined in the field by Paterson personnel in a manner to provide general coverage of the subject site, taking into consideration existing site features and underground services. A previous investigation was conducted by others on site and consisted of six (6) boreholes advance to a maximum of 6.1 m. The locations of the boreholes are illustrated on Drawing PG6804-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson’s geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A 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.

Rock samples were recovered from BH 1-23 to BH 5-23 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.

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

Groundwater

Groundwater monitoring wells were installed in boreholes BH 1-23 and BH 4-23, and flexible standpipe piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevations at the borehole locations, were selected surveyed using a handheld GPS unit and are referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG6804-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and the bedrock core were recovered from the subject site and visually examined in Paterson’s laboratory to review the field logs.

Six representative bedrock sample were tested under unconfined compression strength and three samples of overburden fill were submitted to granular sieve analysis. Furthermore, four samples of shale bedrock where submitted to physical freeze thaw cycle testing and micro deval test for durability.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is presently the parking lot for the Roger Guindon (RGN) campus building which consists of an asphalt-paved parking area.

The site is bordered to the east by the Royal Ottawa Rehabilitation Centre parking lot, to the southeast by the Roger Guindon Campus (RGN) building, to the south by the Children's Hospital of Eastern Ontario (CHEO), to the west by another parking facility associated with the adjacent hospital, and to the north by Ring Road, with a contiguous wooded area and hydro corridor. The RGN building is positioned at a slightly elevated level compared to the subject property. The site exhibits a slight slope towards the northwest, such that the northern section of the site is aligned with the grade of the Ring Road.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the boreholes consists of asphaltic concrete overlying a fill layer consisting of crushed stone and silty sand. The fill layer is underlain by a layer of native sand over a black shale bedrock. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Bedrock was cored at all five borehole locations to confirm refusal. Black Shale bedrock was encountered at a depth ranging from 1.2 m to 1.6 m below the existing ground surface.

Upon review of the core hole sample, the bedrock was found to be of very poor to fair quality at all boreholes and throughout the entire depth, with the exception of BH 1-23, BH 3-23, and BH 4-23. In these specific boreholes, the bedrock quality improved to good starting at depths of 6.5 meters for BH 1-23 and 4.5 meters for BH 3-23 and BH 4-23. Quartz seams were observed in BH 5-23, extending from a depth of 3.4 meters to the end of the borehole. The bedrock is described as very soft to moderately soft, with a hardness ranging between 2 and 3 on the Mohs scale of relative hardness.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of grey sandy shale with some dolomitic layer of the Carlsbad Formation. The overburden drift thickness is anticipated to be between 1 to 4 m.

Representative bedrock core samples were submitted to unconfined compressive strength, freeze thaw and micro-deval. Test results are presented in Table 1 and 2 below, complete test results are presented in Appendix 2.

Table 1 : Unconfined Compression Strength of Bedrock

Borehole	Depth(m)	Unconfined Compressive Strength (MPa)
BH1-23	3.8	37.9
BH2-23	2.7	77.2
BH3-23	3.6	47.3
BH4-23	3.1	70.2
BH5-23	3.8	28.8
BH6-23	2.0	67.3

Table 2 : Bedrock Physical Degradation Testing, Freeze Thaw and Micro Deval

Borehole - RC	Freeze Thaw (%)	Micro-Deval(%)
BH3/-23RC1	33.3	92.0
BH2-23/ RC1 and RC2	82.0	78.5
BH5-23/RC1 and RC 2	81.6	81.0
BH4-23/RC1	84.1	86.7

The physical degradation testing shown in table 2 represents the percentage of degradation of the material following the tests. Tests results indicate that the material is sensitive, soft and degrade easily when exposed to elements and continuous dynamic loading (vehicular).

4.3 Groundwater

Flexible piezometers and monitoring wells were installed as part of our geotechnical investigation. Groundwater level measurements were recorded at the borehole locations and our findings are presented in Table 3. It should also be noted that the groundwater level is subject to seasonal fluctuations. Therefore, groundwater could vary at the time of construction. It should be further noted that groundwater measurements at monitoring well locations can be influenced by surface water entering the backfilled borehole, which can lead to higher-than-normal groundwater level readings. Long-term groundwater levels can also be determined based on observations of the recovered soil samples, such as moisture levels, coloring and consistency. Based on these observations, the long-term groundwater level is expected to be at a 4.5 to 5.5 m depth.

Table 3 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23*	79.78	5.22	74.56	September 13, 2020
BH 2-23	80.89	6.07	74.82	September 11, 2020
BH 3-23	80.05	4.42	75.63	September 11, 2020
BH 4-23*	80.38	2.77	77.61	September 11, 2020
BH 5-23	79.61	4.95	74.66	September 11, 2020

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed multi-storey building is anticipated to be founded on spread footings placed on the bedrock surface and/or approved engineered fill.

Bedrock removal will be required to complete the underground level. Hoe ramming is an option where only small quantities of bedrock need to be removed. It is expected that the bedrock removal will all be completed by hoe ramming and excavating, and that blasting will not be used for this project.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Bedrock Removal

Based on the bedrock encountered in the area, it is expected the bedrock removal will be completed by hoe-ramming. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be completed by an excavator.

Vibration may be induced by rock removal activities. As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 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 carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

It is not recommended that the shale bedrock be reused under settlement sensitive structure or as backfill in the active frost layer (2.1 m).

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 to 20 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, 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 provided in subsection 5.3.

Vibration Considerations

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

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

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

Considering there are several sensitive buildings near the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement

Where 1:1 sloping cannot be accommodated in weathered or fractured horizontal rock anchors, shotcrete and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface during construction. The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations.

Fill Placement

Fill used for grading purposes beneath the proposed buildings should consist 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 to the site. It should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a geo-composite drainage membrane such as Miradrain G100N or Delta Drain 6000 connected to a perimeter drainage system. a composite drainage membrane.

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 150 mm. 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. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. Shale bedrock material should not be used under settlement sensitive structure and pavement areas. The material can be used in landscape areas where settlement will be of minimal impact.

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

5.3 Foundation Design

Bearing Resistance Values

Based on current available plans for the project, the east portion of the building will have an underground basement level. It is expected that footings in that area will be installed at an approximate elevation of 75.5 m. It is expected that the bedrock quality at that elevation will be fair to good. Footings placed at that elevation can be designed using a bearing resistance value at serviceability limit states (SLS) of **1,500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa.** Seismic modeling of the footing deflection can be completed using a factored modulus of subgrade reaction of **90 MPa/m**.

Footings design with an underside elevation of between 76.0 m and 77.4 m can be designed using a bearing resistance value at serviceability limit states (SLS) of **1,200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,450 kPa.** Seismic modeling of the footing deflection can be completed using a factored modulus of subgrade reaction of **70 MPa/m**.

It is expected that footings for the west portion of the structure will be installed above an elevation of 77.5 m and on the upper fractured portion of the shale bedrock. It is expected that a minimum elevation of 78.0 m would need to be achieved to reach a poor to fair quality fractured bearing surface. Footings placed at that elevation can be designed using a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,200 kPa.** Seismic modeling of the footing deflection can be completed using a factored modulus of subgrade reaction of **55 MPa/m**.Note that an excavation of 0.5 to 0.9 m into the weathered shale will be required to reach this proposed bearing surface.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 15 and 10 mm respectively.

Footings placed on the upper weathered shale surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **450 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **650 kPa.** Seismic modeling of the footing deflection can be completed using a factored modulus of subgrade reaction of **40 MPa/m**.

For footings placed upon an existing compact fill layer consisting of silty sand and gravel (above the rock surface), can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. Seismic modeling of the footing deflection can be completed using a factored modulus of subgrade reaction of **28 MPa/m**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

Footings placed on a soil or weathered bedrock bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 15 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 a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:4V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Protection of Potential Expansive Bedrock

The swelling of the shale bedrock formation on surrounding site is well documented and known. The structure should be designed to mitigate the effect of shale swelling.

The swelling of shale formation in Ottawa is attributed to the weathering of pyrite in the formation. Pyrite, most often in the form of iron disulphide mineral, oxidates under the presence of air and water to create sulfuric acid. When calcite is present it reacts with the sulphuric acid produced by oxidation of pyrite to form gypsum. Most often the reaction will take place in a weathered layer of the bedrock allowing for better movement of air and water. The reaction may be aided by autotrophic bacteria.

Based on the success of the nearby Ottawa General Hospital Campus, the adjacent Health Sciences Building of the University of Ottawa (Roger Guindon Hall) and Max Keeping wing of the CHEO, a properly installed concrete skim coat will provide the required protection to the proposed floor slab system.

The recommended treatment measures would be required to be placed within 24 hours of first exposure. The recommendations are aimed at restricting air access and entry into the shale bedrock.

All excavation should be sub-excavated with a minimum of 50 to 75 mm to account for the recommended mud slab thickness. The horizontal surface should be covered with a minimum full strength (25 MPa) concrete at least 50 mm in thickness. It is expected that most of the excavation will be sloped to complete the project and that no vertical faces will be placed directly against the building's foundation wall. However, if a vertical bedrock surface is in direct contact with a foundation wall, then the vertical bedrock surface should also be protected using a vertical form of shotcrete to create a 50 mm coating. It is recommended to reinforce the vertical surface with a light wire mesh. To avoid lateral pressure from potential expansion a minimum 25 mm of compressible insulation should be used. The side and bottoms of excavation for any underfloor services, such as elevator pit, sump pits, sanitary piping, mechanical rooms etc, should be fully protected. Consideration should be taken at fully in-filling the smaller trench with concrete. The concrete should be sulphate resistant as the above noted reaction may also react with the concrete.

Construction planning should ensure the shale surfaces are not left exposed for more than 24hrs. If the area is not ready for coating a minimum of 500 mm of shale or existing soil should remain in place until final excavation at grade is completed.

Furthermore, any cuts or breaks created in the coating during the construction or accidentally should be sealed and repaired within 24 hrs. It is recommended to avoid heavy equipment traffic on the protective coating.

Foundation Option - End Bearing Piled Foundation

Due to environmental, stockpiling and low reuse potential of the shale bedrock on site, it may be uneconomical to complete mass excavation of the upper slab on grade portion of the project. Based on laboratory test data, the bedrock shale material is no suitable for reuse on site once and stockpiled for an extended period of time.

A deep caisson or micropile foundation system would provide minimal spoil generation. A deep pile foundation system will also allow for the existing pavement structure to remain in place under the slab on grade once the asphalt layer is removed.

Driven piles are not recommended for the site. Based on local experience the existing weathered shale bedrock will retain the pile during the driving activities and capacities will not be achieved.

Drilled Shafts and Caissons

Cast-in-place caissons/Drilled shafts can be used where excavation to a proper bearing surface is not achievable. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to fill the caissons. The caissons are to be structurally reinforced over their entire length.

Due to the poor weathered shale surface, it is recommended that caissons be 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 4 below presents the estimated capacity for different typical caisson sizes for a rock socketed caisson extending 5 to 6 m into bedrock or approximately 6 to 7 m below existing grade.

Table 4 - Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)	Factored Capacity Tension at ULS (kN)	Lateral Capacity (KN)
inch	mm	Rock Socket	Rock Socket	
36	900	7,500	750	800
42	1000	8,000	850	900
48	1200	12,000	950	1100
54	1375	15,500	1,050	1400
60	1500	19,000	1,150	1600

notes:
- 5-6 m rock socket in bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

Based on the recent field investigation and bedrock coring it is expected that a highly weathered layer of bedrock will need to be removed to reach a sound surface. The thickness of the weathered layer was evaluated to vary from0.6 to 1.0 m across the site. It is expected that the deep foundation rig will be able to auger through the weathered bedrock layer.

Caisson lateral capacities have been provided assuming a minimum reinforcement ratio of 0.2% and inclusion of shear reinforcement consisting of reinforcement rings place 250 to 300 mm apart along the pile length. An increase lateral capacity can be achieved by increasing the reinforcement ratio and the position of the shear reinforcement. This will increase the general stiffness of the element.

The structural caisson designer should review design loads to provide sufficient resistance to the proposed caissons.

Micropiles

Micropiles are small diameter, deep foundation element composed a high strength steel casing and a high yield threaded steel bar core. The piles are advanced by rotary drills and normally develop their strength within the underlaying rock layer. The steel elements are grouted in place using non shrink grout in a similar fashion to rock anchors. Pressure grouting can be used to achieve greater bond strengths. The geotechnical resistance presented in table 5 below can be used for preliminary purposes, based on the available geotechnical information.

A structural engineer should review the buckling potential, lateral capacity and other structural elements of the micropile design.

Table 5: Micropile Geotechnical Resistance

Casing Size (mm)	Threaded Bar Size (mm)	Cased Length (m)	Bond Length (m)	Compression Capacity (KN) ULS	Tension Capacity (KN) ULS
150	43	4	4	700	400
175	43	4	4	750	475
250	57	4	5	1,500	775
250	63	4	9	1,500	1,400

Since micropiles are considered permanent for a life span of 100 years, double corrosion and/or sacrificial steel corrosion allowance are to be considered for the threaded bar and steel casing. It is recommended that the micropile pile casing have a minimum wall thickness of 12 mm with a minimum of 3 mm of sacrificial steel.

