# Geotechnical Investigation

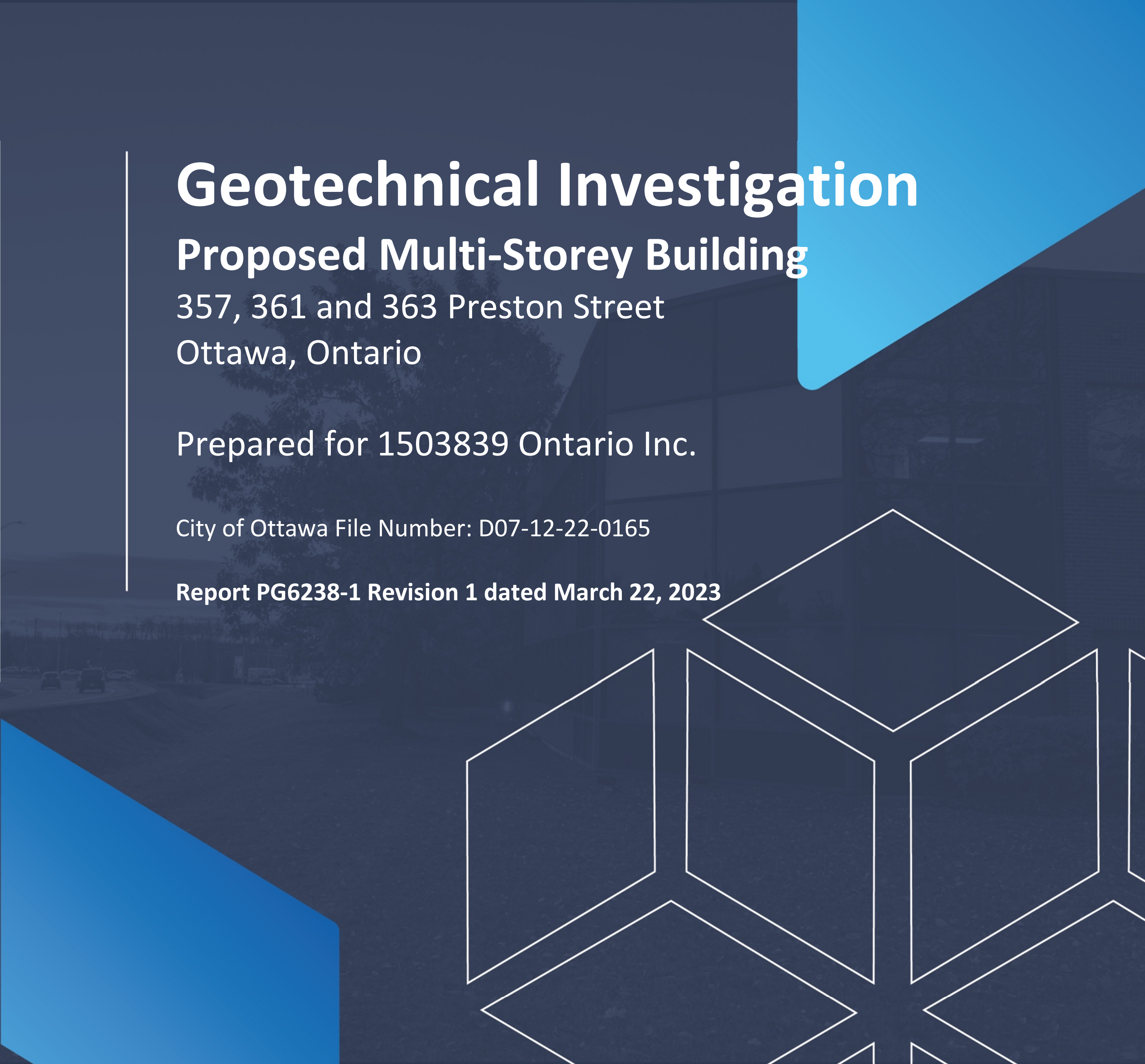
## Proposed Multi-Storey Building

357, 361 and 363 Preston Street
Ottawa, Ontario

Prepared for 1503839 Ontario Inc.

City of Ottawa File Number: D07-12-22-0165

**Report PG6238-1 Revision 1 dated March 22, 2023**



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# 1.0 Introduction

Paterson Group (Paterson) was commissioned by 1503839 Ontario Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 357, 361 and 363 Preston Street in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

- ☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

# 2.0 Proposed Development

Drawings were not available at the time of preparation of this report. However, based on discussions with the client, it is understood that the proposed development will consist of a multi-storey building with one underground parking level which will extend approximately over the entire site.

It is expected that the existing buildings will be demolished in support of the proposed development.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the current geotechnical investigation was carried out on May 31, 2022 and consisted of advancing a total of 4 boreholes to a maximum depth of 6.1 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The approximate locations of the boreholes are shown on Drawing PG6238-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance drill rig operated by a two-person crew. The drilling procedure consisted of augering and coring to the required depths at the selected locations and sampling the overburden and bedrock. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

### Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, 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.

Bedrock samples were recovered from boreholes BH 1-22 to BH 3-22 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run).

The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the bedrock quality.

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

### Groundwater

Groundwater monitoring wells were installed in boreholes BH1-22 to BH3-22 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

### Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted 32 mm diameter PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

## 3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevation at each borehole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG6238-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

## 3.4 Analytical Testing

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

# 4.0 Observations

## 4.1 Surface Conditions

The subject site consists of 3 contiguous properties and is currently occupied by a single-storey commercial building and a two-storey mixed-use building. The remaining portion of the site consists of parking areas covered with asphaltic pavement or gravel.

