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Geotechnical Engineering

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Noise and Vibration Studies

Geotechnical Investigation

Proposed Multi-Storey Building
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

Prepared For

SerCo Realty Group

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Report: PG5952-1

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Table of Contents

PAGE

1.0 Introduction 1

2.0 Proposed Development.. 1

3.0 Method of Investigation 2

3.1 Field Investigation 2
3.2 Field Survey 4
3.4 Analytical Testing 4

4.0 Observations 5

4.1 Surface Conditions 5
4.2 Subsurface Profile 5
4.3 Groundwater 6

5.0 Discussion 7

5.1 Geotechnical Assessment 7
5.2 Site Grading and Preparation 7
5.3 Foundation Design 9
5.4 Design for Earthquakes 10
5.5 Basement Slab 12
5.6 Basement Wall 12
5.7 Rock Anchor Design 14
5.8 Pavement Design 16

6.0 Design and Construction Precautions 19

6.1 Foundation Drainage and Backfill 19
6.2 Protection of Footings Against Frost Action 20
6.3 Excavation Side Slopes 20
6.4 Pipe Bedding and Backfill 22
6.5 Groundwater Control 23
6.6 Winter Construction 24
6.7 Corrosion Potential and Sulphate 25

7.0 Recommendations 26

8.0 Statement of Limitations 27

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Borehole Logs by Others
Analytical Test Results

Appendix 2 Figure 1 - Key Plan
Figures 2 & 3 – Seismic Shear Wave Velocity Profiles
Drawing PG5952-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by SerCo Realty Group to conduct a geotechnical investigation for the proposed multi-storey building to be located at 337 & 345 Montgomery Street and 94 Selkirk Street in the City of Ottawa (refer to Figure 1 - Key Plan 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 boreholes.

- Provide geotechnical recommendations pertaining to 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

Based on the available drawings, the proposed development will consist of a multi-storey building with at least two underground levels. Associated access lanes, walkways, and landscaped areas are anticipated at finished grades surrounding the proposed building. It is further anticipated that the proposed building will be municipally serviced.

Construction of the proposed development will involve demolition of the existing buildings presently located at the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on September 9 and 10, 2021 and consisted of advancing a total of 6 boreholes to a maximum depth of 9.1 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5952-1 - Test Hole Location Plan included in Appendix 2.

A previous field investigation was carried out on April 4 and 5, 2019 by others. At that time, 3 boreholes were drilled to a maximum depth of 9.1 m below existing ground surface. The borehole logs from that investigation are also shown on Drawing PG5952-1 – Test Hole Location Plan, included in Appendix 2.

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

Sampling and In Situ Testing

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

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

Rock samples were recovered from boreholes BH 2-21 and BH 5-21 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes, and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

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

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

Two boreholes were each fitted with a groundwater monitoring well to allow groundwater level monitoring. The observed groundwater levels were recorded in the field. Ground observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

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

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

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.

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 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 test hole locations from the previous investigation by others are understood to be estimated and assumed to be approximate. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5952-1 - 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. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures, which was collected from borehole BH 4-21. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site has an approximately triangular shape, consisting of 3 contiguous properties, and which is occupied by a total of 3 residential dwellings along with driveways and landscaped areas. The site is bordered by Selkirk Street to the north, residential properties to the east, and Montgomery Street to the south and west. The existing ground surface across the site is relatively level at approximate geodetic elevation 57 to 58 m.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of a 20 to 80 mm thick asphalt pavement structure at the ground surface level, underlain by fill material followed by a glacial till deposit.

The fill was generally observed to consist of brown silty sand with trace amounts of clay, crushed stone, and gravel. Glass fragments were also observed in the fill material in borehole BH 6-21, and concrete and bricks were observed in borehole BH 1-21. The fill was generally observed to extend up to approximate depths ranging from 0.2 to 1.3 m below the existing ground surface.

A compact to very dense glacial till deposit was encountered underlying the fill material, which generally consists of brown silty sand with gravel cobbles and boulders.

Practical refusal to augering was encountered at approximate depths ranging from 3 to 5.1 m below the existing ground surface.