5.4 Design for Earthquakes

Seismic Shear Wave Velocity Testing

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. The seismic site classification is based on the V_{s30} value which results from the harmonic mean of the shear wave velocities on the 30 m profile below the foundation depth of the proposed building.

The testing determines shear wave velocities and depth of the bedrock and overburden at the subject site. To do so, the travel time of the direct, reflected and refracted waves are recovered at increasingly distant geophones.

The acquisition and interpretation of the shear wave velocity testing was completed by Paterson personnel.

Field Program

Paterson field personnel placed 18 horizontal 4.5 Hz. geophones mounted to the ground surface by means of two 75 mm spikes attached to the geophone land case. The geophones were spaced at 1 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 was used to strike an I-Beam seated into the ground surface which in turn created a polarized shear wave that travels into the subsurface. Each strike of the hammer on the I-Beam was considered a shot. At each shot, the hammer trigger switch sent a signal to the seismograph to start the recording of the corresponding data.

The hammer shots were repeated between four (4) to eight (8) times at each shot location to improve the signal to noise ratio, and in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). In addition, the shot locations were completed at 10, 1.5 and 1.0 m away from the first and last geophone, and at the center of the seismic array.

During the survey, the weather conditions and ambient noise were considered acceptable and did not affect the quality of the data acquired. Furthermore, the array was located along a flat area. Therefore, topographical effects were not considered to affect the survey results.

The seismic array testing location was placed as presented in Drawing PG6804-1 - Test Hole Location Plan, included in Appendix 2 of the present report.

Data Processing and Interpretation

Interpretation of the shear wave velocity testing was completed by Paterson personnel. Shear wave velocity was estimated using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, 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.

Determination of the V_{s30} value

Based on our testing results, the bedrock shear wave velocity is **1,975 m/s**. It is understood that the overburden will be completely removed as part of the proposed building construction, and footings will be placed directly on the bedrock surface. Therefore, the shear wave velocity of the overburden will not be considered for calculation of the V_{s30} value. Furthermore, it is assumed that the Vs value calculated for the surface of the bedrock can be extrapolated to a depth of 30 m.

Therefore, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{1{,}975\ m/s}\right)}$$

$$V_{s30=}\ 1{,}975\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **1,975 m/s** for conventional spread footings founded on bedrock surface. Therefore, a **Site Class A** is applicable for design of the proposed building where this applies, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab and Slab on Grade

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed building, the native soil surface, bedrock or approved engineered fill pad will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is expected that the basement area will be mostly used for storage or other purposes, and a concrete floor slab will be used. Therefore, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

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. Alternatively, excavated silty sand fill could be used as select subgrade material around the proposed building footings and under the finish floor slabs where more than 300 mm of granular material would be required. Placement and compaction of the material should be reviewed and approved by Paterson at the time construction. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

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. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

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

It is recommended that a minimum of 500 mm be kept from vertical rock face to foundation wall to avoid supplemental lateral pressure from potential expansive shale.

Where blind side pours are proposed on a vertical rock face, a supplemental layer of a minimum 25 mm of compressible insulation material should be used in combination with the shale protection described in sub section 5.3.

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

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

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 material(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 Ottawa is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

The flexible pavement structure presented in Table 6 and Table 7 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 6 - Recommended Pavement Structure – Driveways Car Only Parking Areas

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

Table 7 - Recommended Pavement Structure – Access Lanes

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

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for parking areas and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material’s SPMDD using suitable vibratory equipment.

The proposed pavement structure, where it abuts the existing pavement, should match the existing pavement layers. It is recommended that a 300 mm wide and 50 mm deep stepped joint be provided where the new asphalt layer joins with the existing asphalt layer to provide more resistance to cracking at the joint.

Pavement Structure Drainage

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

Consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

5.8 Rock Anchor Design

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

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

It should be further noted that center to center spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout 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 being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much 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 be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length.

As the depth at which the apex of the shear failure cone develops is 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 plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on the type of rock encountered on site, a factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information for the upper levels of the fractured shale bedrock (fair quality shale), a **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183 and 0.00009**, respectively. For the clean, surface sounded shale bedrock and the bedrock is free of seams, fractures, and voids within 1.5 m below the top of rock elevation at the anchor location (good quality shale), a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 8. Note that lengths provided in table 8 and 9 below in assumption that the bonded portion of the anchors will extend to a good to excellent rock layer found between 5 and 6 m below grade. Table 8 provides resistance where the weathered portion of the rock is considered in the unbonded length of the anchors.

Table 8 – Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - fair quality shale Hoek and Brown parameters	44 m=0.183 and s=0.00009
Rock Mass Rating (RMR) - good quality shale Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength – fair quality shale bedrock	30 MPa
Unconfined compressive strength –good quality shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 9 and 10 below.

Table 9 – Recommended Rock Anchor Lengths – Grouted Rock Anchor – Upper levels of the fractured shale bedrock (fair quality shale)

Diameter of Corehole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	0.8	3.2	4.0	190
	1.6	3.9	5.5	370
	2.6	4.4	7.0	600
125	0.9	4.1	5.0	340
	1.3	4.7	6.0	500
	1.8	5.1	7.0	700

Table 10 – Recommended Rock Anchor Lengths – Grouted Rock Anchor – Clean, surface sounded shale bedrock and the bedrock (good quality shale)

Diameter of Corehole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	1.2	0.8	2.0	270
	3.3	0.7	4.0	760
	5.7	0.3	6.0	1,350
125	0.9	1.1	2.0	340
	2.1	1.4	3.5	825
	3.6	1.4	5.0	1,420

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 tremie 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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

At the time of writing this report, the final grade of the building was still unknown. The working assumption is that the anticipated underground basement level will be at least 4.0 m lower than the existing natural grade, which primarily consists of fractured shale. It is expected that localized dewatering will be achievable for the project.

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. It is recommended that the drainage system consist of the following:

☐ Where foundation walls will be double side poured, the foundation dampproofing and drainage board is recommended to be installed directly onto the exterior foundation wall between the footing and finished grade.

☐ 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 Paterson.

Waterproofing layers for horizontal buried portion of the structure 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.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the underground basement level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s) or gravity outlet. 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 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 elevator shaft foundation wall and a waterproofing membrane such as BSW H (or approved other) be applied below the elevator pit footing (horizontal application).

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the slab and down to the top of the footing in accordance with the manufacturer's specifications. The BSW H waterproofing membrane should be placed horizontaly below the footing and extend up the sides of the footing. The Colphene Toch'n Stick should overlap the BSW H waterproofing membrane.

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.

The foundation wall of the elevator shaft and buildings sump pit should host a PVC sleeve to allow any water trapped within the interior side of the structures to be discharged to the associated sump pump. A minimum 100 mm diameter perforated, corrugated drainage pipe should extend from the sleeve towards the associated drainage system by gravity drainage and mechanical connection to the associated system. Also, the contractor should ensure that the opening is properly sealed to prevent water from entering the subject structure.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the pit structure and bedrock/soil excavation face can be in-filled with lean concrete, OPSS Granular A or Granular B Type II crushed stone.

It should be noted that a waterproofed concrete (with Xypex Additive, or equivalent) is optional for this waterproofing option.

Adverse Effects from Dewatering on Adjacent Structures

It is understood that one level of underground basement is planned for the proposed development. The existing buildings (RGN) along the southeast portion are expected to be founded over bedrock or within the glacial till above the bedrock surface. Based on field observations and assessment, the groundwater level is anticipated at a 5 to 6 m depth below existing grade.

Since the neighboring structures are founded within native glacial till or directly over a bedrock bearing surface based on available soils information. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

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.

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.

6.2 Protection of Footings Against Frost Action

The underground basement level is expected to be heated. Thus, perimeter footings of the underground basement structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible (i.e., weathered bedrock or bedrock with significant fissures filled with soil), foundation insulation will need to be provided. Alternatively, frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 15 to 20 MPa 28-day strength). It is recommended Paterson field personnel review the frost susceptibility of bedrock surface located within 1.8 m of finished grade.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for the majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending through fill material and weathered shale bedrock should be excavated at 1H:1V or shallower. The flatter slope is required for excavation below groundwater level. The subsurface soils are considered to be Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Excavation in the fractured and sound bedrock is expected to be carried with near vertical side walls. Shotcrete protection is expected to be required in the fractured layer of bedrock for worker’s protection where the vertical excavation is greater than 1.2m. Paterson should review the excavation side walls during construction.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.

Temporary Shoring

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

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

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

Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or stacked precast concrete blocks. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

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

The earth pressures acting on the shoring system may be calculated using the following parameters provided in Table 11.

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/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.

Table 11 – Geotechnical Parameters Lateral Properties

Material Description	Unit Weight (kN/m^3)		Friction Angle (°) φ'	Friction Factor, tanδ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
Existing Fill	18	10	30	0.4	0.33	0.5	3
Sandy Silt	18	10	30	0.4	0.33	0.5	3
Engineered Fill (Granular A)	22	13.5	40	0.5	0.22	0.36	4.6
Engineered Fill (Granular B Type II)	22.5	14	42	0.5	0.2	0.33	5.04
Bedrock	23.5	15.2	55	0.6	0.18	0.1	10

Notes:

I. The earth pressure coefficients provided are for horizontal profile.

II. For soil above the groundwater level the “drained” unit weight should be used and below groundwater level the “effective” unit weight should be used.

III. Existing fill should be free of significant amounts of deleterious material such as those containing organic materials, wood chips and peat. The fill should be approved by Paterson prior to placement

Underpinning Program of Adjacent Structure

Where the excavation for the proposed building is located in close proximity to the adjacent existing building, underpinning may be required. Should the excavation extend below the underside of footing elevation of the existing adjacent building, adequate lateral support for the existing footings will need to be maintained. 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:5V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

It is recommended that, where possible, the edge of the excavation for the proposed building should be kept to a minimum distance of 0.5 m from the edge of the adjacent existing building footings. Should the excavation extend closer to the existing building, it is recommended that an underpinning program be implemented which should be designed by a structural engineer. The following parameters are recommended for the underpinning program from a geotechnical perspective:

☐ It is recommended that the foundation backfill material along exterior foundation walls of the adjacent building be removed. Excavation should be stopped at the existing founding elevation below the footing’s edge.

- ☐ The underpinning program will be completed in sections (panels) by excavating each panel individually in a piano key fashion to maintain adequate lateral support for the existing structure.
- ☐ A minimum 2 m spacing is required between each excavated panel.
- ☐ The maximum panel height of 1 m and a maximum panel width of 1 m is acceptable from a geotechnical perspective.
- ☐ Once the concrete in the first set of panels has set (12 to 24 hours), the second set of panels can be completed. The process is then repeated in consecutive order to maintain adequate lateral support during the duration of the underpinning program.
- ☐ The subsequent courses of panels should be offset from the previous course.
- ☐ The underpinning program should extend down to the proposed underside of footing elevation of the future basement level of the proposed building at the subject site.
- ☐ Each panel should be excavated using suitable excavation equipment down to an undisturbed bearing medium and infilled with a minimum 25 MPa concrete and matching the sulphate resistance requirement described in section 5.3.

Due to the depth of the proposed building excavation and proximity to adjacent properties, underpinning of the existing building foundations may be required to ensure the lateral support zone is maintained. Underpinning requirements for adjacent nearby buildings should be assessed at the time of construction.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

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

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation 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, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The bedrock and overburden material present on site are considered frost susceptible.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions, in particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is 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 samples 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.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed.

The following aspects be performed by the geotechnical consultant:

- ☐ Review preliminary and detailed grading, servicing, and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ☐ Review the bedrock stabilization and excavation requirements at the time of construction.
- ☐ Review and inspection of the installation of the foundation and underfloor drainage systems and elevator waterproofing.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson. All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractor's construction methods, equipment capabilities and schedules.