The site is bordered by Aberdeen Street to the north, an at-grade parking area to the east, Preston Street to the west, and residential properties to the south. The existing ground surface is relatively flat and at grade with surrounding roadways and properties at approximate geodetic elevation 61 m.

## 4.2 Subsurface Profile

### Overburden

Generally, the subsurface profile encountered at the borehole locations consists of a thin layer of asphaltic pavement and/or fill underlain by a glacial till deposit. The fill was observed to generally consist of a layer of crushed stone and gravel underlain by brown silty sand with gravel, trace of clay, asphalt and organics. The fill was observed to depths ranging between approximately 0.7 to 1.8 m below existing ground surface.

A glacial till deposit was encountered underlying the fill, and was generally observed to consist of a compact to dense, brown silty sand with gravel, cobbles, and boulders. Refusal to augering was encountered at all boreholes at an approximate depth range between 1.5 and 2.1 m below existing ground surface.

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

### Bedrock

The bedrock was cored in boreholes BH 1-22 to BH 3-22 to a maximum depth of 6.1 m, and was observed to consist of limestone. The RQD values of the bedrock were observed to range from 54 to 100%, which corresponds to bedrock which is fair to excellent in quality. The recovery values equaled 100% at all boreholes.

Based on available geological mapping, the bedrock in the subject area consists of interbedded limestone and shale of the Verulam Formation, with an overburden drift thickness of 2 to 3 m.

## 4.3 Groundwater

Groundwater levels were measured within the installed monitoring wells. The measured groundwater levels noted at the time are presented in Table 1 below.

**Table 1 - Summary of Groundwater Levels**

| Test Hole Number | Ground Surface Elevation (m) | Measured Groundwater Level | | Recording Date |
|---|---|---|---|---|
| | | Depth (m) | Elevation (m) | |
| BH 1-22 | 61.20 | 3.97 | 57.23 | June 3, 2022 |
| BH 2-22 | 61.09 | 2.54 | 58.55 | June 3, 2022 |
| BH 3-22 | 61.45 | 2.75 | 58.70 | June 3, 2022 |

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

The gong-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 2 to 3 m below the existing ground surface.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed building and associated stormwater management structures. It is recommended that foundation support for the proposed building and stormwater management structure consist of conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground level. The above and other considerations are discussed in the following paragraphs.

## 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, paved areas and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

### Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

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

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding structures.

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

### Vibration Considerations

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

The following construction equipment could be the source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is 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 that 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.

### Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness.
Fill placed beneath the building and paved areas should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill and beneath exterior parking areas, where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s 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 composite drainage membrane.

## 5.3 Foundation Design

### Conventional Spread Footings

Footings placed directly on clean, surface sounded bedrock can be designed using a bearing resistance value at ultimate limit states (ULS) of **2,500 kPa,** incorporating a geotechnical resistance factor of 0.5.

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

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

### Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. Weathered bedrock will require a lateral support zone of 1H:1V (or flatter).

## 5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 2 and 3, which are presented in Appendix 2 of this report.

### Field Program

The seismic array testing location was placed as presented in Drawing PG6238-1 - Test Hole Location Plan, attached to the present report.

Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 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 trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations were also completed in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 1.5 and 1.0 m away from the first geophone, 1.5 and 1.0 m away from the last geophone, and at the centre of the seismic array.

## Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, $V_{s30}$, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

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

It is understood that the overburden will be completely removed as part of the proposed building, and footings will be placed directly on the bedrock surface.

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

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}012\ m/s}\right)}$$

$$V_{s30=}\ 2{,}012\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity $V_{s30}$, is 2,012 m/s. Therefore, a **Site Class A** is applicable for the design of proposed buildings bearing on the bedrock surface, 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

The overburden will generally be removed from the proposed building footprint, leaving the bedrock as the founding medium for the underground parking level. The recommended pavement structure noted in Section 5.7 will generally be applicable for the underground parking level, however, if storage or other uses of the lower level are proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab consist of 19 mm of clear crushed stone.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. This is discussed further in Section 6.1.

## 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/$m^3$.

### Lateral Earth Pressures

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

$K_o$ = at-rest earth pressure coefficient of the applicable retained soil (0.5)
$\gamma$ = 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 ($\Delta P_{AE}$).

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

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

The peak ground acceleration, ($a_{max}$), for the Ottawa area is 0.32 g according to the OBC 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 the OBC 2012.

## 5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%.

The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structures presented in Tables 3 and 4 should be used for car only and at grade access lanes parking areas, if required at the subject site.

**Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level**

| Thickness (mm) | Material Description |
|---|---|
| 150 | **Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment) |
| 300 | **BASE** – OPSS Granular A |
| **SUBGRADE** – Existing imported fill, or OPSS Granular B Type I or II material material placed over bedrock. | |

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

**Table 3 - Recommended Pavement Structure – Car Only Parking Area**

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

**Table 4 - Recommended Pavement Structure – Access Lanes**

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

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

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

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

### Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated PVC pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system or vertical bedrock face and extending to a series of drainage sleeves inlets through the building foundation wall at the footing/foundation wall interface.

The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the underslab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower underground area.

A waterproofing system should be provided for any elevator pits (pit bottom and walls).

The maximum groundwater flow rate from the foundation drainage system, including the underslab drainage, will be 200,000 L/day.