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

Bedrock

The bedrock was cored in boreholes BH 2-21 and BH 5-21 beginning at approximate depths of 4.6 and 3.5 m, respectively. Based on the recovered rock core, the bedrock consists of interbedded limestone and shale, varying from poor to excellent in quality, generally increasing in quality with depth. The bedrock was cored to a maximum depth of 9.1 m.

4.3 Groundwater

Groundwater levels were recorded in the monitoring wells installed at Paterson's borehole locations on September 16, 2021, and at the monitoring wells installed by others in April 2019. The groundwater level readings noted at those times are presented in Table 1 below.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH2-21	57.58	6.20	51.38	September 16, 2021
BH5-21	57.86	6.72	51.14	
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.				
Groundwater Readings by Others				
BH 1	57.64	3.60	54.04	April 16, 2021
BH 2	57.95	3.20	54.75	
BH 3	57.82	3.20	54.62	
Note: The ground surface elevation at each borehole location are shown on the borehole logs by others and are considered approximate.				

Based on field observations of the recovered soil samples, the long-term groundwater table is anticipated at an approximate depth of 3 to 6 m. However, it should be noted that the groundwater levels are subject to seasonal fluctuations and 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 development. The proposed building is recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Based on the anticipated excavation depth and the proximity of the proposed development to the site boundaries, it is anticipated that a temporary excavation support system will be required to support the overburden during the construction period.

Furthermore, bedrock removal will be required to complete the underground levels. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Due to the anticipated founding level for the proposed building, it is expected that all existing overburden material will be excavated from within the proposed building footprint.

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

Bedrock Removal

As noted above, where bedrock removal is required, line drilling and controlled blasting may be required where large quantities of bedrock need to be removed.

Where the bedrock is weathered and/or where small quantities of bedrock need to be removed, hoe-ramming may be sufficient.

The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

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

Vibration Considerations

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

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

Two parameters determine the recommended vibration limit: the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for 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 to minimize the risks of claims during or following the construction of the proposed building.

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 and beneath exterior parking areas 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 these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

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

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless a composite drainage blanket connected to a perimeter drainage system is provided.

Excess Soils

Excess soils generated by construction activities that will be transported off-site should be handled as per *Ontario Regulation 406/19: On-Site Excess Soil Management*.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at ultimate limits states (ULS) of **2,500 kPa**. A geotechnical factor of 0.5 was applied to the above noted bearing resistance value.

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 sound bedrock bearing media when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as that of the bearing medium. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building 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 in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5952-1 - Test Hole Location Plan, attached to the present letter 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 are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 1, 1.5, and 15 m away from the first geophone, 1, 1.5 and 14 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.

Based on our testing results, the average overburden shear wave velocity is **268 m/s**, while the bedrock shear wave velocity is **2,050 m/s**. It is anticipated that the overburden will be completely removed as part of the proposed building construction and footings will be placed directly on the bedrock surface.

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

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}050\ m/s}\right)}$$

$$V_{s30=}\ 2{,}050\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building founded on bedrock is **2,050 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012.

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

5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, the bedrock medium, approved by the Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The recommended pavement structures noted in Section 5.7 will be applicable for the founding level of the proposed building if underground parking garage is considered. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill should 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 at least 98% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of the construction to confirm groundwater infiltration levels, if any. This is discussed further in Subsection 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.

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

It is also expected that a portion of the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m^3 (effective unit weight of 15.5 kN/m^3) where this condition occurs. A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures.

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

Lateral Earth Pressures

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

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Rock Anchor Design

Overview of Anchor Features

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

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond breaker, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of sound limestone and shale is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 below:

Table 2 – Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength – Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) – Good quality Limestone Hoek and Brown Parameters	65 m=0.575 and s=0.00293
Unconfined Compressive Strength – Limestone or Shale bedrock	40 MPa
Unit weight – Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	Mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3 below. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below. The flexible pavement structure presented in Tables 5 and 6 should be used for exterior, at-grade parking and access lanes, respectively.

Table 4 - 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 Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II or material placed over bedrock.	