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 PCL Construction Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 1 , 2023
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Fabrice Venadiambu, EIT, CPI

Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ☐ PCL Construction Inc. (e-mail copy)
- ☐ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE ANALYSIS RESULTS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

AGGREGATE PHYSICAL PROPERTIES AND GRADATION

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

451 Smyth Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 371265.739 **NORTHING:** 5029636.579 **ELEVATION:** 80.89

PROJECT: Supplemental Geotechnical Inveseitgation

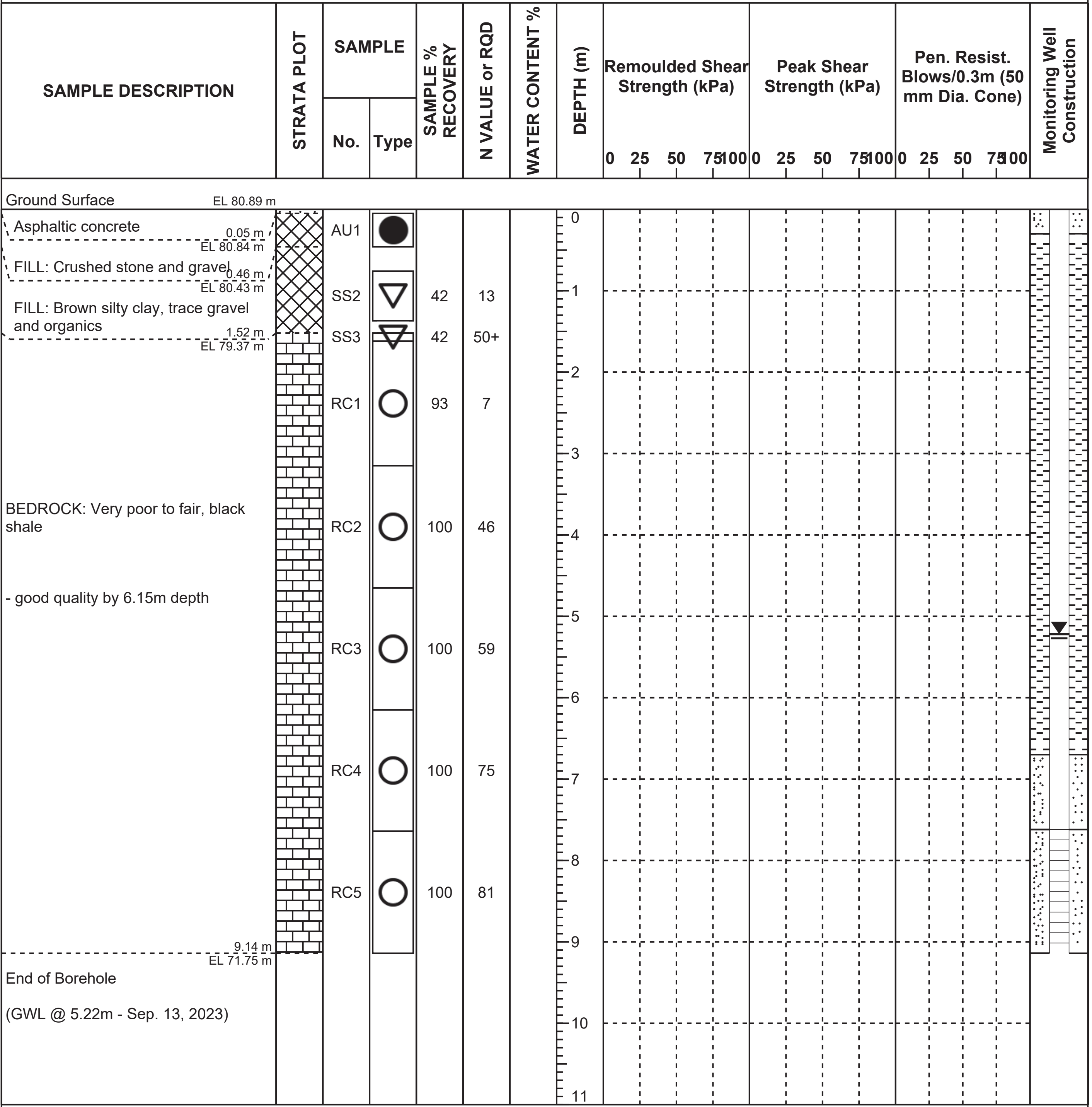
BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: August 31, 2023

FILE NO. **PG6804**

HOLE NO. BH 1-23



SAMPLE DESCRIPTION	Depth	Elevation
Ground Surface		EL 80.89 m
Asphaltic concrete	0.05 m	EL 80.84 m
FILL: Crushed stone and gravel	0.46 m	EL 80.43 m
FILL: Brown silty clay, trace gravel and organics	1.52 m	EL 79.37 m
BEDROCK: Very poor to fair, black shale - good quality by 6.15m depth	9.14 m	EL 71.75 m
End of Borehole (GWL @ 5.22m - Sep. 13, 2023)		

SAMPLE No.	Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1	●			
SS2	▽	42	13	
SS3	▽	42	50+	
RC1	○	93	7	
RC2	○	100	46	
RC3	○	100	59	
RC4	○	100	75	
RC5	○	100	81	

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

451 Smyth Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 371217.836 **NORTHING:** 5029633.24 **ELEVATION:** 79.78

PROJECT: Supplemental Geotechnical Inveslitgation

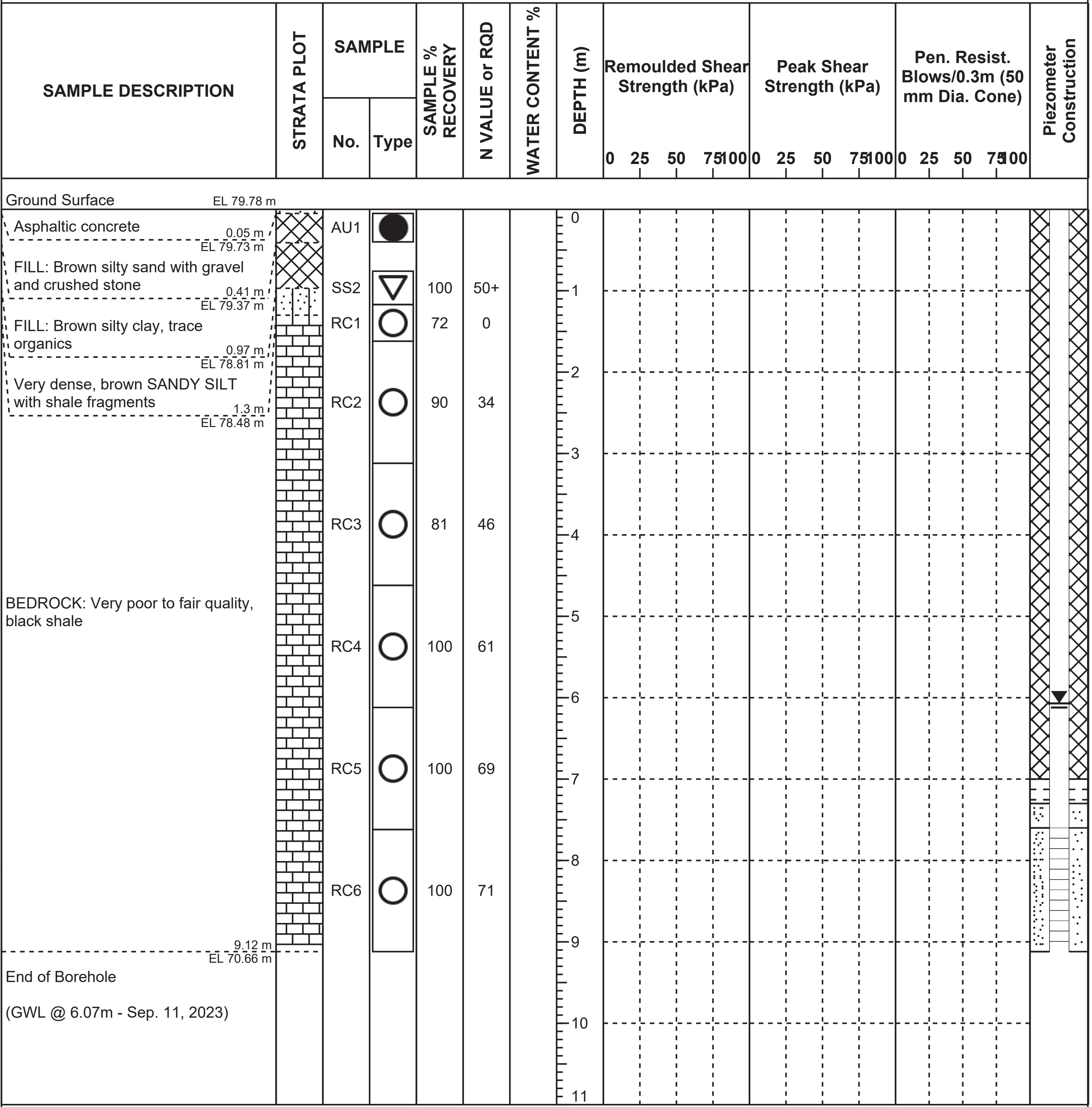
BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: August 31, 2023

FILE NO. PG6804

HOLE NO. BH 2-23



Ground Surface EL 79.78 m

SAMPLE DESCRIPTION		SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD
Asphaltic concrete	0.05 m EL 79.73 m	AU1	●		
FILL: Brown silty sand with gravel and crushed stone	0.41 m EL 79.37 m	SS2	▽	100	50+
FILL: Brown silty clay, trace organics	0.97 m EL 78.81 m	RC1	○	72	0
Very dense, brown SANDY SILT with shale fragments	1.3 m EL 78.48 m	RC2	○	90	34
BEDROCK: Very poor to fair quality, black shale	9.12 m EL 70.66 m	RC3	○	81	46
		RC4	○	100	61
		RC5	○	100	69
		RC6	○	100	71

End of Borehole

(GWL @ 6.07m - Sep. 11, 2023)

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

451 Smyth Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 371258.305 **NORTHING:** 5029613.825 **ELEVATION:** 80.38

PROJECT: Supplemental Geotechnical Invesitgation

BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: August 31, 2023

FILE NO. **PG6804**

HOLE NO. BH 3-23

SAMPLE DESCRIPTION	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)
Ground Surface EL 80.38 m						0
Asphaltic concrete 0.05 m EL 80.33 m	AU1	●				
FILL: Brown silty sand with gravel and crushed stone 0.69 m EL 79.69 m	SS2	▽		36		1
FILL: Brown silty clay with shale fragments 1.6 m EL 78.78 m						
BEDROCK: Very poor to fair quality, black shale	RC1	○	100	8		2
	RC2	○	100	42		3
- good quality by 4.5m depth						4
6 m EL 74.38 m	RC3	○	100	76		5
End of Borehole						6
(GWL @ 4.42m - Sep. 11, 2023)						7
						8
						9
						10
						11

Remoulded Shear Strength (kPa): 0 25 50 75 100

Peak Shear Strength (kPa): 0 25 50 75 100

Pen. Resist. Blows/0.3m (50 mm Dia. Cone): 0 25 50 75 100

Piezometer Construction

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

451 Smyth Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 371241.116 **NORTHING:** 5029583.071 **ELEVATION:** 80.05

PROJECT: Supplemental Geotechnical Invesitgation

BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: September 1, 2023

FILE NO. **PG6804**

HOLE NO. BH 4-23

SAMPLE DESCRIPTION	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Monitoring Well Construction
Ground Surface EL 80.05 m										
Asphaltic concrete 0.05 m EL 80 m	AU1	●			10	0				
FILL: Brown silty sand with gravel and crushed stone 0.76 m EL 79.29 m	SS2/ AU3	▽	100	50+	7	1				
FILL: Brown silty sand with shale fragments 1.42 m EL 78.63 m										
	RC1	○	97	31		2				
BEDROCK: Poor quality, black shale						3				▼
	RC2	○	98	42		4				
- good quality by 4.6m depth										
	RC3	○	100	86		5				
6.04 m EL 74.01 m						6				
End of Borehole						7				
(GWL @ 2.77m - Sep. 11, 2023)						8				
						9				
						10				
						11				

RSLog / Geotechnical Borehole - Geodetic - MW / paterson-group / admin / September 14, 2023 03:22 PM

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

451 Smyth Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 371195.665 **NORTHING:** 5029629.866 **ELEVATION:** 79.61

PROJECT: Supplemental Geotechnical Invesitgation

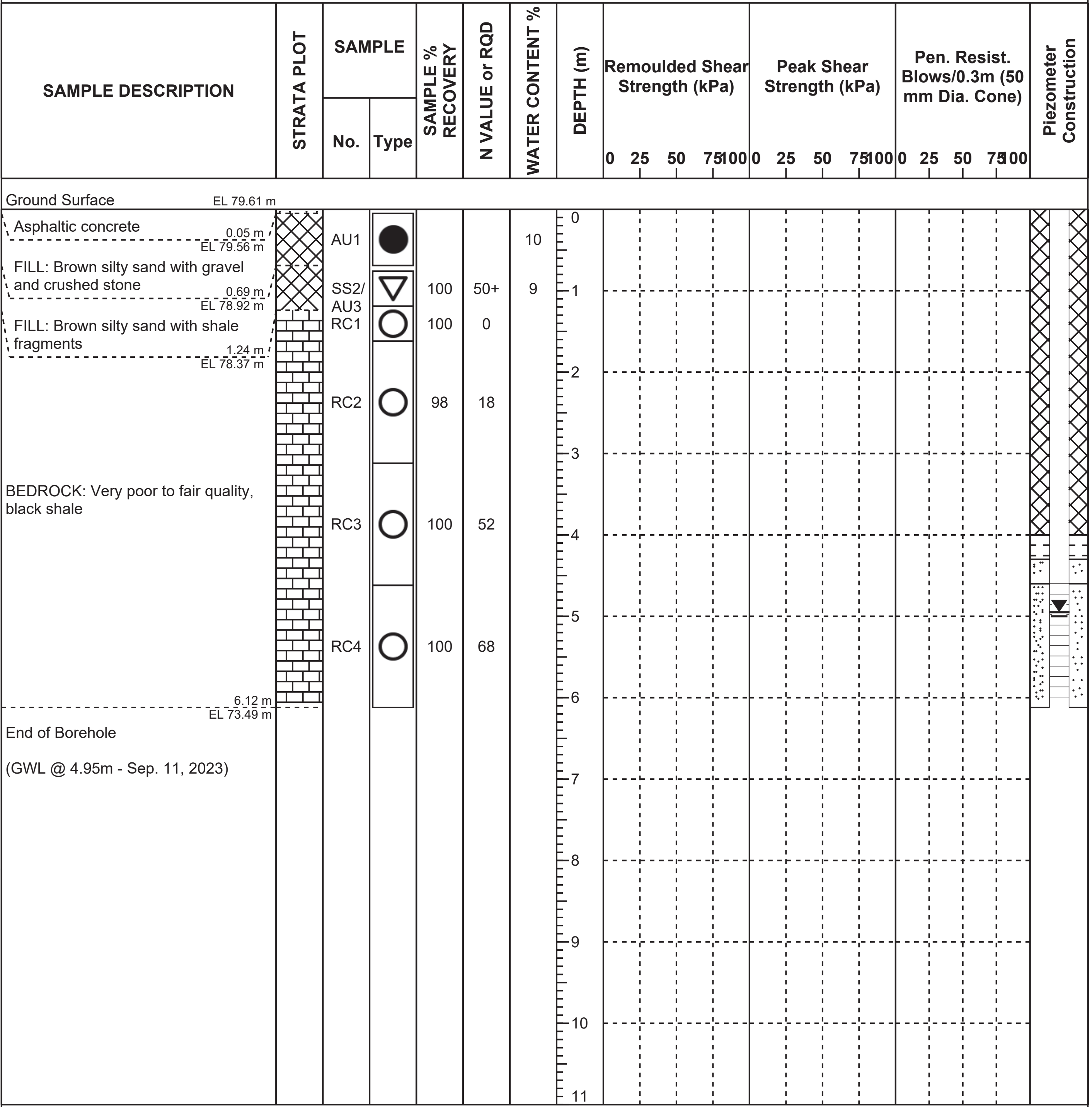
BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: September 1, 2023