### Underslab Drainage

Underslab drainage is recommended to control water infiltration for the basement area. For preliminary design purposes, it is recommended that 150 mm diameter perforated PVC pipes be placed at 6 m spacing. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

### Foundation Backfill

Where sufficient space is available for conventional backfilling, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

## 6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 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 in this regard.

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided at entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided to these areas.

## 6.3 Excavation Side Slopes

The side slopes of shallow anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled.

### Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

### Rock Stabilization

Excavation side slopes in sound bedrock can be completed with almost vertical side walls. A minimum of 1 m horizontal ledge should be left between the bottom of the overburden and the top of the sound bedrock surface to provide an area for potential sloughing.

Horizontal rock anchors, chainlink fencing, and/or shotcrete may be required at specific locations to stabilize the bedrock excavation face and to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation operations and determined by Paterson at the time of construction.

### Temporary Shoring

Temporary shoring may be required to support the overburden soil where insufficient room is available for open cut methods. If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. 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. 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 earth pressures acting on the shoring system may be calculated with the following parameters.

**Table 5 – Soil Parameters for Shoring System Design**

| **Parameters** | **Values** |
|---|---|
| Active Earth Pressure Coefficient ($K_a$) | 0.33 |
| Passive Earth Pressure Coefficient ($K_p$) | 3 |
| At-rest Earth Pressure Coefficient ($K_o$) | 0.5 |
| Total Unit Weight (γ), kN/m$^3$ | 20 |
| Submerged Unit Weight (γ'), kN/m$^3$ | 13 |

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

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

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

## 6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS

Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

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 provided 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 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

## 6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

### Groundwater Control for Building Construction

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

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

### Impacts on Neighbouring Structures

Based on the subsurface conditions encountered at the subject site, it is anticipated that the adjacent structures are founded on bedrock or the glacial till deposit.

Therefore, no adverse effects from short term and/or long term dewatering are expected for the surrounding structures.

## 6.6 Winter Construction

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

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

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

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

## 6.7 Corrosion Potential and Sulphate

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

## 6.8 Subsurface Stormwater Management Tank

It is understood that a stormwater management tank will be located in the lower western portion of the subject site. It is further understood that the founding depth of the stormwater management tank will be below the bedrock elevation encountered. It is recommended that the stormwater management tank be founded on clean, surface sounded surface and therefore can be designed using a bearing resistance value at ultimate limit states (ULS) of **2,500 kPa,** incorporating a geotechnical resistance factor of 0.5.

It is further recommended that the stormwater management tank be provided with a waterproofing system. Application of a spray applied elastomeric waterproofing membrane such as Sprayseal NS-F300+GE or equivalent to all interior faces of the stormwater management tank is typical. It is recommended but not required that a Xypex additive be included for the concrete in the construction of the stormwater management tank, in order to minimize permeability of the concrete.

# 7.0 Recommendations

It is a requirement for the foundation data provided herein to be applicable that the following material testing, and observation program be performed by the geotechnical consultant.

☐ Review of the bedrock stabilization and excavation requirements.

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

☐ Sampling and testing of the concrete and fill materials.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by Paterson.

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

# 8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 1503839 Ontario Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

**Paterson Group Inc.**

Otillia McLaughlin, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Mar. 22, 2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

**Report Distribution:**

- ☐ 1503839 Ontario Inc. (email copy)
- ☐ Paterson Group (1 copy)

# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

**patersongroup** Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Development - 357, 361 & 363 Preston St.**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** May 31, 2022

**FILE NO.** PG6238

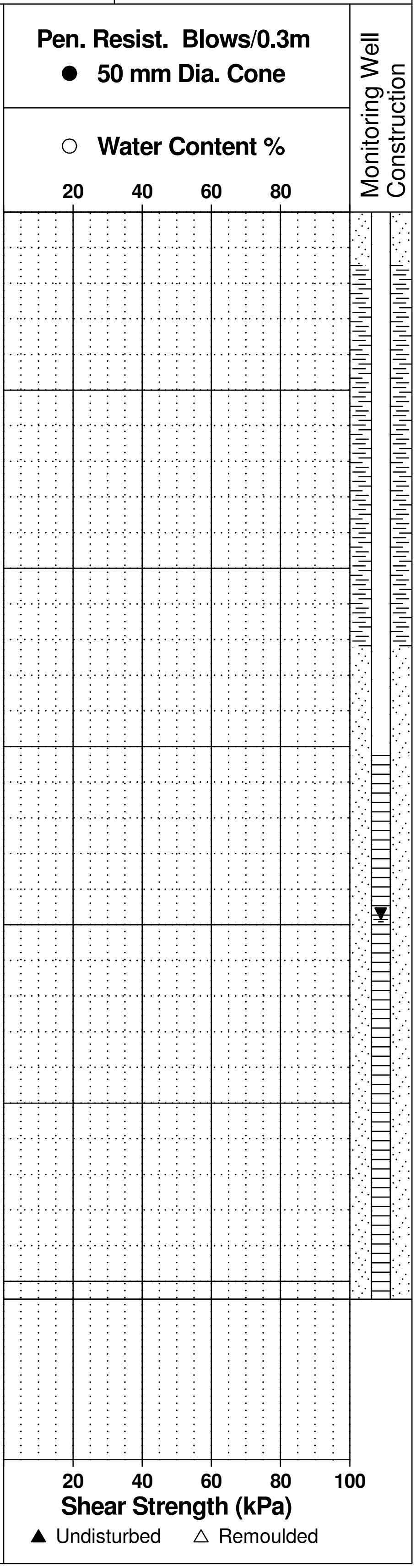
**HOLE NO.** BH 1-22

| SOIL DESCRIPTION | Depth to base (m) |
|---|---|
| GROUND SURFACE | |
| **FILL:** Crushed stone and gravel | 0.13 |
| **FILL:** Brown silty sand | 1.45 |
| **FILL:** Brown silty sand with gravel, trace clay tile, asphalt and wood chips | 1.83 |
| **GLACIAL TILL:** Dense, brown silty sand with gravel, cobbles and boulders | 2.16 |
| **BEDROCK:** Good to excellent quality, grey limestone | 6.10 |
| End of Borehole | |

