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Geotechnical Engineering
Environmental Engineering
Hydrogeology
Materials Testing
Building Science
Rural Development Design
Retaining Wall Design
Noise and Vibration Studies

patersongroup.ca

July 11, 2022
Report: PG5707-1. Revision 1

St Laurent Volvo
1300 Michael Street
Ottawa, Ontario
K2E1B2

Attention: **John Mierins – St. Laurent Volvo**
Christine McCuiag – Q9 Planning & Design

Subject: **Geotechnical Investigation**
Proposed Building
1300 Michael Street – Ottawa

Dear Mr. Mierins,

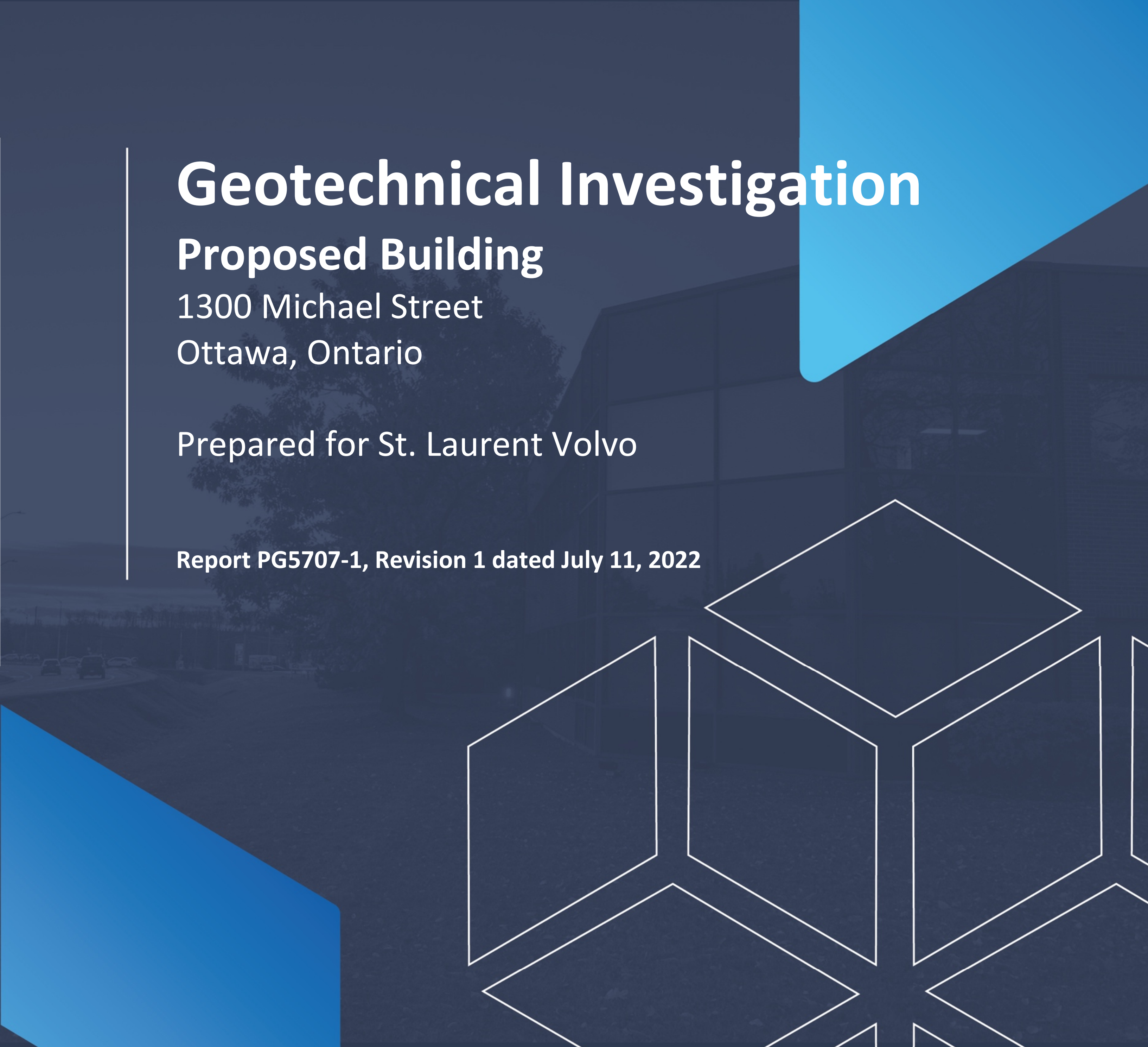
Please find enclosed 3 copies of Report PG5707-1 Revision 1, regarding the geotechnical investigation conducted for the aforementioned location.