FILE NO. PG6804

HOLE NO. BH 5-23



SAMPLE DESCRIPTION	Depth / Elevation	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
Ground Surface	EL 79.61 m					
Asphaltic concrete	0.05 m EL 79.56 m	AU1	●			10
FILL: Brown silty sand with gravel and crushed stone	0.69 m EL 78.92 m	SS2/ AU3	▽	100	50+	9
FILL: Brown silty sand with shale fragments	1.24 m EL 78.37 m	RC1	○	100	0	
BEDROCK: Very poor to fair quality, black shale		RC2	○	98	18	
		RC3	○	100	52	
	6.12 m EL 73.49 m	RC4	○	100	68	

End of Borehole

(GWL @ 4.95m - Sep. 11, 2023)

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

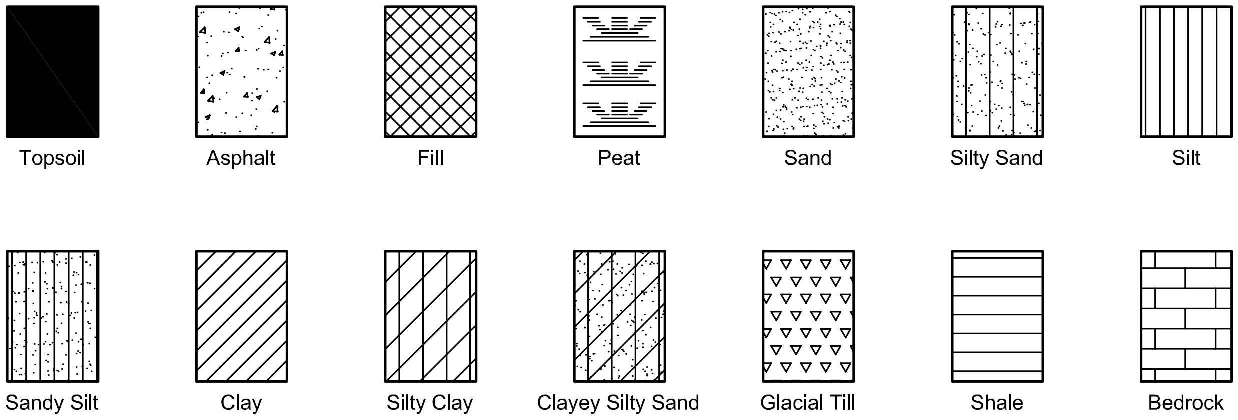
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

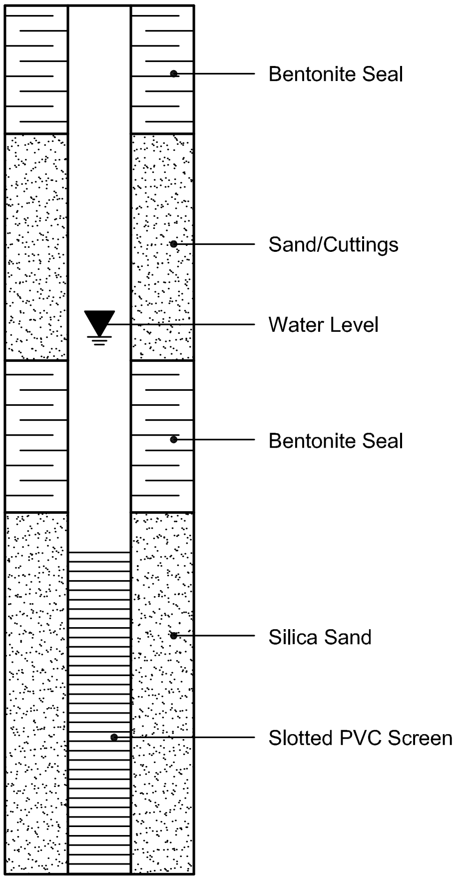
SYMBOLS AND TERMS (continued)

STRATA PLOT

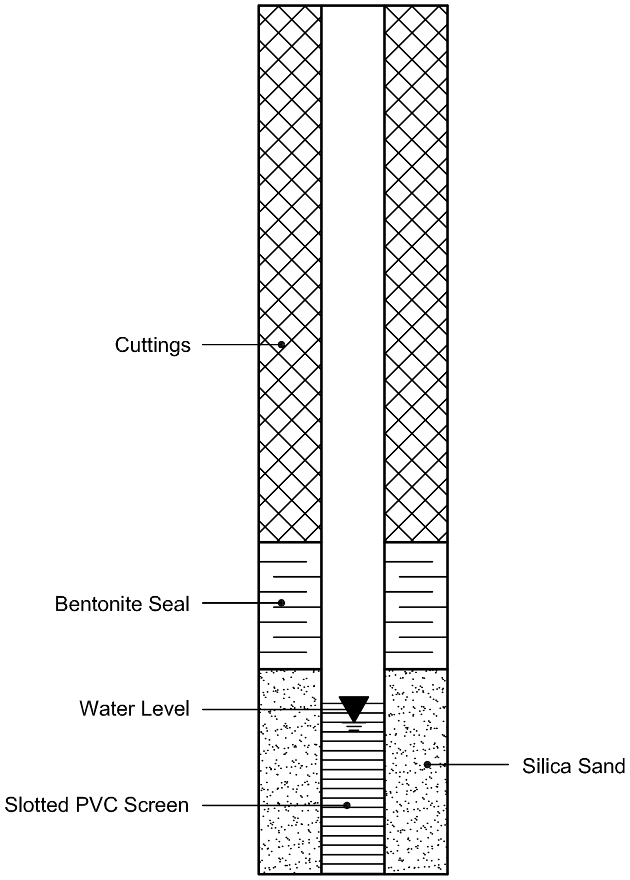


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



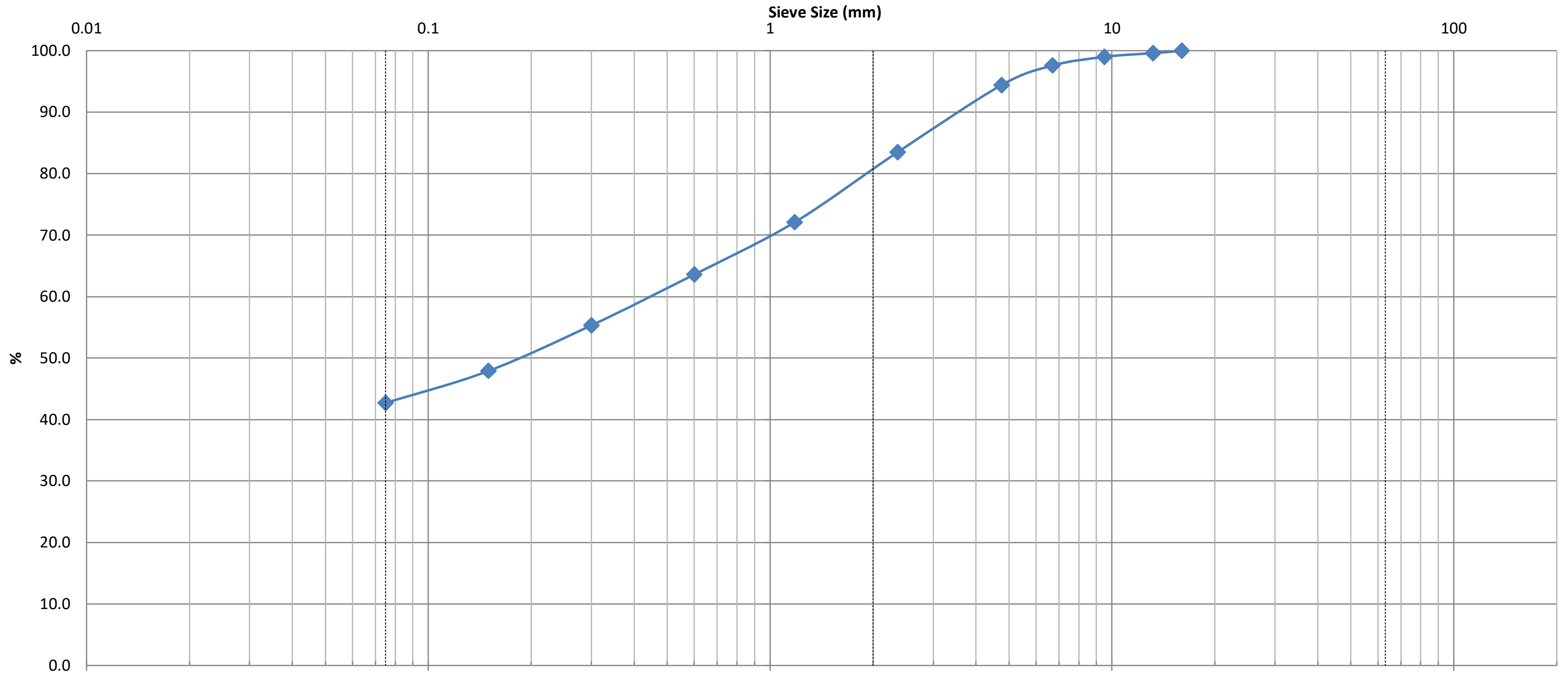
PIEZOMETER CONSTRUCTION



PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	PCL Constructors	DESCRIPTION:	Fine Aggregate	FILE NO:	PG6804
CONTRACT NO.:	-	SPECIFICATION:	**Granular**	LAB NO:	47439
PROJECT:	451 Smyth Road	INTENDED USE:	-	DATE RECEIVED:	6-Sep-23
		PIT OR QUARRY:	-	DATE TESTED:	7-Sep-23
DATE SAMPLED:	31-Aug-23	SOURCE LOCATION:	BH4-23	DATE REPORTED:	12-Sep-23
SAMPLED BY:	C.E.	SAMPLE LOCATION:	AU3 - 2'6"	TESTED BY:	C.P/R.R



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.06	45.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	16	0.45	0.016	0.01	5.6	51.7	42.7			