(GWL @ 3.97m - June 3, 2022)

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| SS | 2 | 58 | 16 |
| SS | 3 | 25 | 22 |
| RC | 1 | 100 | 89 |
| RC | 2 | 100 | 100 |
| RC | 3 | 100 | 95 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 61.20 |
| 1 | 60.20 |
| 2 | 59.20 |
| 3 | 58.20 |
| 4 | 57.20 |
| 5 | 56.20 |
| 6 | 55.20 |



**patersongroup** Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Development - 357, 361 & 363 Preston St.**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** May 31, 2022

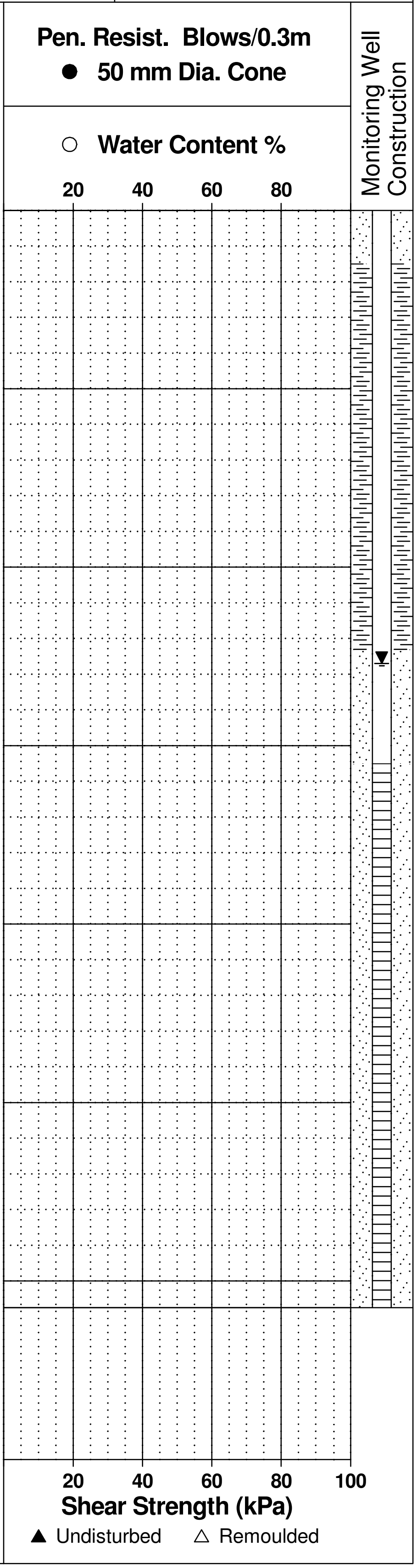
**FILE NO.** **PG6238**

**HOLE NO.** **BH 2-22**

| SOIL DESCRIPTION | Depth to base (m) |
|---|---|
| **GROUND SURFACE** | |
| Asphaltic concrete | 0.05 |
| **FILL:** Crushed stone | 0.69 |
| **FILL:** Brown silty sand with gravel, some crushed stone | 1.07 |
| **GLACIAL TILL:** Dense, brown silty sand with gravel, cobbles and boulders | 1.68 |
| **BEDROCK:** Excellent quality, grey limestone | 6.15 |
| End of Borehole | |
| (GWL @ 2.54m - June 3, 2022) | |

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| SS | 2 | 42 | 45 |
| SS | 3 | 67 | 50+ |
| RC | 1 | 100 | 90 |
| RC | 2 | 100 | 100 |
| RC | 3 | 100 | 100 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 61.09 |
| 1 | 60.09 |
| 2 | 59.09 |
| 3 | 58.09 |
| 4 | 57.09 |
| 5 | 56.09 |
| 6 | 55.09 |



# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Development - 357, 361 & 363 Preston St.**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** May 31, 2022