We trust that this information is to your satisfaction.

Sincerely,

Paterson Group Inc.

David J. Gilbert, P.Eng.



PATERSON GROUP

Geotechnical Investigation

Proposed Building

1300 Michael Street
Ottawa, Ontario

Prepared for St. Laurent Volvo

Report PG5707-1, Revision 1 dated July 11, 2022

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Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Test Results
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1.0 Introduction

Paterson Group (Paterson) was commissioned by St. Laurent Volvo to conduct a geotechnical investigation for the proposed building to be located at 1300 Michael Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of a borehole program.

- Provide geotechnical recommendations pertaining 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

It is understood that the proposed building is to be located within the southeast corner of the property. The proposed building is expected to consist of a two-storey structure with associated parking areas. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on March 15, 2021 and consisted of advancing a total of three (3) boreholes (BH 1-21, BH 2-21, and BH 3-21) to a maximum depth of 2.3 m below existing ground surface. Previous geotechnical investigations were completed for the adjacent car dealership building. The test holes were distributed in a manner to provide general coverage of the subject building. The test hole locations are shown on Drawing PG5707-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance drill rig operated by a two-person crew. The drilling procedure consisted of drilling to the required depths at the selected locations, and sampling and testing the overburden. The test pit procedures consisted of excavating to the required depth at the selected location and sampling the overburden. 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 recovered from the boreholes and test pits. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. Soil samples were collected from the boreholes using auger flights and a 50 mm diameter split-spoon sampler. Grab samples were collected from the test pits at selected intervals. The depths at which the auger, split-spoon, rock cores and grab samples were recovered from the boreholes and test pits are shown as AU, SS and G respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Rock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to our laboratory.

The subsurface conditions observed in the test holes 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.

Sample Storage

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.

Groundwater

All boreholes from the current investigation were fitted with flexible piezometers to allow groundwater level monitoring. For the previous investigation, groundwater monitoring wells were installed in the boreholes, and the groundwater levels were observed in the open test pits at the time of the excavation.

Typical monitoring well construction details are described below:

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

The groundwater level measurements are shown on the Soil Profile and Test Data sheets and are tabulated in Subsection 4.3**.**

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5707-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures, one of which was collected from test hole BH2-SS3. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The ground surface across the subject site is relatively flat and at grade with the surrounding roadways and properties. A residential dwelling with an associated driveway currently occupies the site. The subject site is bordered by Michael Street to the northeast and northwest and by Parisien Street to the south.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the current test holes (BH 1-21, BH2-21, and BH 3-21) consists of a thin layer of asphaltic concrete and fill which extends to an approximate depth of 1.5 to 1.8m below existing ground surface. The upper portion of the fill consists of crushed stone with brown silty sand and the lower portion consists of brown silty sand with topsoil, clay and gravel. The fill is underlain by very stiff to stiff brown silty clay/silty sand deposit with traces of gravel. A layer of glacial till was encountered below the silty clay deposit at the location of BH 2-21. The glacial till was observed to consist of brown silty sand with fragmented shale, cobbles, and boulder. Refusal to augering was encountered in all the boreholes at an average depth of 2.2 m below existing ground surface.

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

Bedrock

From the current investigation, bedrock was encountered in the current boreholes (BH 1-21, BH 2-21, and BH 3-21) at depths ranging from 1.8 m in BH 1 to 2.1 m in BH 2. The recovery values and RQD values for the bedrock cores were calculated. The recovery values varied between 42 and 100%, while the RQD values ranged between 35 and 94%. The quality of bedrock is therefore considered to range from very poor to excellent.

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale from the Carlsbad formation, with an overburden drift thickness of 2 to 3 m depth.

4.3 Groundwater

Groundwater levels were measured during the current investigation on March 22, 2021 within the installed wells. In addition, groundwater monitoring wells were installed in the previous boreholes. The groundwater level measurements are presented in Table 1 below.