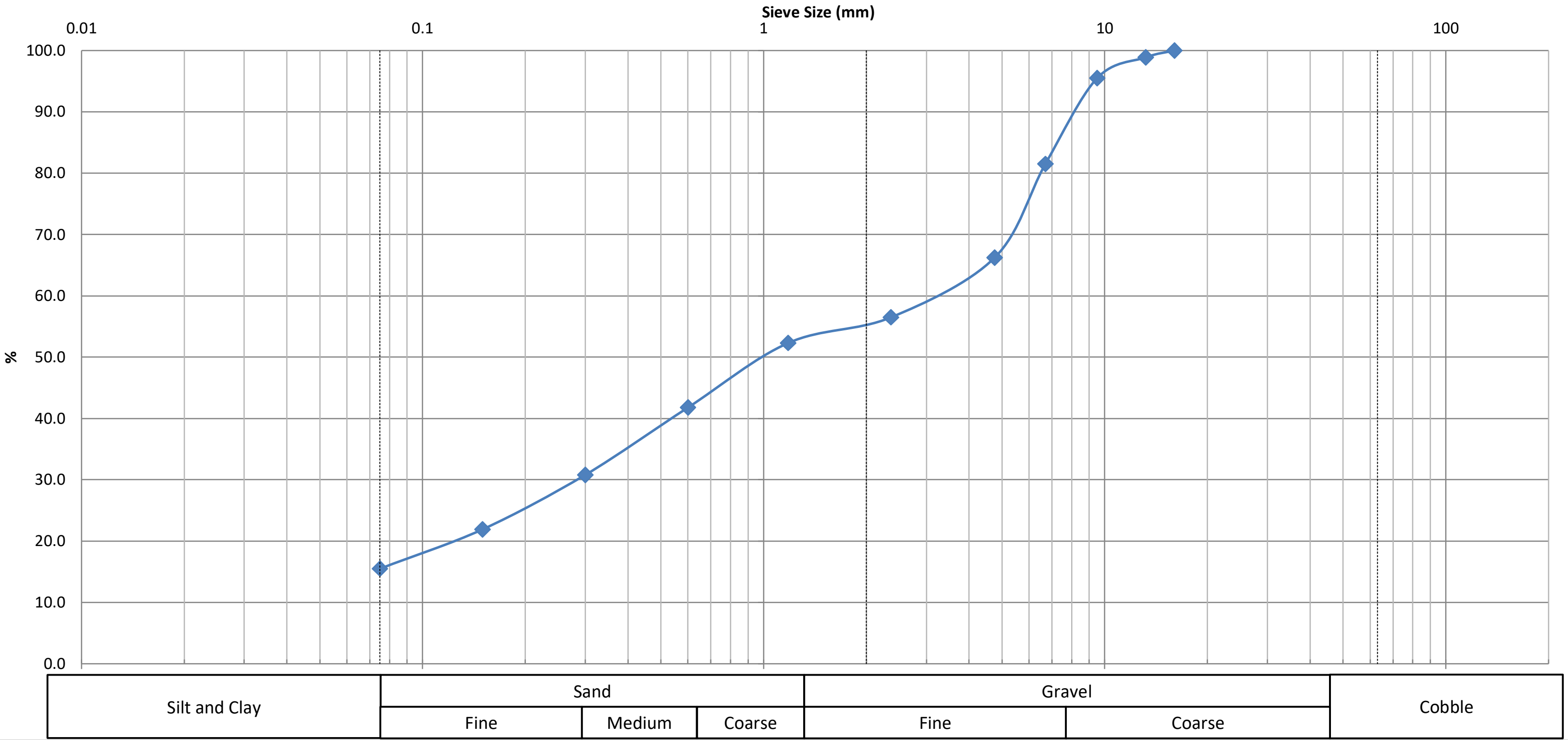
Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	PCL Constructors	DESCRIPTION:	Fine Aggregate	FILE NO:	PG6804
CONTRACT NO.:	-	SPECIFICATION:	**Granular**	LAB NO:	47441
PROJECT:	451 Smyth Road	INTENDED USE:	-	DATE RECEIVED:	6-Sep-23
		PIT OR QUARRY:	-	DATE TESTED:	7-Sep-23
DATE SAMPLED:	31-Aug-23	SOURCE LOCATION:	BH4-23	DATE REPORTED:	12-Sep-23
SAMPLED BY:	C.E.	SAMPLE LOCATION:	AU1 - 2'-2'6"	TESTED BY:	C.P/R.R



Identification	**Soil Classification**	**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
						0.54	72.2

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
16	3.25	0.28	0.045	33.8	50.7	15.5	

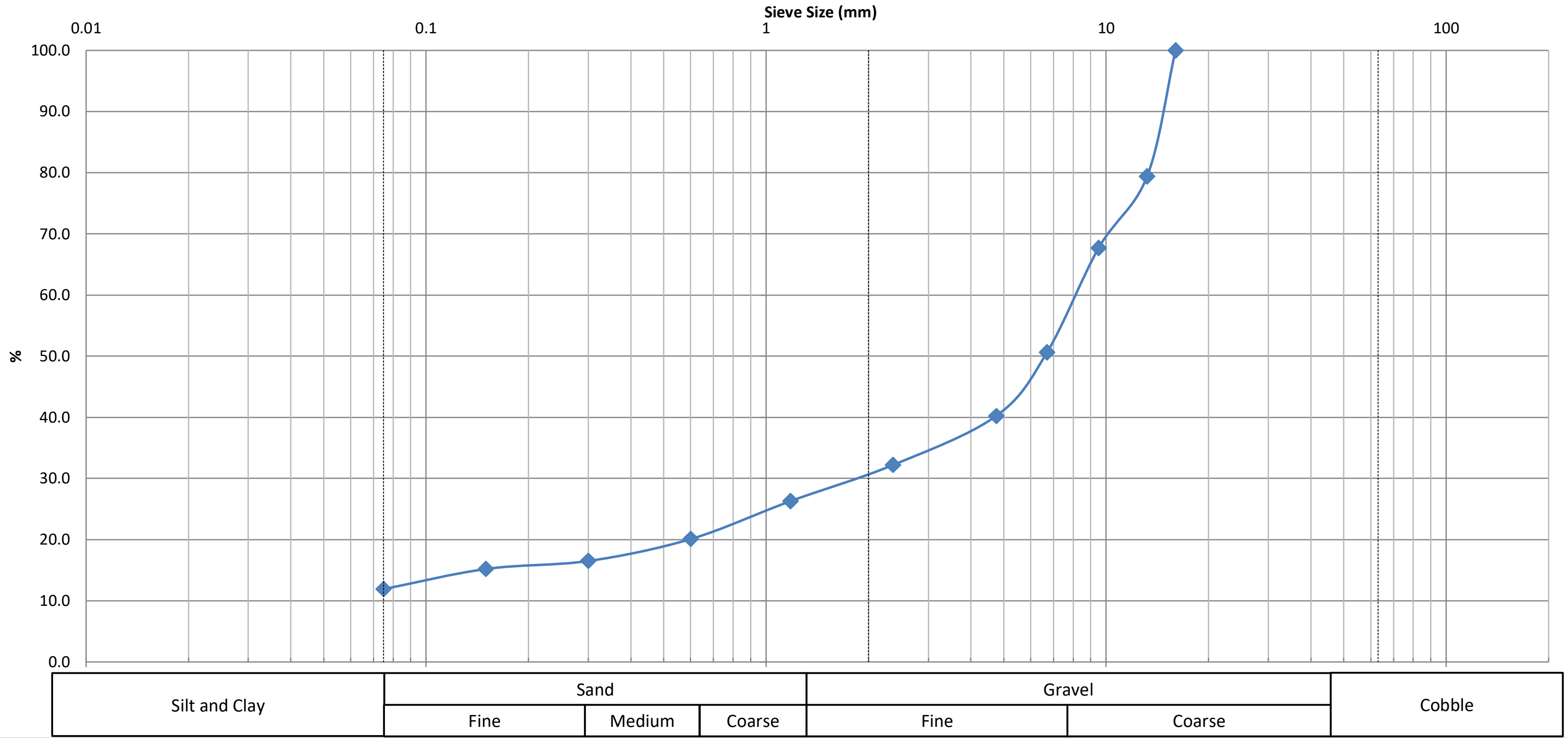
Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	PCL Constructors	DESCRIPTION:	Fine Aggregate	FILE NO:	PG6804
CONTRACT NO.:	-	SPECIFICATION:	**Granular**	LAB NO:	47440
PROJECT:	451 Smyth Road	INTENDED USE:	-	DATE RECEIVED:	6-Sep-23
		PIT OR QUARRY:	-	DATE TESTED:	7-Sep-23
DATE SAMPLED:	31-Aug-23	SOURCE LOCATION:	BH5-23	DATE REPORTED:	12-Sep-23
SAMPLED BY:	C.E.	SAMPLE LOCATION:	AU1 - 2'-2'3"	TESTED BY:	C.P/R.R



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						7.73	146.4

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
16	8.05	1.85	0.055	59.8	28.3	11.9	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH
CSA A23.2-14C

CLIENT:	**PCL Constructors**		FILE No.: **PG6804**
PROJECT:	Geotechnical Investigation, Design Build & Inspections		REPORT No.: 1
SITE ADDRESS:	**451 Smyth Road**		DATE REPT'D: 12-Sep-23
STRUCTURE TYPE & LOCATION:	Rock Core		
SAMPLE INFORAMTION			
LAB NO.:	**47372 A**	**47372 B**	**47372 C**
SAMPLE NO.:	RC2	RC2	RC2
LOCATION:	BH1-23 / 12'7"-12'11"	BH2-23 / 8'9"-9'1"	BH3-23 / 11'10"-12'3"
SAMPLE DATES			
DATE CAST	-	-	-
DATE CORED	-	-	-
DATE RECEIVED	1-Sep-23	1-Sep-23	1-Sep-23
DATE TESTED	11-Sep-23	11-Sep-23	11-Sep-23
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00	47.00	47.00
HEIGHT (mm)	90.00	89.00	90.00
WEIGHT (g)	420	420	420
AREA (mm^2)	1735	1735	1735
VOLUME (cm^3)	156	154	156
UNIT WEIGHT (kg/m^3)	**2690**	**2720**	**2690**
TEST RESULTS			
H / D RATIO	1.91	1.89	1.91
CORRECTION FACTOR	0.992	0.991	0.992
LOAD (lbs)	14900	30400	18600
GROSS Mpa	38.2	77.9	47.7
MPa CORRECTED	**37.9**	**77.2**	**47.3**
FORM OF BREAK	**Type A**	**Type A**	**Type A**
DIRECTION OF LOADING	PARALLEL	PARALLEL	PARALLEL
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		
REMARKS			

TECHNICAL PERSONNEL

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

CERTIFIED LAB

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

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**PCL Constructors**	FILE No.:	**PG6804**
PROJECT:	Geotechnical Investigation, Design Build & Inspections	REPORT No.:	2
SITE ADDRESS:	**451 Smyth Road**	DATE REPT'D:	12-Sep-23
STRUCTURE TYPE & LOCATION:	Rock Core		

SAMPLE INFORAMTION			
LAB NO.:	**47372 D**	**47372 E**	**47372 F**
SAMPLE NO.:	RC2	RC2	RC2
LOCATION:	BH2-23 / 9'11"-10'3"	BH4-23 / 12'5"-12'9"	BH5-23 / 6'2"-6'7"
SAMPLE DATES			
DATE CAST	-	-	-
DATE CORED	-	-	-
DATE RECEIVED	1-Sep-23	1-Sep-23	1-Sep-23
DATE TESTED	11-Sep-23	11-Sep-23	11-Sep-23
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00	47.00	47.00
HEIGHT (mm)	87.00	85.00	91.00
WEIGHT (g)	420	400	420
AREA (mm^2)	1735	1735	1735
VOLUME (cm^3)	151	147	158
UNIT WEIGHT (kg/m^3)	**2783**	**2712**	**2660**
TEST RESULTS			
H / D RATIO	1.85	1.81	1.94
CORRECTION FACTOR	0.988	0.984	0.994
LOAD (lbs)	27700	11400	26400
GROSS Mpa	71.0	29.2	67.7
MPa CORRECTED	**70.2**	**28.8**	**67.3**
FORM OF BREAK	**Type A**	**Type A**	**Type A**
DIRECTION OF LOADING	PARALLEL	PARALLEL	PARALLEL
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		
REMARKS			

TECHNICAL PERSONNEL

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

CERTIFIED LAB

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

PATERSON GROUP

CONCRETE AGGREGATE PHYSICAL PROPERTIES & GRADATION

CLIENT:	PCL Conctuctors	DESCRIPTION:	Coarse Aggregate	FILE NO.:	PG6804
PROJECT:	451 Smyth Rd.	SPECIFICATION:		LAB NO.:	49058
		INTENDED USE:			
		PIT OR QUARRY:			
DATE SAMPLED:	31-Aug-23	SOURCE LOCATION:		DATE REQUESTED:	12-Oct-23
CONTRACTOR:		SAMPLE LOCATION:	BH3-23 / RC1	DATE TESTED:	12-Oct-23
DATE & TIME RECEIVED:	12-Oct-23			DATE REPORTED:	27-Oct-23
				TESTED BY:	C.P.

WEIGHT BEFORE WASH							A+B	4975.8
WEIGHT AFTER WASH					A	B	A+B	

SIEVE SIZE (mm)	TOTAL WEIGHT RETAINED	WEIGHT RETAINED (A)	WEIGHT RETAINED (B)	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
75								
63								
53								
37.5								
26.5								
19	**0.0**	0.0		0.0	**100.0**			
16	**496.6**	496.6		10.0	**90.0**			
13.2	**1232.7**	1232.7		24.8	**75.2**			
9.5	**2209.6**	2209.6		44.4	**55.6**			
6.7	**3024.6**	3024.6		60.8	**39.2**			for physicals only
4.75	**3555.2**	3555.2		71.4	**28.6**			
PAN	**1417.0**	1417.0						
SIEVE CHECK COARSE					0.07			0.3% max.

OTHER TESTS		**RESULT**	**REFERENCE MATERIAL**	
			LAB NO.	**RESULT**
LS-601	WASH PASS	-		
LS-604	RELATIVE DENSITY (OVEN DRY)	-		
LS-604	RELATIVE DENSITY (SSD)	-		
LS-604	RELATIVE DENSITY (APPARENT)	-		
LS-604	ABSORPTION	-		
LS-606	MgSO$_4$ SOUNDNESS	-		
LS-608	FLAT AND ELONGATED	-		
LS-609	PETROGRAPHIC NUMBER	-		
LS-614	FREEZE THAW	**33.3%**		
LS-618	MICRO DEVAL	**92.0%**		

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

CONCRETE AGGREGATE PHYSICAL PROPERTIES & GRADATION

CLIENT:	PCL Conctuctors	DESCRIPTION:	Coarse Aggregate	FILE NO.:	PG6804
PROJECT:	451 Smyth Rd.	SPECIFICATION:		LAB NO.:	49059
		INTENDED USE:			
		PIT OR QUARRY:			
DATE SAMPLED:	31-Aug-23	SOURCE LOCATION:		DATE REQUESTED:	12-Oct-23
CONTRACTOR:		SAMPLE LOCATION:	BH2-23 / RC1 & 2	DATE TESTED:	12-Oct-23
DATE & TIME RECEIVED:	12-Oct-23			DATE REPORTED:	27-Oct-23
				TESTED BY:	C.P.