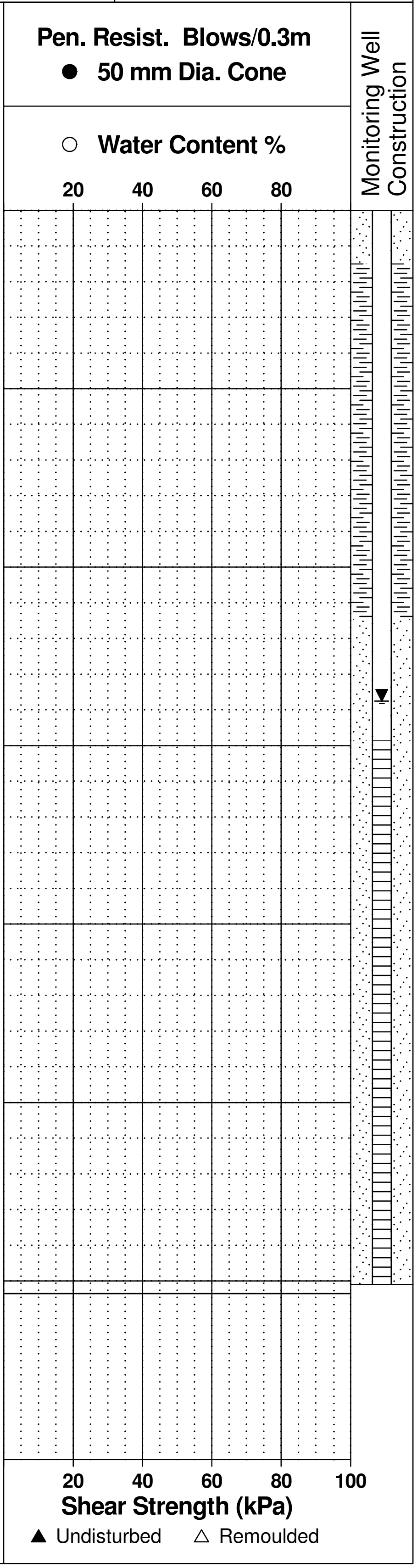
**FILE NO.** **PG6238**

**HOLE NO.** **BH 3-22**

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | |
| Asphaltic concrete | 0.05 | | | | |
| **FILL:** Crushed stone | 0.69 | AU | 1 | | |
| **FILL:** Black/brown silty sand with gravel, occasional cobbles | 1.22 | SS | 2 | 32 | 34 |
| **GLACIAL TILL:** Dense, brown silty sand wih gravel, cobbles and boulders | 1.58 | | | | |
| **BEDROCK:** Fair to excellent quality, grey limestone | 6.07 | RC | 1 | 100 | 54 |
| | | RC | 2 | 100 | 100 |
| | | RC | 3 | 100 | 100 |
| End of Borehole | | | | | |

(GWL @ 2.75m - June 3, 2022)

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 61.45 |
| 1 | 60.45 |
| 2 | 59.45 |
| 3 | 58.45 |
| 4 | 57.45 |
| 5 | 56.45 |
| 6 | 55.45 |



**patersongroup** Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 357, 361 & 363 Preston St.
Ottawa, Ontario

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** May 31, 2022

**FILE NO.** PG6238

**HOLE NO.** BH 4-22

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone<br>○ Water Content % | Monitoring Well Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 61.56 | | |
| Asphaltic concrete 0.05 | | | | | | | | | |
| **FILL:** Brown silty sand with gravel, trace topsoil 0.69 | | AU | 1 | | | | | | |
| **GLACIAL TILL:** Loose to compact, brown silty sand with gravel, occasional cobbles 1.50 | | SS | 2 | 58 | 8 | 1 | 60.56 | | |
| End of Borehole<br>Practical refusal to augering at 1.50m depth | | | | | | | | | |

20 40 60 80

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

# SYMBOLS AND TERMS

## SOIL DESCRIPTION

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

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

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

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

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

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

# SYMBOLS AND TERMS (continued)

## SOIL DESCRIPTION (continued)

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

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

## ROCK DESCRIPTION

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

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

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

## SAMPLE TYPES

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

# SYMBOLS AND TERMS (continued)

## GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 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 $p'_c$)

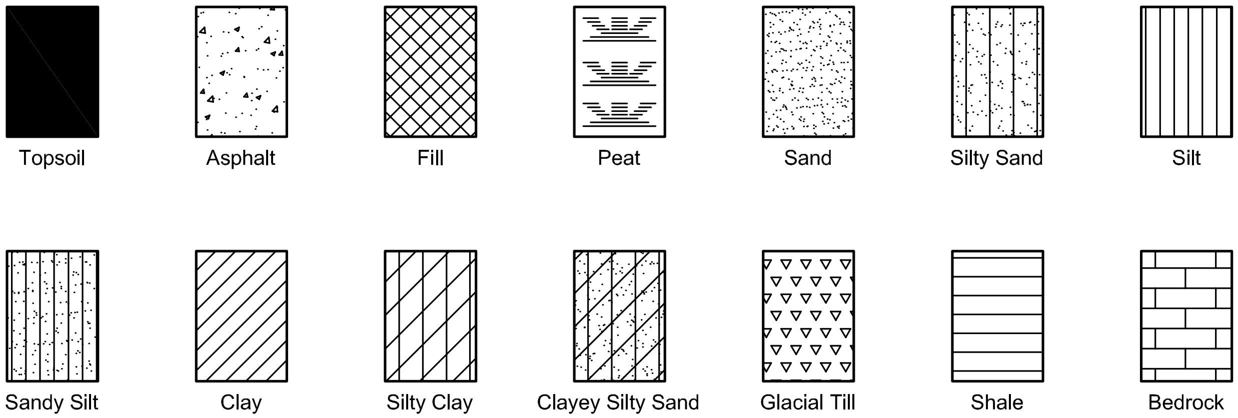
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