<table>
<tr><th colspan="5">Table 1 – Summary of Groundwater Levels</th></tr>
<tr><th colspan="5">PG5707 – Current Investigation</th></tr>
<tr><th rowspan="2">Test Hole Number</th><th rowspan="2">Ground Surface Elevation (m)</th><th colspan="2">Measured Groundwater Level / Groundwater Infiltration for Test Pits</th><th>Dated Recorded</th></tr>
<tr><th>Depth (m)</th><th>Elevation (m)</th><th></th></tr>
<tr><td>BH 1-21</td><td>69.08</td><td>-</td><td>-</td><td rowspan="3">March 22, 2021</td></tr>
<tr><td>BH 2-21</td><td>69.27</td><td>1.06</td><td>68.21</td></tr>
<tr><td>BH 3-21</td><td>69.45</td><td>1.43</td><td>68.02</td></tr>
<tr><th colspan="5">PG4561 – Previous Investigation</th></tr>
<tr><td>BH 1</td><td>-</td><td>1.81</td><td>-</td><td rowspan="2">January 16, 2014</td></tr>
<tr><td>BH 2</td><td>-</td><td>1.84</td><td>-</td></tr>
<tr><td colspan="5">Note: The ground surface elevation at each borehole location for the current investigation was surveyed using a handheld GPS referenced to a geodetic datum.</td></tr>
</table>

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels 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. It is anticipated that the proposed building will be supported over conventional footings placed over an undisturbed, very stiff to stiff silty clay/silty sand, glacial till or bedrock. Where existing fill is encountered below the footings of the proposed structure, a geotechnical evaluation of the fill should be completed to determine if the underlying fill is suitable to remain in place and a proof-rolling program would improve the fill layer's compactness. A 300 mm thick granular pad should be placed over a proof-rolled fill layer, which will reviewed and approved by Paterson personnel at the time of construction.

Permissible Grade Raise

Due to the presence of a silty clay layer, the subject section of the site is subjected to a permissible grade restriction. Based on our review of the subsoil profile, a permissible grade raise restriction of **2.0 m** above existing ground surface will be assigned where the footings are expected to be founded on the silty clay deposit.

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If excavated brown silty sand, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, it is recommended that the material be placed under dry conditions and in above freezing temperatures. The silty sand fill should be compacted in thin lifts using a suitable compaction equipment for the lift thickness by making several passes and approved by Paterson personnel.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Consideration could be given to placing the proposed footings and floor slab over the existing fill, free of deleterious materials and organics, provided the fill is approved by Paterson at the time of construction. The approved existing fill material should be proof-rolled using suitable compaction equipment under dry conditions, tested and approved by Paterson personnel. Also, a minimum 300 mm thick granular pad, consisting of a Granular A crushed stone, compacted to 98% of its SPMDD is recommended to be placed at footing level over the approved fill subgrade. Where the fill is deemed inadequate below the proposed footings, the fill should be sub-excavated below the design underside of footing and replaced with engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum 98% of the material's SPMDD. The engineered fill should be extended a minimum 300 mm horizontally beyond the footing face in all directions at footing level and throughout the lateral support zone of the footing.

Bedrock Removal

In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe ramming.

5.3 Foundation Design

Bearing Resistance Values

As noted above, based on the subsurface profile encountered in the test holes, it is recommended that the proposed building be founded on conventional spread footings placed on undisturbed, very stiff to stiff silty clay, compact silty sand/glacial till, or clean, surface sounded bedrock. Alternatively, where the existing fill is approved by Paterson at the time of construction, a granular pad placed over the proof-rolled fill, free of deleterious materials, which has been approved by Paterson personnel.

Overburden Bearing Surface

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff to stiff silty clay bearing surface or approved engineered fill placed directly over the undisturbed very stiff to stiff silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

Footings placed on an undisturbed, compact silty sand, compact to dense glacial till, or approved engineered fill placed directly over the undisturbed, compact silty sand or compact to dense glacial till, 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**, incorporating a geotechnical resistance factor of 0.5.

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

For areas where existing fill, free of significant amounts of deleterious materials and organics, is encountered at design underside of footing level, the following ground improvement program is recommended to provide a suitable founding bearing surface. The existing fill should be sub-excavated to 300 mm below design underside of footing elevation. A minimum 300 mm thick layer of Granular A or Granular B Type II, should be placed in maximum 225 mm loose lifts and compacted to 98% of its SPMDD under dry and above freezing temperatures. The exposed fill surface should be proof-rolled by a vibratory roller making several passes and approved by the geotechnical consultant before placement of the granular pad. Any poor performing areas noted during the proof-rolling operation should be either removed and replaced with an approved engineered fill.

Bedrock Bearing Surface

Footings supported on clean, surface-sounded bedrock can be designed using a bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**, incorporating a geotechnical resistance factor of 0.5. A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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.

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 very stiff to stiff silty clay, compact silty sand/glacial till, and/or engineering fill, above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes 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:3V (or shallower).