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

For design purposes, the pavement structure presented in the following tables could be used for the design of exterior car parking areas and access lanes.

Table 5 – Recommended Pavement Structure – Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock, or concrete fill.	

Table 6 – Recommended Pavement Structure – Access Lanes	
Thickness (mm)	**Material Description**
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock, or concrete fill.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment, nothing that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

For the proposed underground parking levels, it is anticipated that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a composite drainage board, such as Delta Drain 6000 or approved equivalent, which is fastened to the bedrock face or temporary shoring system.

For the lower portion of the foundation walls, the following is recommended:

- Line drill the excavation perimeter.
- Hoe ram any irregularities and prepare the bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspections by Paterson.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing interface to allow for the infiltration of water to flow to an interior perimeter sub-slab drainage pipe. The perimeter drainage pipe should direct water to the sump pit within the lower basement area.

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

Sub-Slab Drainage

Sub-slab drainage will be required to control water infiltration under the lowest level floor slab. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, 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 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior columns, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent thickness 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 in these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by 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. The subsurface soil 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.

Bedrock Stabilization

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors and chainlink fencing are anticipated to be required over the upper 2 to 3 m of the vertical bedrock face, which is generally of lower quality, to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

However, the specific requirements for bedrock stabilization measures should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. It is the responsibility of the shoring contractor to ensure that the temporary shoring system is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

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

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

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

Table 7 – Soils Parameter for Shoring System Design

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

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

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

Generally, it is expected that the shoring system will be provided with tie-backs rock anchors to ensure their stability. However, tie backs may encounter highly fractured shale bedrock for the upper several meters. As a result, extra cementitious grout may be required for the tie backs due to open fractures.

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

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD. The bedding should extend at least to the spring line of the pipe.

The cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 99% of the material's standard Proctor maximum dry density (SPMDD).

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

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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

Permit to Take Water

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

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

Impacts on Neighboring Properties

Due to the relatively shallow bedrock present at this site and in the general area, no adverse effects to the neighbouring properties are expected as a result of dewatering which may be required for the subject site.

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.

Precautions must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.

Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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.

7.0 Recommendations

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

- ❑ Review of the geotechnical aspects of the excavating program, prior to construction.
- ❑ Review of waterproofing details for elevator shafts and building sump pits.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Periodic observation of the condition of the vertical bedrock face during excavation.
- ❑ 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 herein 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 SerCo Realty Group 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
Sept. 30, 2021
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Fernanda Carozzi PhD. Geoph.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ SerCo Realty Group (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 10, 2021

FILE NO. PG5952

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	57.42		
Asphaltic concrete 0.05		AU	1						
FILL: Dark brown silty sand with crushed stone and gravel, trace clay, concrete and brick 1.07		AU	2						
		SS	3	71	19	1	56.42		
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders 2.95		SS	4	78	50+				
						2	55.42		
		SS	5	40	50+				
End of Borehole Practical refusal to augering at 2.95m depth									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5952

REMARKS

HOLE NO. BH 2-21

BORINGS BY CME-55 Low Clearance Drill **DATE** September 9, 2021

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	4.60	AU	1		
		SS	2		
		SS	3		76
		SS	4		50+
		RC	1	28	
BEDROCK: Fair to excellent quality, grey interbedded limestone and shale	9.04	RC	2	50	50
		RC	3	98	82
		RC	4	100	90
End of Borehole					
(GWL @ 6.20m - Sept. 16, 2021)					

DEPTH (m)	ELEV. (m)
0	57.58
1	56.58
2	55.58
3	54.58
4	53.58
5	52.58
6	51.58
7	50.58
8	49.58
9	48.58

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 10, 2021

FILE NO. PG5952

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	57.75
Asphaltic concrete	0.02	AU	1				
FILL: Dark brown silty sand with crushed stone and gravel	0.20	AU	2				
						1	56.75
GLACIAL TILL: Compact to very dense, brown silty sand to sandy silt with gravel, cobbles and boulders		SS	3	8	14		
		SS	4	82	50+		
						2	55.75
		SS	5	67	50+		
	3.05					3	54.75
End of Borehole							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5952