WEIGHT BEFORE WASH						A+B	6241.9
WEIGHT AFTER WASH				A	B	A+B	

SIEVE SIZE (mm)	TOTAL WEIGHT RETAINED	WEIGHT RETAINED (A)	WEIGHT RETAINED (B)	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
75								
63								
53								
37.5								
26.5	**0.0**	0.0		0.0	**100.0**			
19	**29.0**	29.0		0.5	**99.5**			
16	**939.4**	939.4		15.0	**85.0**			
13.2	**2168.9**	2168.9		34.7	**65.3**			
9.5	**3533.0**	3533.0		56.6	**43.4**			
6.7	**4397.1**	4397.1		70.4	**29.6**			for physicals only
4.75	**4894.6**	4894.6		78.4	**21.6**			
PAN	**1342.0**	1342.0						
SIEVE CHECK COARSE					0.08		0.3% max.	

OTHER TESTS		**RESULT**	**REFERENCE MATERIAL**	
			LAB NO.	**RESULT**
LS-601	WASH PASS	-		
LS-604	RELATIVE DENSITY (OVEN DRY)	-		
LS-604	RELATIVE DENSITY (SSD)	-		
LS-604	RELATIVE DENSITY (APPARENT)	-		
LS-604	ABSORPTION	-		
LS-606	$MgSO_4$ SOUNDNESS	-		
LS-608	FLAT AND ELONGATED	-		
LS-609	PETROGRAPHIC NUMBER	-		
LS-614	FREEZE THAW	**82.0%**		
LS-618	MICRO DEVAL	**78.5%**		

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

CONCRETE AGGREGATE PHYSICAL PROPERTIES & GRADATION

CLIENT:	PCL Conctuctors	DESCRIPTION:	Coarse Aggregate	FILE NO.:	PG6804
PROJECT:	451 Smyth Rd.	SPECIFICATION:		LAB NO.:	49060
		INTENDED USE:			
		PIT OR QUARRY:			
DATE SAMPLED:	01-Sep-23	SOURCE LOCATION:		DATE REQUESTED:	12-Oct-23
CONTRACTOR:		SAMPLE LOCATION:	BH5-23 / RC1 & 2	DATE TESTED:	12-Oct-23
DATE & TIME RECEIVED:		12-Oct-23		DATE REPORTED:	27-Oct-23
				TESTED BY:	C.P.

WEIGHT BEFORE WASH						A+B	6608.6
WEIGHT AFTER WASH				A	B	A+B	

SIEVE SIZE (mm)	TOTAL WEIGHT RETAINED	WEIGHT RETAINED (A)	WEIGHT RETAINED (B)	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
75								
63								
53								
37.5								
26.5	**0.0**	0.0		0.0	**100.0**			
19	**10.7**	10.7		0.2	**99.8**			
16	**906.7**	906.7		13.7	**86.3**			
13.2	**2283.3**	2283.3		34.6	**65.4**			
9.5	**3851.3**	3851.3		58.3	**41.7**			
6.7	**4771.5**	1771.5		72.2	**27.8**			for physicals only
4.75	**5294.4**	5294.4		80.1	**19.9**			
PAN	**1314.2**	1314.2						
SIEVE CHECK COARSE					0.00	0.3% max.		

OTHER TESTS		**RESULT**	**REFERENCE MATERIAL** **LAB NO.**	**RESULT**
LS-601	WASH PASS	-		
LS-604	RELATIVE DENSITY (OVEN DRY)	-		
LS-604	RELATIVE DENSITY (SSD)	-		
LS-604	RELATIVE DENSITY (APPARENT)	-		
LS-604	ABSORPTION	-		
LS-606	$MgSO_4$ SOUNDNESS	-		
LS-608	FLAT AND ELONGATED	-		
LS-609	PETROGRAPHIC NUMBER	-		
LS-614	FREEZE THAW	**81.6%**		
LS-618	MICRO DEVAL	**81.0%**		

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

CONCRETE AGGREGATE PHYSICAL PROPERTIES & GRADATION

CLIENT:	PCL Conctuctors	DESCRIPTION:	Coarse Aggregate	FILE NO.:	PG6804
PROJECT:	451 Smyth Rd.	SPECIFICATION:		LAB NO.:	49061
		INTENDED USE:			
		PIT OR QUARRY:			
DATE SAMPLED:	01-Sep-23	SOURCE LOCATION:		DATE REQUESTED:	12-Oct-23
CONTRACTOR:		SAMPLE LOCATION:	BH4-23 / RC1	DATE TESTED:	12-Oct-23
DATE & TIME RECEIVED:		12-Oct-23		DATE REPORTED:	27-Oct-23
				TESTED BY:	C.P.

WEIGHT BEFORE WASH							A+B	6782.0
WEIGHT AFTER WASH					A	B	A+B	
SIEVE SIZE (mm)	**TOTAL WEIGHT RETAINED**	**WEIGHT RETAINED (A)**	**WEIGHT RETAINED (B)**	**PERCENT RETAINED**	**PERCENT PASSING**	**LOWER SPEC**	**UPPER SPEC**	**REMARK**
75								
63								
53								
37.5								
26.5	**0.0**	0.0		0.0	**100.0**			
19	**80.8**	80.8		1.2	**98.8**			
16	**1047.2**	1047.2		15.4	**84.6**			
13.2	**2108.3**	2108.3		31.1	**68.9**			
9.5	**3589.7**	3589.7		52.9	**47.1**			
6.7	**4771.5**	4547.1		70.4	**29.6**			for physicals only
4.75	**5112.6**	5112.6		75.4	**24.6**			
PAN	**1669.4**	1669.4						
SIEVE CHECK COARSE					0.00	0.3% max.		

OTHER TESTS		**RESULT**	**REFERENCE MATERIAL**	
			LAB NO.	**RESULT**
LS-601	WASH PASS	-		
LS-604	RELATIVE DENSITY (OVEN DRY)	-		
LS-604	RELATIVE DENSITY (SSD)	-		
LS-604	RELATIVE DENSITY (APPARENT)	-		
LS-604	ABSORPTION	-		
LS-606	$MgSO_4$ SOUNDNESS	-		
LS-608	FLAT AND ELONGATED	-		
LS-609	PETROGRAPHIC NUMBER	-		
LS-614	FREEZE THAW	**84.1%**		
LS-618	MICRO DEVAL	**86.7%**		

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Report Date: 12-Sep-2023

Client: **Paterson Group Consulting Engineers**

Order Date: 6-Sep-2023

Client PO: 58313

Project Description: PE6237

	Client ID:	BH3-23-AU1	BH3-23-SS2	BH4-23-AU1	BH4-23-AU3	-	-
	Sample Date:	31-Aug-23 09:00	31-Aug-23 09:00	01-Sep-23 09:00	01-Sep-23 09:00		
	Sample ID:	2336195-05	2336195-06	2336195-07	2336195-08		
	Matrix:	Soil	Soil	Soil	Soil		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	93.7	92.5	93.0	96.4	-	-
General Inorganics							
SAR	0.01 N/A	7.85	4.73	13.4	23.1	-	-
Conductivity	5 uS/cm	4730	1940	5940	1710	-	-
pH	0.05 pH Units	-	-	-	8.81	-	-
Resistivity	0.1 Ohm.m	-	-	-	5.8	-	-
Anions							
Chloride	10 ug/g	-	-	-	637	-	-
Sulphate	10 ug/g	-	-	-	482	-	-
Metals							
Antimony	1 ug/g	<1.0	<1.0	<1.0	<1.0	-	-
Arsenic	1 ug/g	4.2	6.4	3.9	7.5	-	-
Barium	1 ug/g	88.0	90.0	66.2	80.7	-	-
Beryllium	0.5 ug/g	0.5	1.2	<0.5	1.1	-	-
Boron	5 ug/g	10.9	6.6	8.5	9.8	-	-
Cadmium	0.5 ug/g	<0.5	<0.5	<0.5	<0.5	-	-
Chromium	5 ug/g	21.7	31.3	17.9	31.7	-	-
Cobalt	1 ug/g	6.1	18.8	5.4	19.8	-	-
Copper	5 ug/g	12.6	44.1	14.8	38.0	-	-
Lead	1 ug/g	14.3	17.4	10.7	26.7	-	-
Molybdenum	1 ug/g	2.1	<1.0	1.9	1.1	-	-
Nickel	5 ug/g	14.3	44.5	12.1	45.9	-	-
Selenium	1 ug/g	<1.0	<1.0	<1.0	<1.0	-	-
Silver	0.3 ug/g	<0.3	<0.3	<0.3	<0.3	-	-
Thallium	1 ug/g	<1.0	<1.0	<1.0	<1.0	-	-

LOG NO: MW-1-21

BOREHOLE LOG

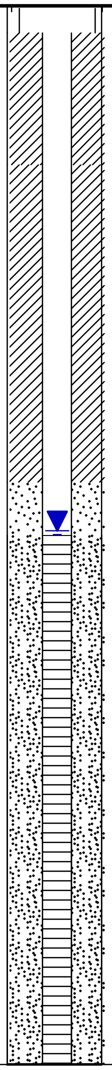
PAGE 1 OF 1

PROJECT NUMBER	30054299	**CLIENT**	University of Ottawa
PROJECT NAME	Geotechnical and Environmental Assessment	**PROJECT LOCATION**	RGN Campus- 451 Smyth Road, Ottawa, Ontario
DRILLING CONTRACTOR	Downing	**PROJECTION**	NAD83
DRILLING METHOD	Hollow Stem -HQ coring	**NORTHING**	45.403417
		EASTING	75.652381
DRILL DATE	21-1-27	**GROUND ELEVATION**	
		TOC ELEVATION	97.195
LOGGED BY	Lennart de Groot	**CHECKED BY**	Troy Austrins
GROUND WATER LEVEL ▼	3.02 mbtoc (2021-02-02)	**GROUND WATER ELEVATION** ▼	mbgs (2021-02-02)
HOLE DIAMETER	4 "	**WELL DIAMETER**	2 "

DEPTH (m)	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS
0.08	**ASPHALT**		Auger Cuttings				
0.30	**SAND (Fill)**, gravel, dark blackish brown, coarse grained, dry						
	SAND, rock fragments, light grayish brown, coarse grained	1	Split Spoon	75	52,52,44, 50		P, F1, F2, M, V
0.91	**SAND**, rock fragments, dark grayish brown, coarse grained, dry	2	Split Spoon	90	24,48, 38,52	0 ppm	
1.68	SHALE, completely weathered, Layer RQD = 0%	3	Rock Core	90			
1.86	SHALE, Layer RQD = 10% gray, weathered upper section	4	Rock Core	90			
2.62	SHALE, Layer RQD = 19% gray to black, thin, horizontal bedding	5	Rock Core	100			
4.47	SHALE, Layer RQD = 67% gray to black, thin to medium, horizontal bedding	6	Rock Core	90			
5.97 6.11	SHALE, Layer RQD = 76% gray to black, thin to medium, horizontal bedding	7	Rock Core	100			

WELL DIAGRAM

Casing Type: Flushmount



End of Borehole at 6.11 meters

NOTES:

masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:

M = Metals
I = Inorganics
V = VOC's
B = BTEX
P = PAH
F1 = PHC F1
F2 = PHC F2-F4
PCB = PCB's
Pes = Pesticides

SAMPLE TYPE:

▮ Auger Cuttings
☒ Split Spoon
▯ Rock Core

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LOG NO: MW-2-21

BOREHOLE LOG

PAGE 1 OF 1

PROJECT NUMBER 30054299

PROJECT NAME Geotechnical and Environmental Assessment

CLIENT University of Ottawa

PROJECT LOCATION RGN Campus- 451 Smyth Road, Ottawa, Ontario

DRILLING CONTRACTOR Downing

DRILLING METHOD Hollow Stem -HQ coring

DRILL DATE 21-1-27

LOGGED BY Lennart de Groot **CHECKED BY** Troy Austrins

HOLE DIAMETER 4 " **WELL DIAMETER** 2 "

PROJECTION NAD83

NORTHING 45.403592 **EASTING** 75.651058

GROUND ELEVATION **TOC ELEVATION** 99.410

GROUND WATER LEVEL ▼ 4.81 mbtoc (2021-02-02)

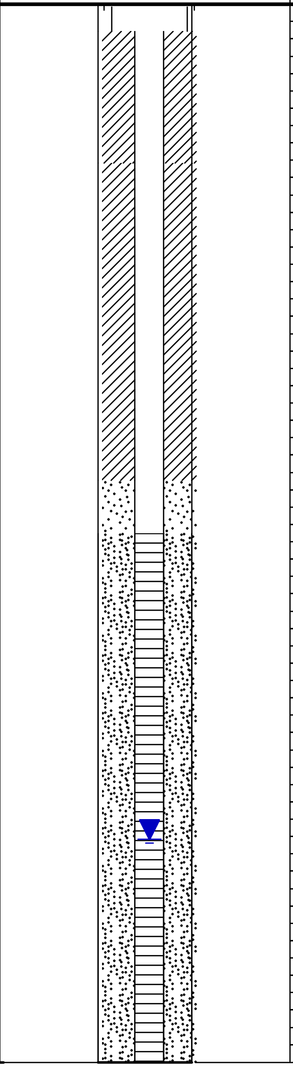
GROUND WATER ELEVATION ▼ mbgs (2021-02-02)

DEPTH (m)	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS
0.08	**ASPHALT**		▐				
0.30	**SAND (Fill)**, dark brown, coarse grained, dry		▐				
0.91	**SAND AND GRAVEL (Fill)**, light gray, coarse grained, dry, rubber debris		⧗	75	44,46, 32,7		
1.52	**CLAYEY SAND**, rock fragments, light grayish brown, coarse grained, dry	1	☒	90	4,12, 16,28		P, F1, F2, M, V
1.80	SHALE, completely weathered, Layer RQD = 0%	2	▯	90			
2.86	SHALE, gray, Layer RQD = 5% heavily broken	3	▯	85			
4.41	SHALE, Layer RQD = 16% heavily broken	4	▯	100			
5.96	SHALE, Layer RQD = 49% grey to black, thin to medium, horizontal bedding	5	▯	100			
6.10	SHALE, Layer RQD = 70% grey to black, thin to medium,horizontal bedding	6	▯	100			

End of Borehole at 6.10meters

WELL DIAGRAM

Casing Type: Flushmount



NOTES:

masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:

M = Metals
I = Inorganics
V = VOC's
B = BTEX
P = PAH
F1 = PHC F1
F2 = PHC F2-F4
PCB = PCB's
Pes = Pesticides

SAMPLE TYPE:

▐ Auger Cuttings
⧗ Standard Penetration Test
☒ Split Spoon
▯ Rock Core

Note: Any decisions/actions made by a third party based on this log are the sole responsibility of the third party. ARCADIS accepts no liability for third party decisions/actions made based on this log.