Settlement

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

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

Bedrock/Soil Transition

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

5.4 Design for Earthquakes

A previous site-specific shear wave velocity test was completed by Paterson in 2018 to accurately determine the applicable seismic site classification for foundation design of the proposed building as presented in Table 4.1.8.4.A of the Ontario Building Code 2012. Two (2) shear wave velocity profiles (Figures 2 and 3) from the previous on-site testing are presented in Appendix 2.

Field Program

At the time of the previous investigation, the location of the seismic array was chosen to provide adequate coverage of the area. The seismic array testing location is presented in Drawing PG5707-1 - Test Hole Location Plan in Appendix 2.

At the seismic array location, Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was 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 five to ten times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shots were located at 3, 4.5 and 22 m away from the first and last geophones and at the center 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, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

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

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_i(m)}{Vs_i(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{1{,}962m/s}\right)}$$

$$V_{s30} = 1{,}962m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, beneath the foundation is 1,962 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building modifications, 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 Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill, such as those containing significant amounts of organic matter and other deleterious material, within the footprint of the proposed building, the existing fill approved by the geotechnical consultant at the time of construction will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is recommended that the existing fill layer, free of deleterious and organic materials, be proof-rolled several times and approved by the geotechnical consultant at the time of construction. Any soft areas should be removed and backfilled with appropriate granular material. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Pavement Design

Permeable Pavement Structures

It is understood that the installation of permeable pavers is being considered for the car-only parking areas located in close proximity to the proposed building and landscaped areas along Parisien Street and Michael Street.

Based on the developed nature of the subject site, significant grade raises are not anticipated where permeable pavers are to be placed. Therefore, it is anticipated that the permeable pavers will be placed at the approximate existing grade, overlying the existing sandy fill material which is conducive to infiltration. A conservative infiltration rate of 45 to 85 mm/hr can be assumed for the fill material based on the composition of the fill material encountered during the geotechnical investigation.

Groundwater levels were recorded at the borehole locations in March of 2021 during spring conditions. At that time, groundwater levels were observed to range from approximately 1.1 to 1.4 m below the existing ground surface. However, it is anticipated that the long-term groundwater table is located within the bedrock which was encountered at approximate depths of 1.8 to 2.1 m below the existing ground surface.

Based on the groundwater observations at the borehole locations during spring conditions, it is anticipated that a minimum of 1 m of separation from the underside of the permeable pavers will be provided.

It is anticipated that the overall permeable pavement structures will be specified by others specializing in permeable pavement construction. However, should an additional subbase course be required below the recommended permeable paver system, the following is recommended.

Table 2 - Recommended Permeable Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
-	**Permeable Paver Structure (as per manufacturer specifications)**
400	**SUBBASE** – OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Asphalt Pavement Structures

Car only parking areas, driveways and access lanes are anticipated at this site. The proposed pavement structures are shown in Tables 4 and 5.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is considered optional for the proposed building. The system, if implemented, should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

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

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 foundation insulation, should be provided.

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 proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available in selected areas of the excavation to be undertaken by open-cut methods (i.e. 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. 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.

6.4 Pipe Bedding and Backfill

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

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

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

6.5 Groundwater Control

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

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

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

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 very aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following recommendations be completed by the geotechnical consultant.

- Review detailed grading plan(s) from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 11, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Maha Saleh, M.A.Sc. P.Eng.

David J. Gilbert, P.Eng

Report Distribution:

☐ St Laurent Volvo (1 email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building
1300 Michael Street - Ottawa, Ontario