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill

DATE September 10, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	57.83
Asphaltic concrete (0.02)	AU	1				
FILL: Dark brown silty sand with crushed stone and gravel, trace clay	AU	2				
(1.07)	SS	3	62	20	1	56.83
GLACIAL TILL: Compact to very dense, brown silty sand to sandy silt with gravel, cobbles and boulders	SS	4	0	50+		
					2	55.83
	SS	5	44	50+		
					3	54.83
(3.25)	SS	6	40	50+		
End of Borehole						
Practical refusal to augering at 3.25m depth						

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5952

REMARKS

HOLE NO. BH 5-21

BORINGS BY CME-55 Low Clearance Drill **DATE** September 9, 2021

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	57.86
Asphaltic concrete	0.08						
FILL: Brown silty sand with crushed stone and gravel	0.60	AU	1				
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders		SS	2	33	45	1	56.86
		SS	3	25	50+	2	55.86
		SS	4	42	66		
	3.53	SS	5	43	50+	3	54.86
BEDROCK: Fair quality, grey interbedded limestone and shale		RC	1	78	73	4	53.86
		RC	2	95	54	5	52.86
		RC	3	95	53	6	51.86
						7	50.86
	9.12	RC	4	100	75	8	49.86
End of Borehole						9	48.86

(GWL @ 6.72m - Sept. 16, 2021)

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

○ Water Content %
20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
337 & 345 Montgomery Street and 94 Selkirk Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5952

REMARKS

HOLE NO. BH 6-21

BORINGS BY CME-55 Low Clearance Drill **DATE** September 10, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	57.69		
Asphaltic concrete — 0.05		AU	1						
FILL: Brown silty sand with crushed stone and gravel — 0.23		AU	2						
FILL: Brown silty sand with clay, crushed stone, gravel and glass fragments — 0.91		SS	3	52	55	1	56.69		
GLACIAL TILL: Very dense, dark brown silty sand to sandy silt with gravel, cobbles and boulders, trace clay		SS	4	67	71	2	55.69		
		SS	5	62	50+	3	54.69		
— 3.20		SS	6	29	50+				
End of Borehole Practical refusal to augering at 3.20m depth									

Shear Strength (kPa): 20 40 60 80 100

▲ 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

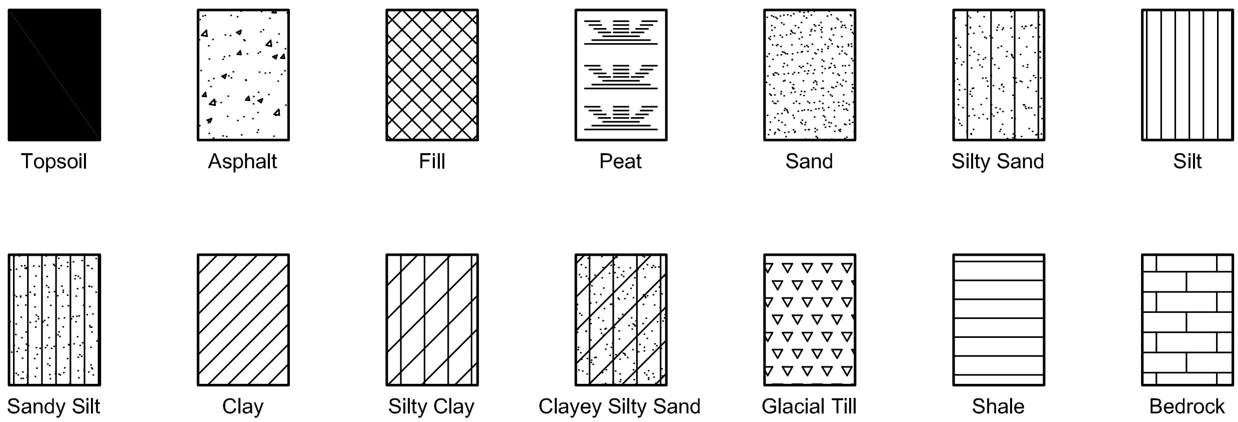
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