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LOG NO: BH-3-21

BOREHOLE LOG

PAGE 1 OF 1

PROJECT NUMBER 30054299

PROJECT NAME Geotechnical and Environmental Assessment

CLIENT University of Ottawa

PROJECT LOCATION RGN Campus- 451 Smyth Road, Ottawa, Ontario

DRILLING CONTRACTOR Downing

DRILLING METHOD Hollow Stem

DRILL DATE 21-1-27

LOGGED BY Lennart de Groot **CHECKED BY** Troy Austrins

HOLE DIAMETER 4 " **WELL DIAMETER**

PROJECTION NAD83

NORTHING 45.403194 **EASTING** 75.650653

GROUND ELEVATION **TOC ELEVATION**

GROUND WATER LEVEL ▼ ( )

GROUND WATER ELEVATION ▼ mbgs ( )

DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS	WELL DIAGRAM	DEPTH (m)
		ASPHALT 0.08		▮						
0.2	▨	**SILTY SAND (Fill)**, gravel, light grayish brown, coarse grained 0.30		▮						0.2
0.4	▨	**SILTY SAND (Fill)**, light brown		☒		48,49,39	P, F1,			0.4
0.6	▨		1	☒	75	,21	F2, M			0.6
0.8	▨			☒						0.8
1.0	▨		2	☒	75	10,13,15	V			1.0
1.2	▨	1.22				,4				1.2
1.4										1.4
1.6										1.6
1.8										1.8
2.0										2.0
2.2										2.2
2.4										2.4
2.6										2.6
2.8										2.8
3.0										3.0
3.2										3.2
3.4										3.4
3.6										3.6
3.8										3.8
4.0										4.0
4.2										4.2
4.4										4.4
4.6										4.6
4.8										4.8
5.0										5.0
5.2										5.2
5.4										5.4
5.6		Refusal on Top of Inferred Bedrock Surface								5.6
5.8										5.8
6.0										6.0

End of Borehole at 1.22 meters

NOTES:

masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:

M	= Metals	F1	= PHC F1
I	= Inorganics	F2	= PHC F2-F4
V	= VOC's	PCB	= PCB's
B	= BTEX	Pes	= Pesticides
P	= PAH		

SAMPLE TYPE:

- ▮ Auger Cuttings
- ☒ Split Spoon

Note: Any decisions/actions made by a third party based on this log are the sole responsibility of the third party. ARCADIS accepts no liability for third party decisions/actions made based on this log.

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LOG NO: BH-4-21

BOREHOLE LOG

PAGE 1 OF 1

PROJECT NUMBER 30054299

CLIENT University of Ottawa

PROJECT NAME Geotechnical and Environmental Assessment

PROJECT LOCATION RGN Campus- 451 Smyth Road, Ottawa, Ontario

DRILLING CONTRACTOR Downing

PROJECTION NAD83

DRILLING METHOD Hollow Stem

NORTHING 45.403161

EASTING 75.651442

DRILL DATE 21-1-27

GROUND ELEVATION

TOC ELEVATION

LOGGED BY Lennart de Groot

CHECKED BY Troy Austrins

GROUND WATER LEVEL ▼ ( )

HOLE DIAMETER 4 "

WELL DIAMETER

GROUND WATER ELEVATION ▼ mbgs ( )

DEPTH (m)	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS	WELL DIAGRAM
0.08	**ASPHALT**		▮					
0.30	**SAND (Fill)**, gravel, light grayish brown, coarse grained							
1.13	**SAND (Fill)**, gravel, light grayish brown, coarse grained, brick debris	1	☒	75	29,11, 46,24	0.8 ppm	P, F1, F2, M, V	
	Refusal on Top of Inferred Bedrock Surface							

End of Borehole at 1.13 meters

NOTES:

masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:

M = Metals
I = Inorganics
V = VOC's
B = BTEX
P = PAH
F1 = PHC F1
F2 = PHC F2-F4
PCB = PCB's
Pes = Pesticides

SAMPLE TYPE:

▮ Auger Cuttings
☒ Split Spoon

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LOG NO: BH-5-21

BOREHOLE LOG

PAGE 1 OF 1

PROJECT NUMBER 30054299
CLIENT University of Ottawa
PROJECT NAME Geotechnical and Environmental Assessment
PROJECT LOCATION RGN Campus- 451 Smyth Road, Ottawa, Ontario

DRILLING CONTRACTOR Downing
PROJECTION NAD83
DRILLING METHOD Hollow Stem
NORTHING 45.40305 **EASTING** 75.65189
DRILL DATE 21-1-26
GROUND ELEVATION ______ **TOC ELEVATION** ______
LOGGED BY Lennart de Groot **CHECKED BY** Troy Austrins
GROUND WATER LEVEL ▼ ( )
HOLE DIAMETER 4 " **WELL DIAMETER** ______
GROUND WATER ELEVATION ▼ mbgs ( )

DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS	WELL DIAGRAM	DEPTH (m)
		ASPHALT 0.08		▮						
0.2 0.4 0.6	☒	**SAND (Fill)**, trace weathered rock, light brown grayish, coarse grained, dry 0.61	1	☒	50	50,39, 44,50	0 ppm	P, F1, F2, M, V		0.2 0.4 0.6
0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0		Refusal on Top of Inferred Bedrock Surface								0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0

End of Borehole at 0.61 meters

NOTES:
masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:
M = Metals
I = Inorganics
V = VOC's
B = BTEX
P = PAH
F1 = PHC F1
F2 = PHC F2-F4
PCB = PCB's
Pes = Pesticides

SAMPLE TYPE:
▮ Auger Cuttings
☒ Split Spoon

Note: Any decisions/actions made by a third party based on this log are the sole responsibility of the third party. ARCADIS accepts no liability for third party decisions/actions made based on this log.

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LOG NO: MW-6-21

BOREHOLE LOG

PAGE 1 OF 1

PROJECT NUMBER 30054299

PROJECT NAME Geotechnical and Environmental Assessment

CLIENT University of Ottawa

PROJECT LOCATION RGN Campus- 451 Smyth Road, Ottawa, Ontario

DRILLING CONTRACTOR Downing

DRILLING METHOD Hollow Stem -HQ coring

DRILL DATE 21-1-27

LOGGED BY Lennart de Groot **CHECKED BY** Troy Austrins

HOLE DIAMETER 4 " **WELL DIAMETER** 2 "

PROJECTION NAD83

NORTHING 45.402742 **EASTING** 75.651528

GROUND ELEVATION **TOC ELEVATION** 98.895

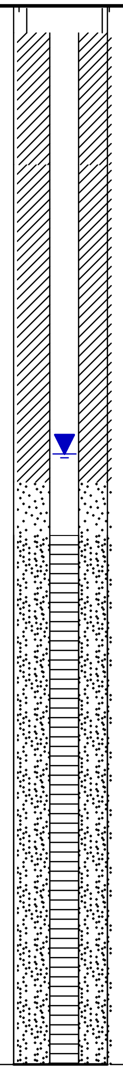
GROUND WATER LEVEL ▼ 2.58 mbtoc (2021-02-02)

GROUND WATER ELEVATION ▼ mbgs (2021-02-02)

DEPTH (m)	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY %	BLOW COUNTS	RKI READINGS	LAB ANALYSIS
0.08	**ASPHALT**		▮				
	SILTY SAND (Fill), dark brownish black, coarse grained, dry						
0.30	**SILTY SAND (Fill)**, dark brownish black, coarse grained, dry	1	☒	50			F1, F2, M, V, P
0.91	**SAND**, rock fragments, light brown, coarse grained, dry	2	☒	50	16,39, 59,6	0 ppm	
1.22	SHALE, moderately weathered, thinly bedded, horizontal, gray,Layer RQD = 10%	3	▯	95			
2.33	SHALE, thinly bedded, horizontal, gray to black, Layer RQD =48%	4	▯	100			
3.88	SHALE, thin to medium bedded, horizontal, gray to black, Layer RQD = 54%	5	▯	100			
5.40	SHALE, thin to medium bedded, horizontal, gray to black, Layer RQD = 79%	6	▯	100			
6.10	End of Borehole at 6.10meters						

WELL DIAGRAM

Casing Type: Flushmount



NOTES:

masl = Meters Above Sea Level
mbgs = Meters Below Ground Surface
toc = Top of Casing
ppm = Parts Per Million
LEL = Lower Explosive Limit

LAB ANALYSIS:

M = Metals
I = Inorganics
V = VOC's
B = BTEX
P = PAH
F1 = PHC F1
F2 = PHC F2-F4
PCB = PCB's
Pes = Pesticides

SAMPLE TYPE:

▮ Auger Cuttings
☒ Split Spoon
▯ Rock Core

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APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6804-1 - TEST HOLE LOCATION PLAN