DATUM Geodetic

REMARKS

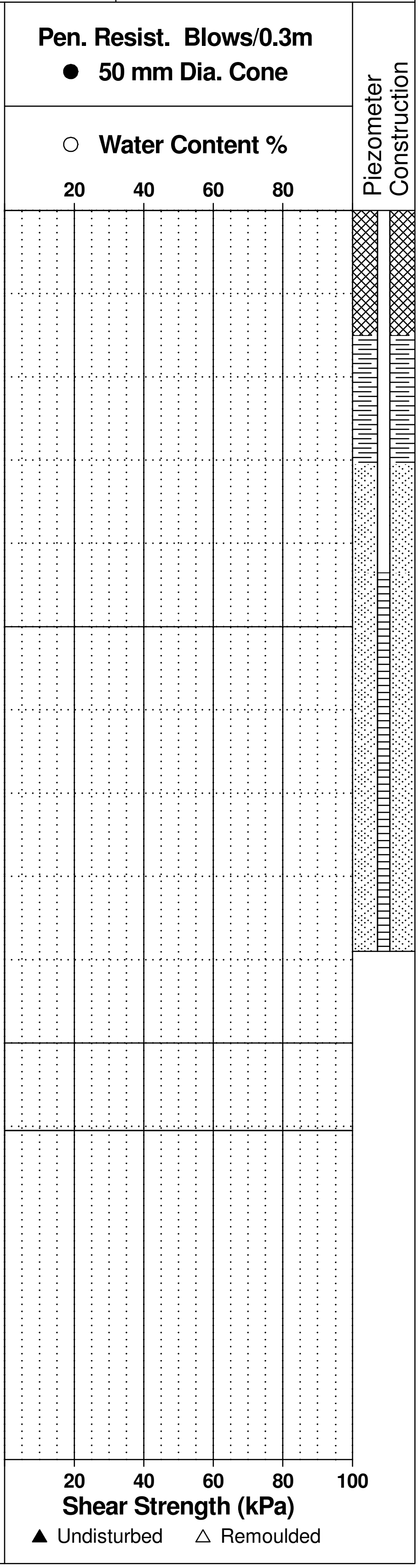
BORINGS BY CME 55 Power Auger

DATE 2021 March 15

FILE NO. **PG5707**

HOLE NO. **BH 1-21**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.08
FILL: Brown silty sand with crushed stone 0.61	AU	1				
FILL: Brown silty sand with topsoil, organics, and gravel 1.45	SS	2	42	8	1	68.08
Very stiff to stiff brown **SILTY CLAY** trace sand 1.78						
BEDROCK: Poor to fair quality black Shale 2.21	SS	3	58	+50	2	67.08
End of Borehole Practical refusal to augering at 2.21 m depth (Piezometer blocked - March 22, 2021)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building
1300 Michael Street - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 15

FILE NO. PG5707

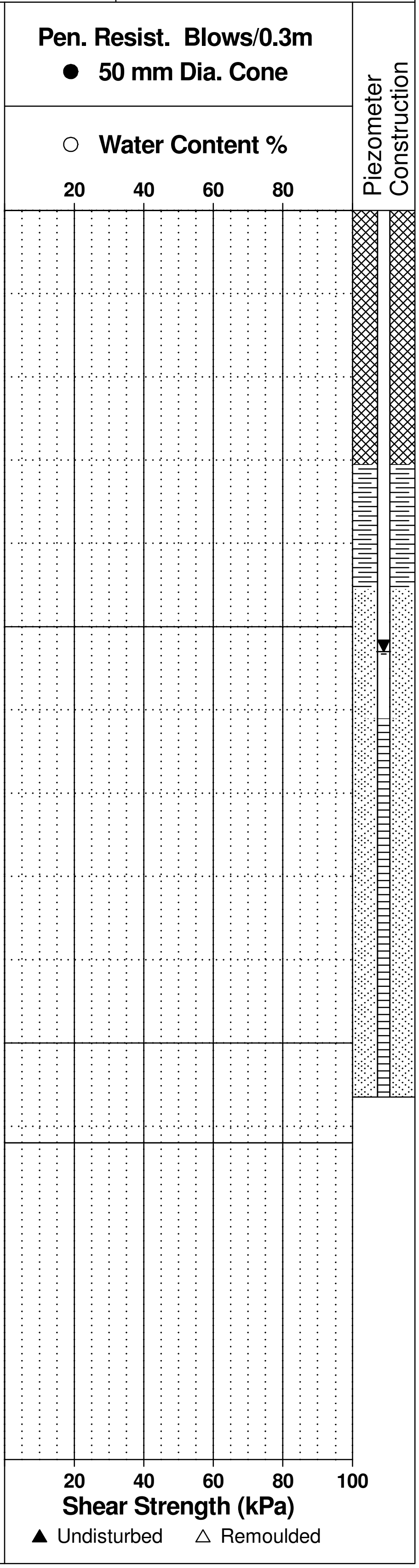
HOLE NO. BH 2-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.27
Asphaltic Concrete (0.08)						
FILL: Crushed stone with brown silty sand (0.56)	AU	1				
FILL: Brown silty sand with topsoil, some clay, trace gravel and wood fragments (1.78)	SS	2	75	16	1	68.27
Very stiff to stiff brown **SILTY CLAY** with sand, trace gravel (1.98)	SS	3	100	16		
GLACIAL TILL: Brown silty sand with fragmented shale, cobbles and boulders (2.13)					2	67.27
BEDROCK: Poor to fair quality black Shale (2.24)						

End of Borehole

Practical refusal to augering at 2.24 m depth

(GWL @ 1.06 m depth - March 22, 2021)