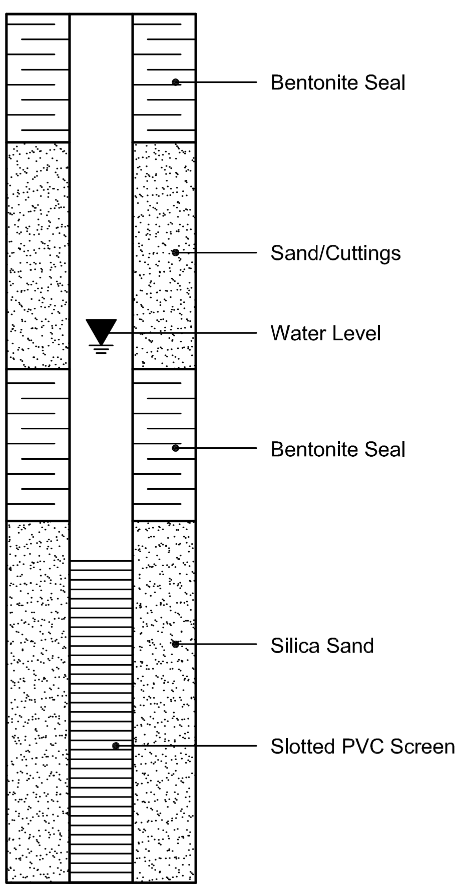
SYMBOLS AND TERMS (continued)