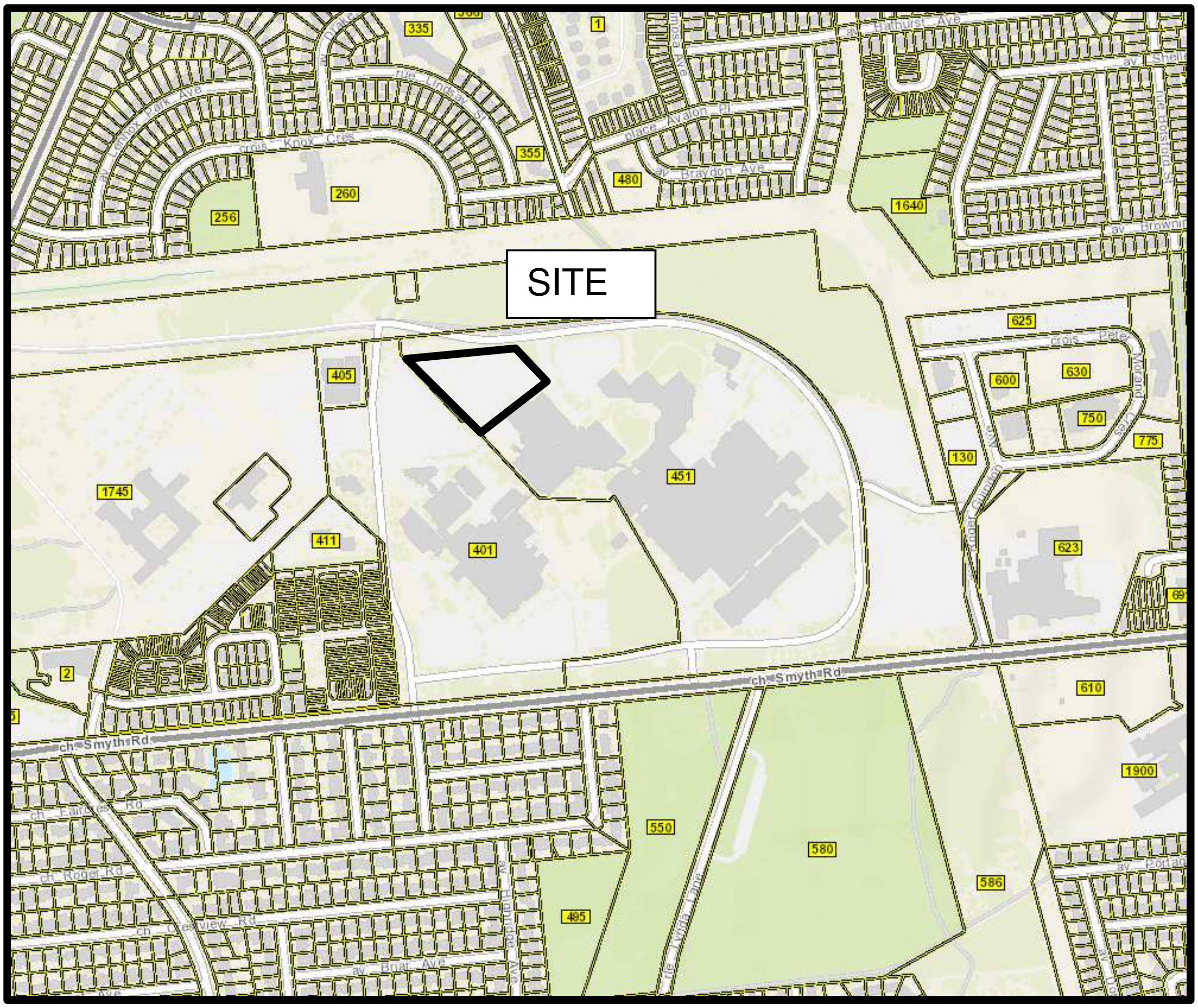


FIGURE 1

KEY PLAN

PATERSON GROUP

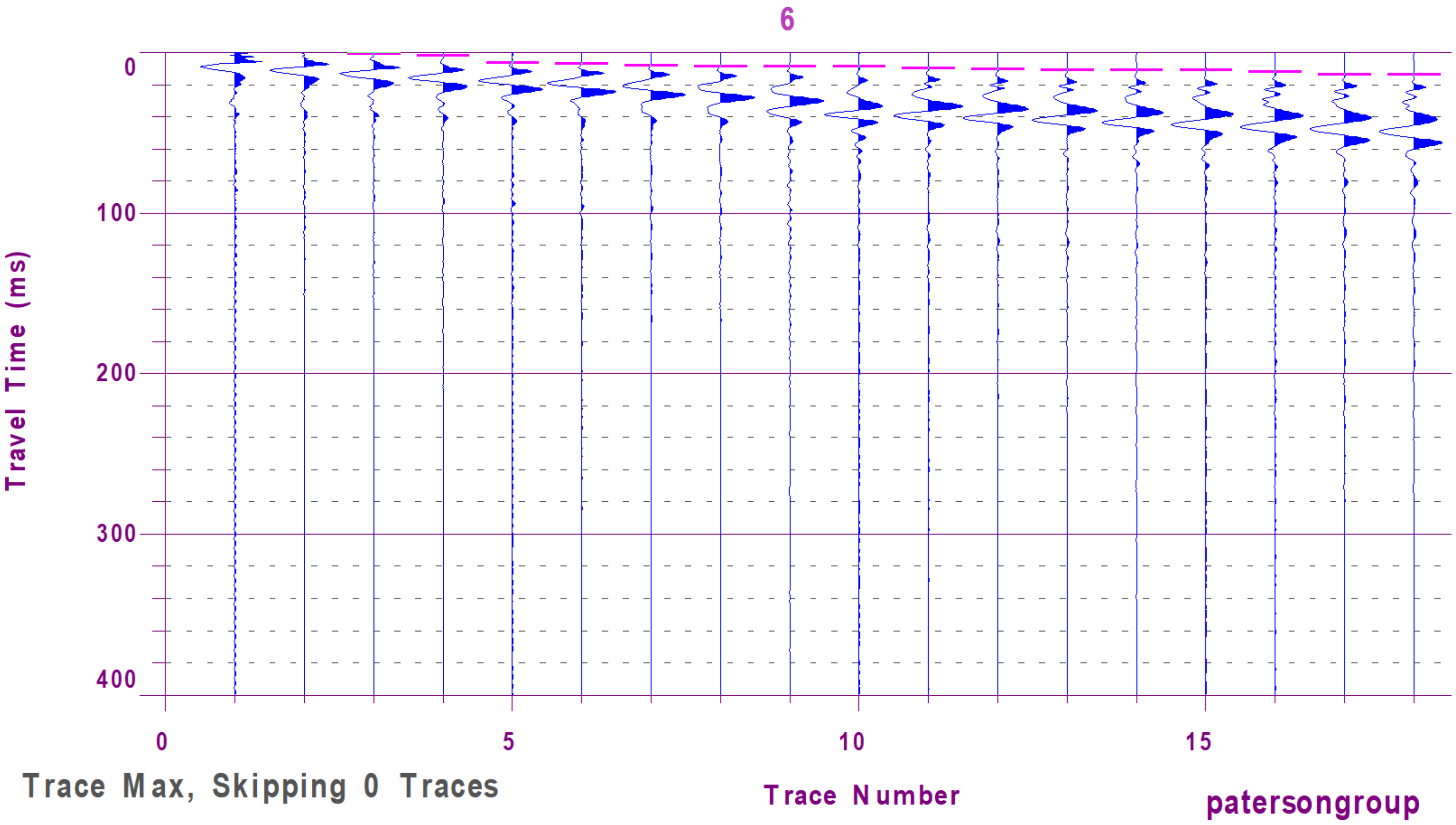


Figure 2 – Shear Wave Velocity Profile at Shot Location -1.5 m

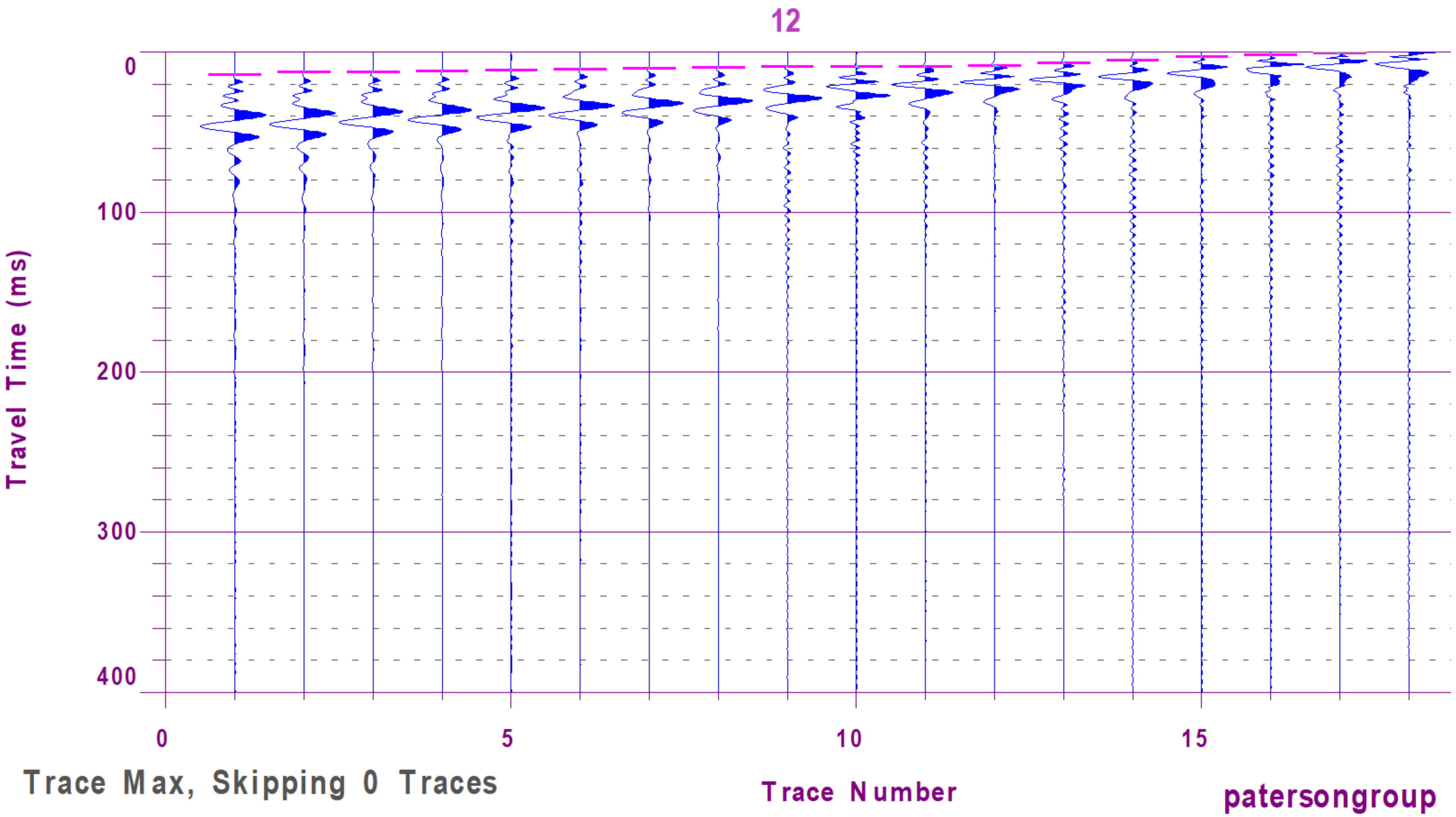
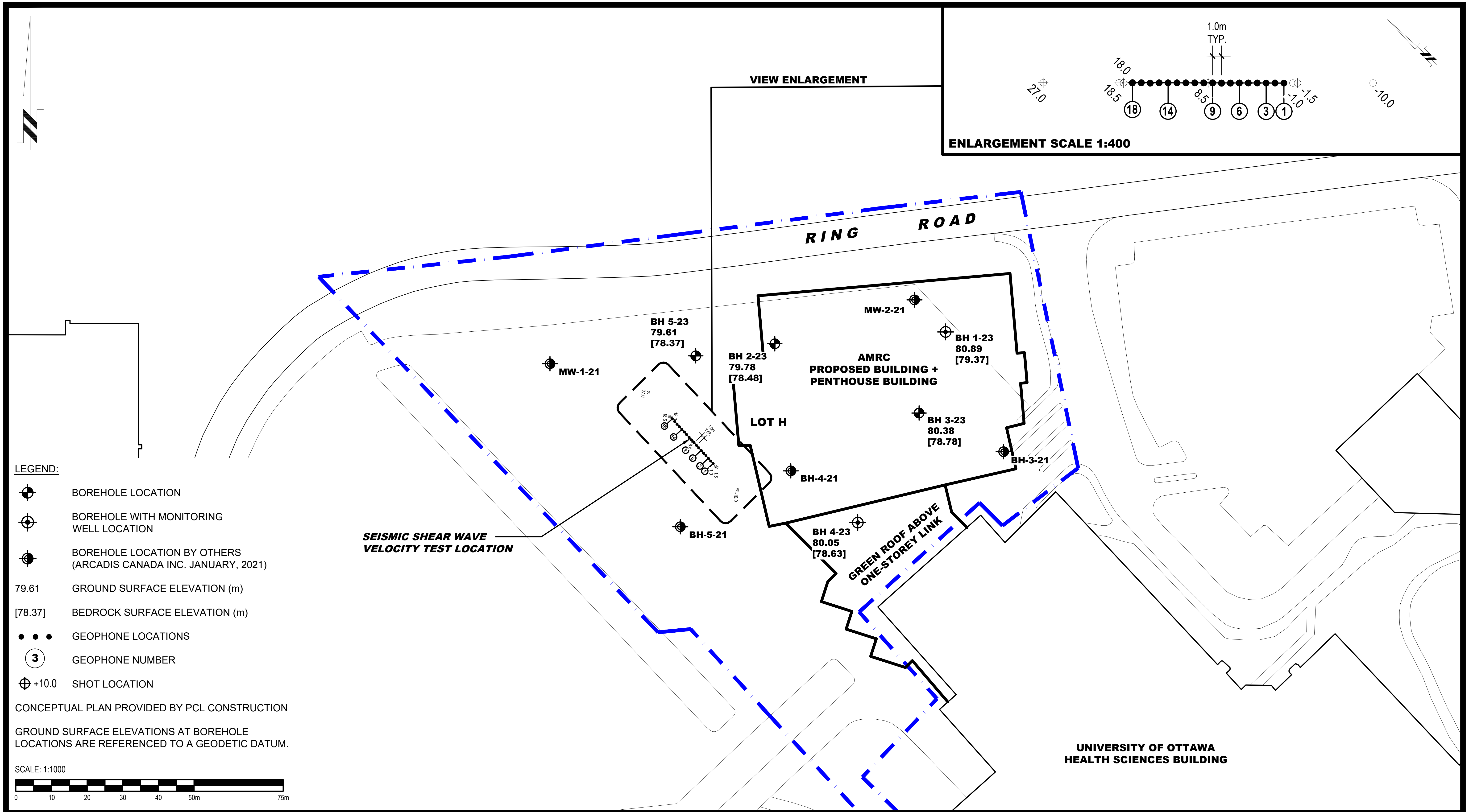


Figure 3 – Shear Wave Velocity Profile at Shot Location 18 m



LEGEND:

◉	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
◈	BOREHOLE LOCATION BY OTHERS (ARCADIS CANADA INC. JANUARY, 2021)
79.61	GROUND SURFACE ELEVATION (m)
[78.37]	BEDROCK SURFACE ELEVATION (m)
●–●–●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⌖ +10.0	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY PCL CONSTRUCTION

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

PCL CONSTRUCTORS INC.
SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
451 SMYTH ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 09/2023
Drawn by: YA	Report No.: PG6804-1
Checked by: NS	Dwg. No.: PG6804-1
Approved by: JV	Revision No.:

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