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building
1300 Michael Street - Ottawa, Ontario

DATUM Geodetic
FILE NO. PG5707
REMARKS
HOLE NO. BH 3-21
BORINGS BY CME 55 Power Auger
DATE 2021 March 15

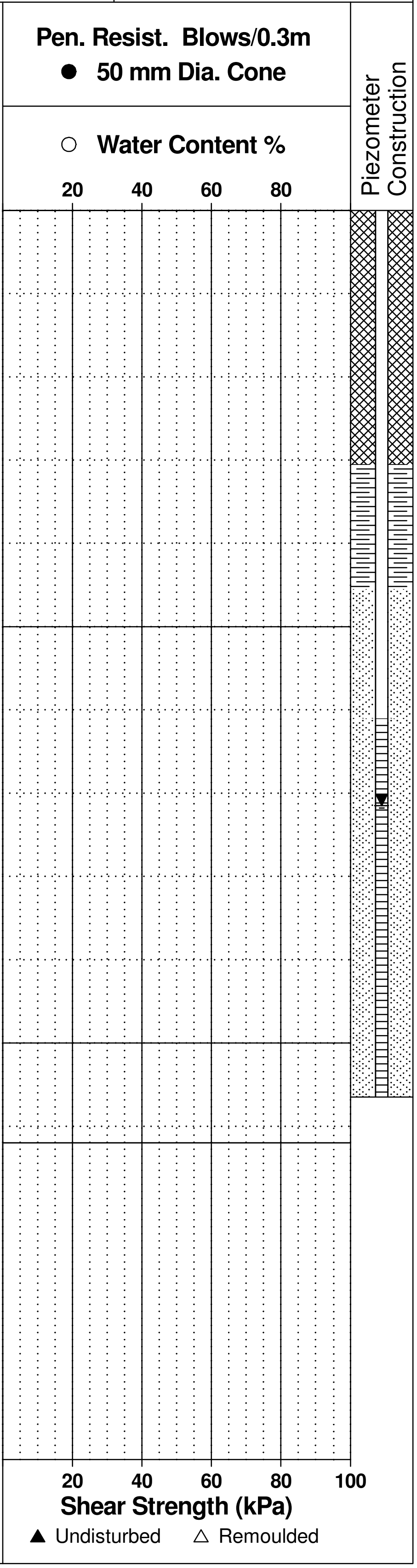
SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic Concrete — 0.10				
FILL: Crushed stone with brown silty sand — 0.76	AU	1		
FILL: Brown silty sand with gravel, topsoil, and cobbles — 1.37	SS	2	50	+50
Very stiff brown **SILTY CLAY** to **SILTY SAND** — 1.96	SS	3	75	+50
BEDROCK: Poor to fair quality quality black Shale — 2.24				

DEPTH (m)	ELEV. (m)
0	69.45
1	68.45
2	67.45

End of Borehole

Practical refusal to augering at 2.24 m depth

(GWL @ 1.43 m depth - March 22, 2021)



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)

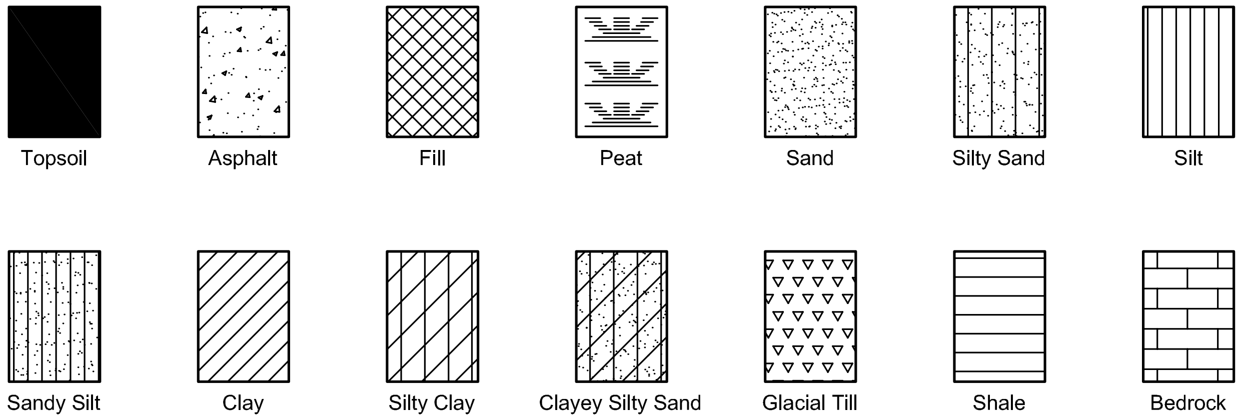
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