## 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.

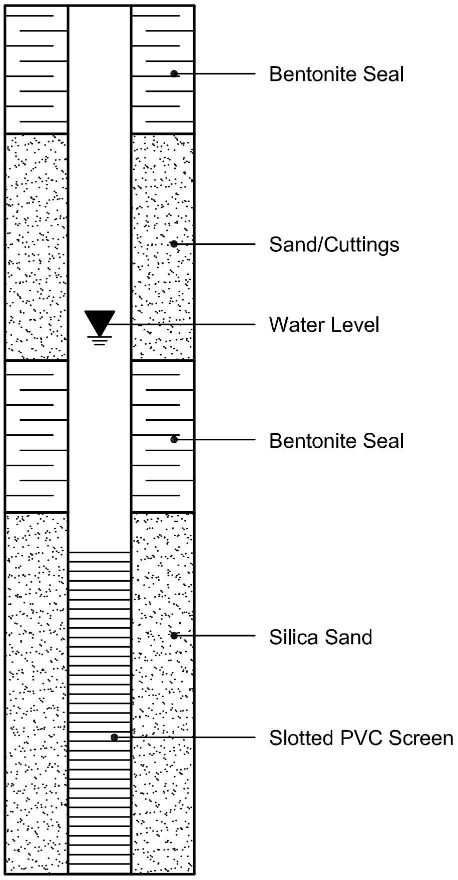
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

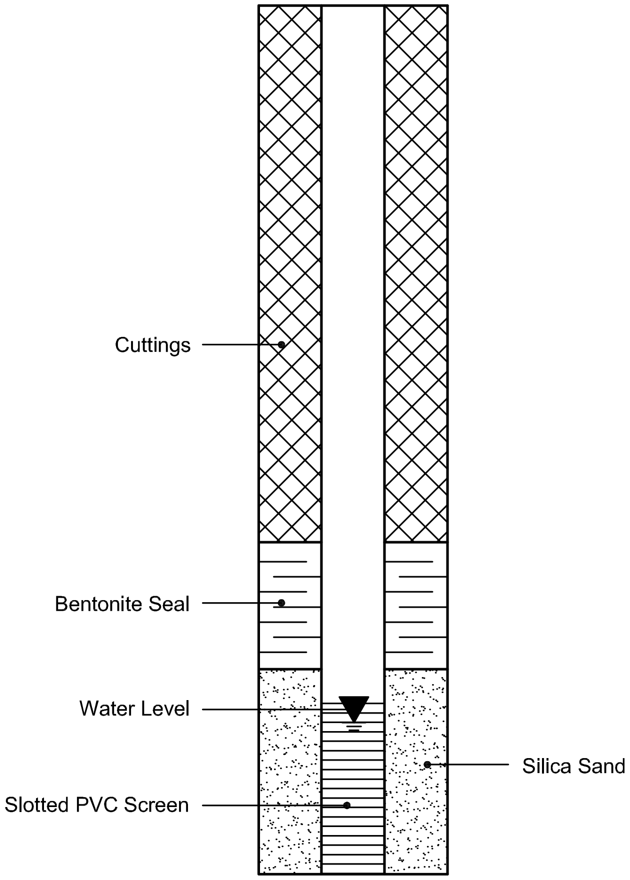


## MONITORING WELL AND PIEZOMETER CONSTRUCTION

### MONITORING WELL CONSTRUCTION



### PIEZOMETER CONSTRUCTION



Certificate of Analysis

**Client: Paterson Group Consulting Engineers**

**Client PO: 54828**

Report Date: 09-Jun-2022

Order Date: 1-Jun-2022

**Project Description: PG6238**

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH2-22 SS2 | - | - | - |
| | **Sample Date:** | 31-May-22 09:00 | - | - | - |
| | **Sample ID:** | 2223415-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 92.4 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.97 | - | - | - |
| Resistivity | 0.10 Ohm.m | 37.1 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 44 | - | - | - |
| Sulphate | 5 ug/g dry | 37 | - | - | - |

# APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6238-1 – TEST HOLE LOCATION PLAN

# FIGURE 1

## KEY PLAN

PATERSON GROUP

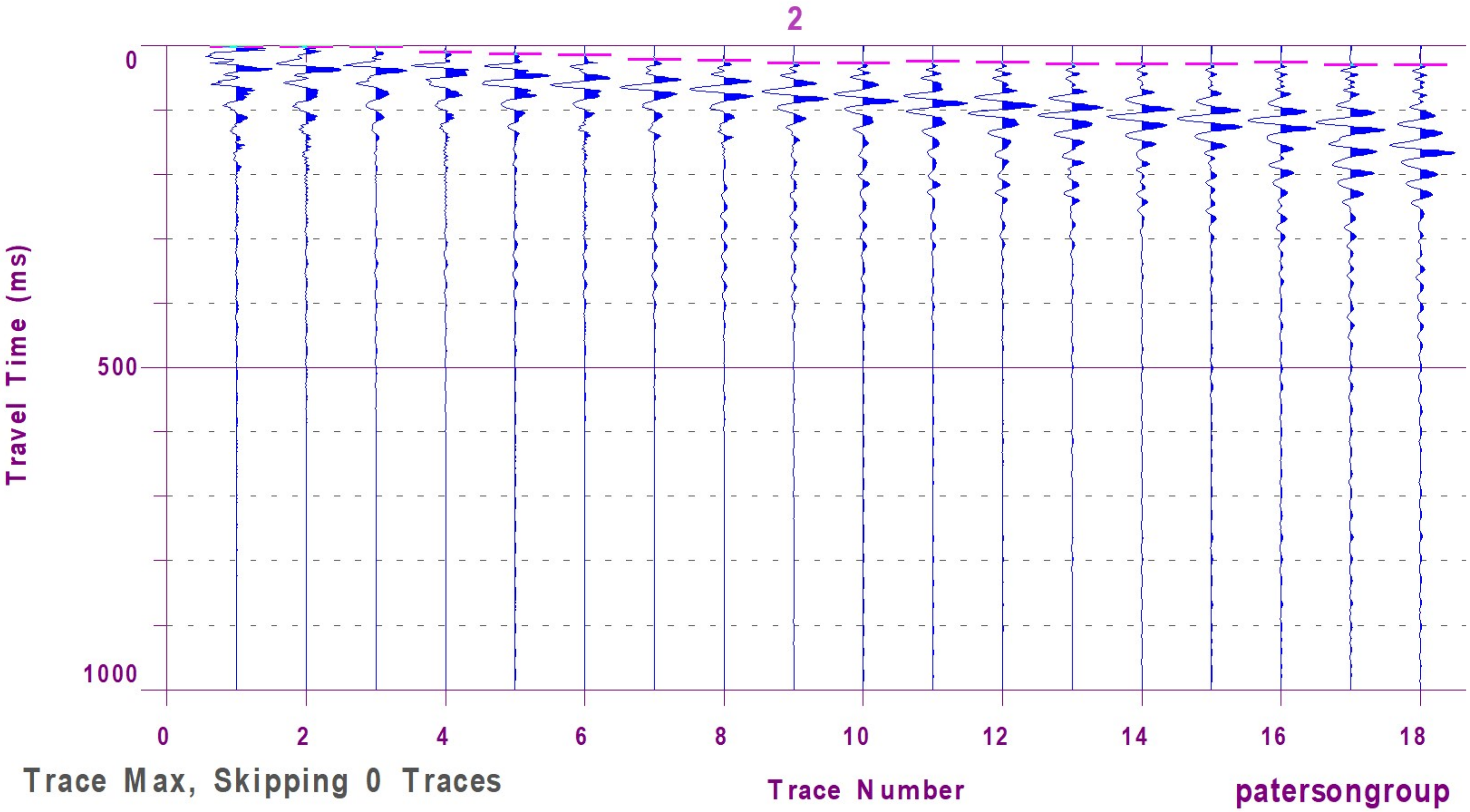


Figure 2 – Shear Wave Velocity Profile at Shot Location -1.5 m

PATERSON GROUP

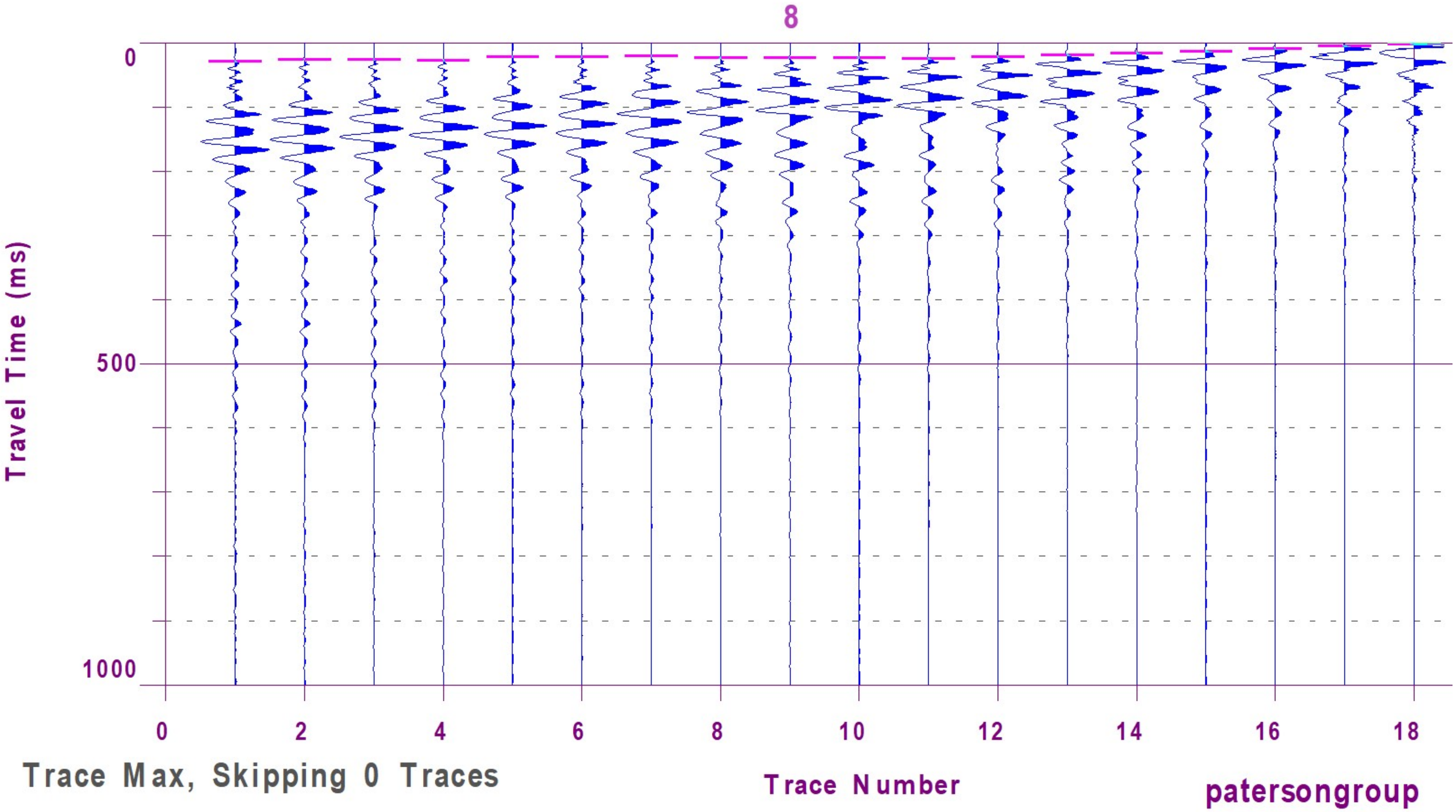
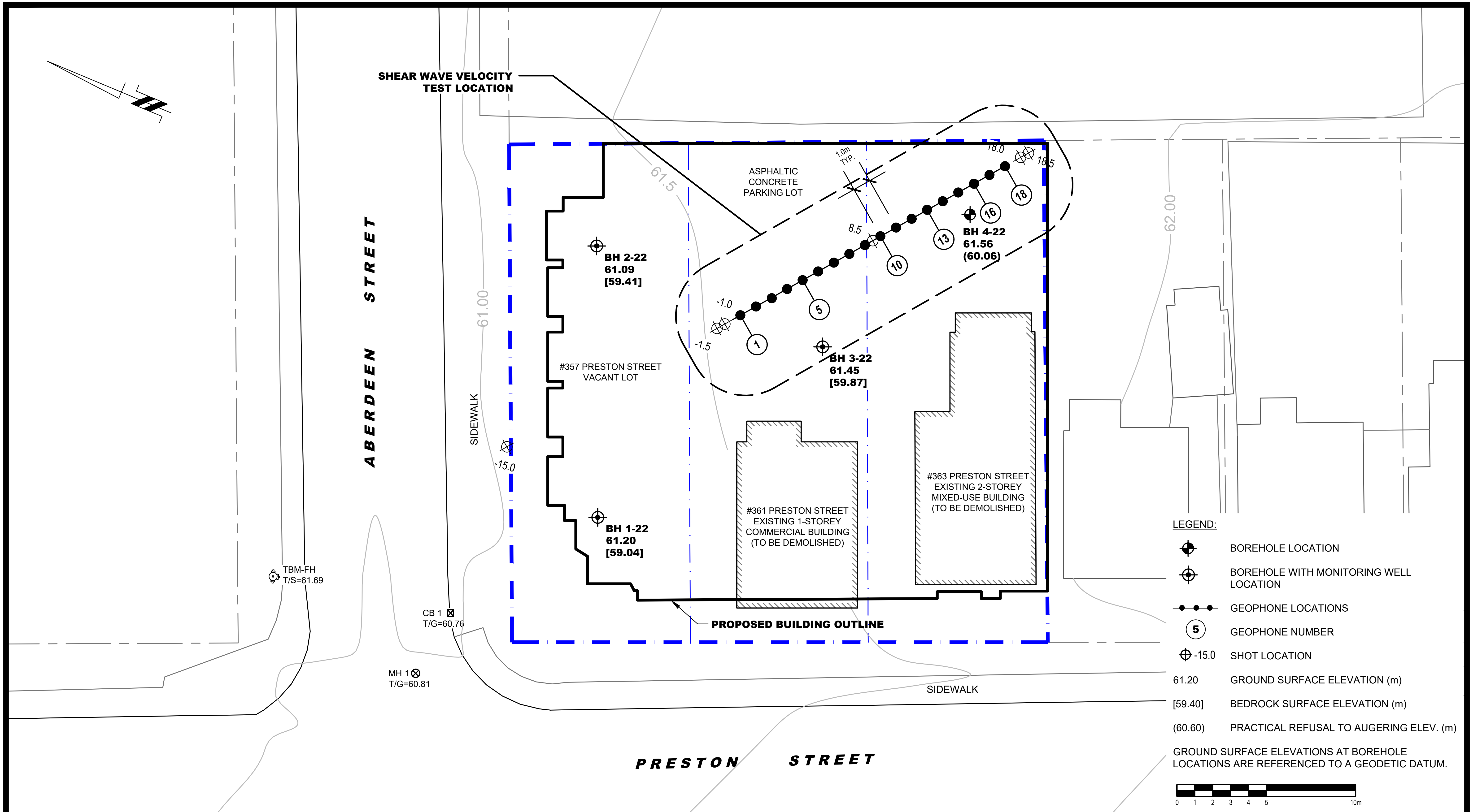


Figure 3 – Shear Wave Velocity Profile at Shot Location 18 m



LEGEND:

| | |
|---|---|
| ⊕ | BOREHOLE LOCATION |
| ⊕ | BOREHOLE WITH MONITORING WELL LOCATION |
| •–•–• | GEOPHONE LOCATIONS |
| ⑤ | GEOPHONE NUMBER |
| ⊕ -15.0 | SHOT LOCATION |
| 61.20 | GROUND SURFACE ELEVATION (m) |
| [59.40] | BEDROCK SURFACE ELEVATION (m) |
| (60.60) | PRACTICAL REFUSAL TO AUGERING ELEV. (m) |

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

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| | | | |
| | | | |
| 2 | PROPOSED BUILDING FOOTPRINT ADDED | 22/03/2023 | OM |
| 1 | SEISMIC SHEAR WAVE VELOCITY TEST ADDED TO PLAN | 26/06/2022 | FC |

1503839 ONTARIO INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY DEVELOPMENT
357, 361 & 363 PRESTON STREET

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

Scale: 1:200
Date: 06/2022
Drawn by: JM
Report No.: PG6238-1
Checked by: FC
Dwg. No.: PG6238-1
Approved by: SD
Revision No.: 2

11x17

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