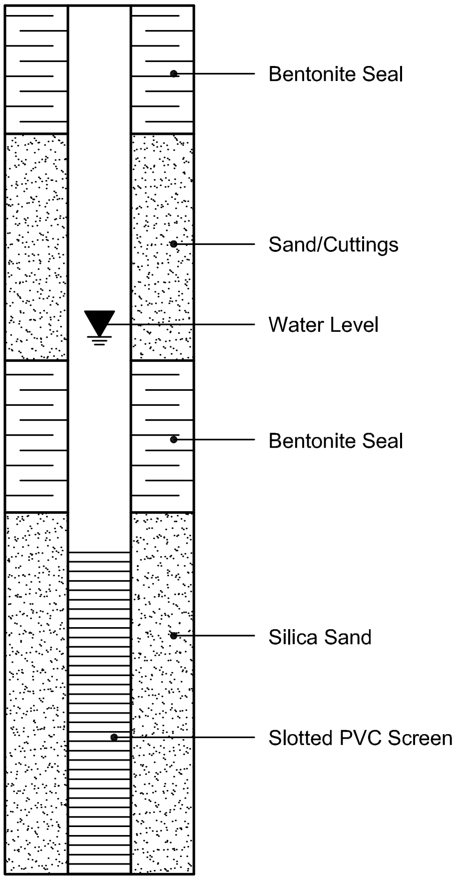
SYMBOLS AND TERMS (continued)

STRATA PLOT

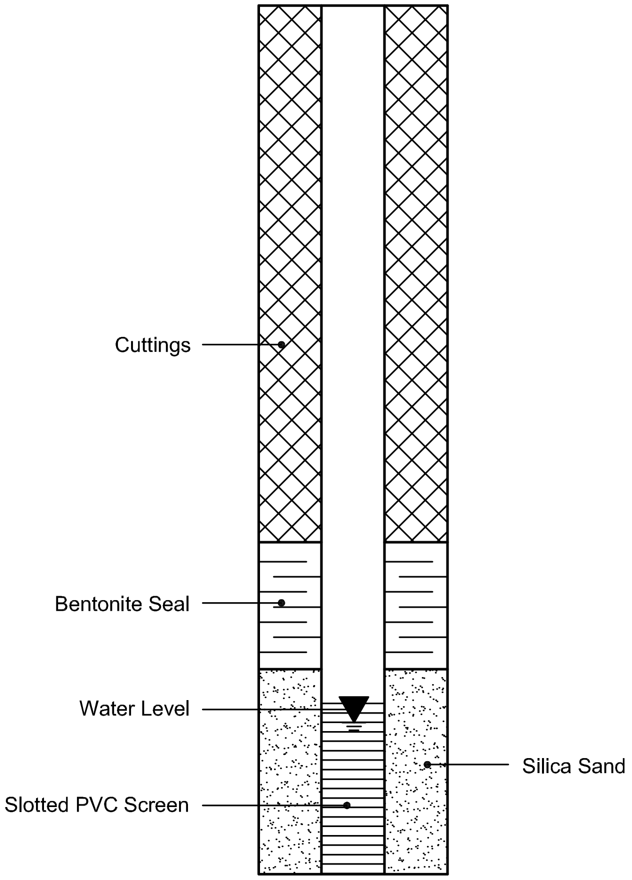


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 32760

Report Date: 18-Mar-2021

Order Date: 15-Mar-2021

Project Description: PG5707

	Client ID:	BH2-SS3A	-	-	-
	Sample Date:	15-Mar-21 09:00	-	-	-
	Sample ID:	2112118-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.05	-	-	-
Resistivity	0.10 Ohm.m	25.1	-	-	-
Anions					
Chloride	5 ug/g dry	109	-	-	-
Sulphate	5 ug/g dry	13	-	-	-

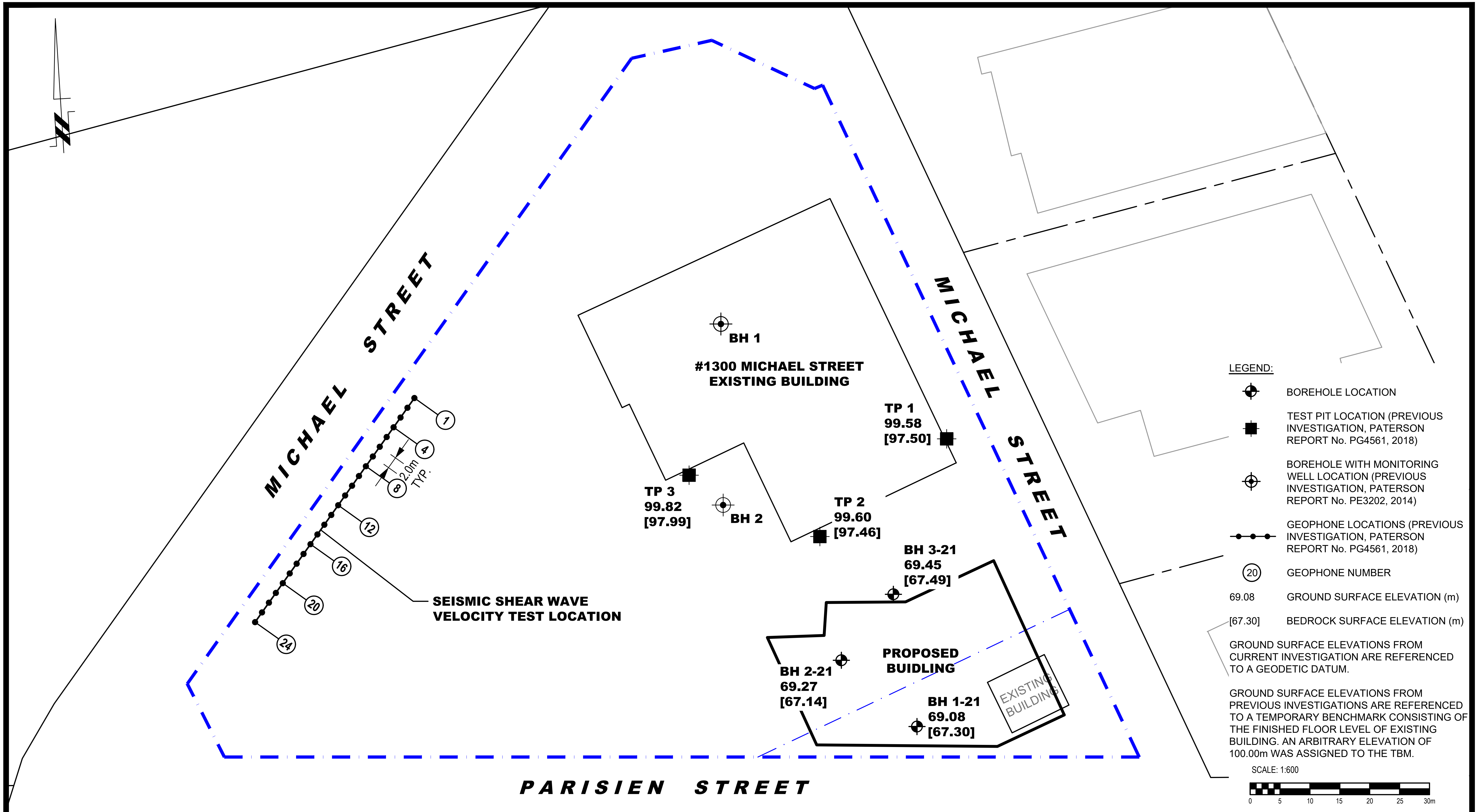
APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5707-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
■	TEST PIT LOCATION (PREVIOUS INVESTIGATION, PATERSON REPORT No. PG4561, 2018)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PREVIOUS INVESTIGATION, PATERSON REPORT No. PE3202, 2014)
•–•–•	GEOPHONE LOCATIONS (PREVIOUS INVESTIGATION, PATERSON REPORT No. PG4561, 2018)
⑳	GEOPHONE NUMBER
69.08	GROUND SURFACE ELEVATION (m)
[67.30]	BEDROCK SURFACE ELEVATION (m)

GROUND SURFACE ELEVATIONS FROM CURRENT INVESTIGATION ARE REFERENCED TO A GEODETIC DATUM.

GROUND SURFACE ELEVATIONS FROM PREVIOUS INVESTIGATIONS ARE REFERENCED TO A TEMPORARY BENCHMARK CONSISTING OF THE FINISHED FLOOR LEVEL OF EXISTING BUILDING. AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

patersongroup
consulting engineers

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

ST. LAURENT VOLVO
GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING
1300 MICHAEL STREET
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 03/2021
Drawn by: NFRV	Report No.: PG5707-1
Checked by: MS	Dwg. No.: PG5707-1
Approved by: DJG	Revision No.:

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