STRATA PLOT

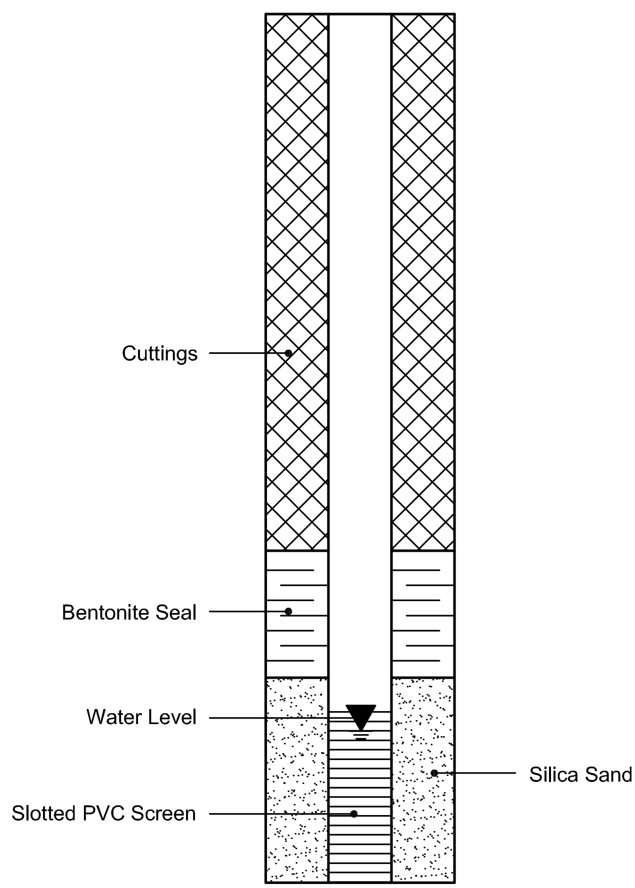


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole BH1

Project No: OTT-00252128-A0

Project: Apartment Building

Location: 337 Montgomery Street, Ottawa, Ontario

Date Drilled: 'April 4, 2019

Drill Type: CME45 Track Mounted Drill Rig

Datum: Approximate Elevation

Logged by: MAD   Checked by: SMP

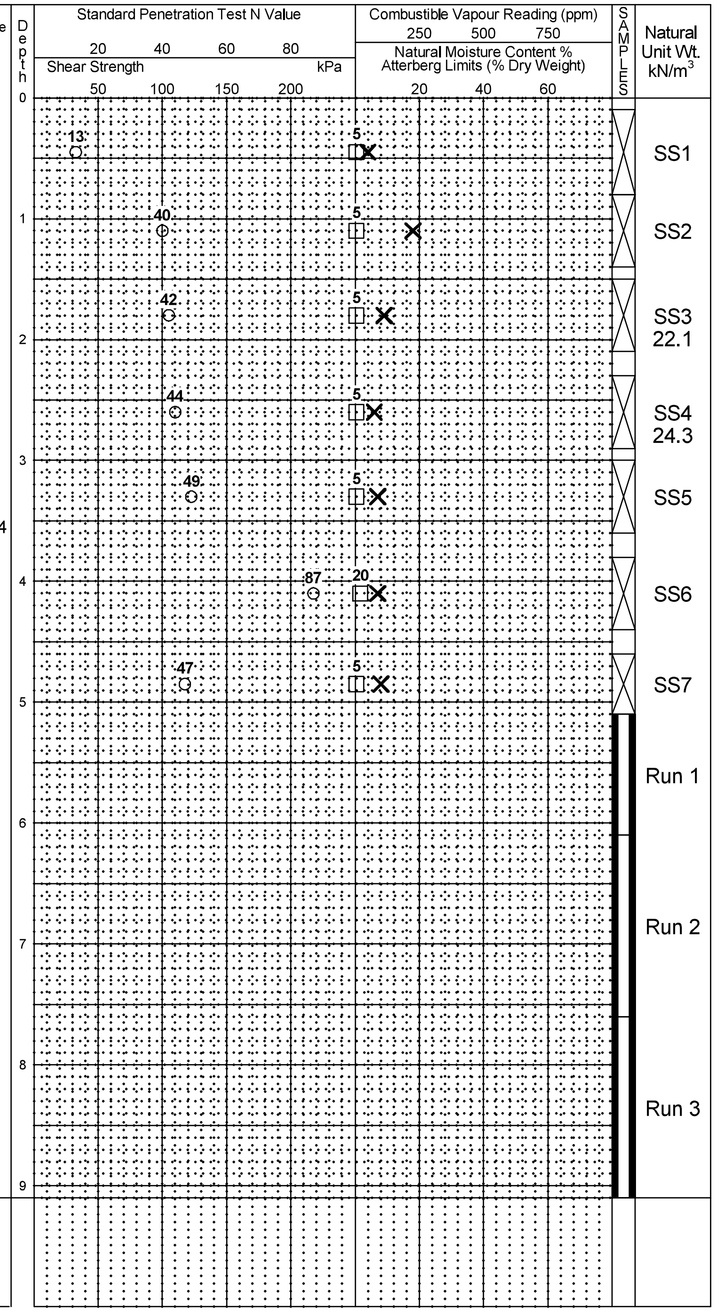
Figure No. 3

Page. 1 of 1

Split Spoon Sample
Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Shear Strength by Vane Test

Combustible Vapour Reading
Natural Moisture Content
Atterberg Limits
Undrained Triaxial at % Strain at Failure
Shear Strength by Penetrometer Test

SOIL DESCRIPTION	Approximate Elevation m
	57.64
ASPHALTIC CONCRETE ~ 12 mm	57.6
GRANULAR FILL (BASE) ~ 250 mm Crushed gravel with silt and sand, grey, damp, no odours, no stains	57.3
FILL Silty sand with gravel, reddish brown mottling, brown, damp, no odours, no stains, (compact)	56.8
GLACIAL TILL Silty sand with gravel, shale fragments, cobbles, boulders, dark brown to black, moist, no odours, no stains, (dense to very dense)	
	54.04
SHALE BEDROCK Rubble-filled rock seams, mud seams, near vertical fractures, dark grey to black, (very poor to good quality) Weathered from 5.1m to 5.3 m depths	52.5
	48.5
Borehole Terminated at 9.1 m Depth	



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in the borehole as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252128-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
12 days	3.6	-

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	5.1 - 6.1	100	19
2	6.1 - 7.6	100	78
3	7.6 - 9.1	100	78

Log of Borehole BH2

Project No: OTT-00252128-A0

Project: Apartment Building

Location: 337 Montgomery Street, Ottawa, Ontario

Date Drilled: 'April 4 and 5, 2019

Drill Type: CME45 Track Mounted Drill Rig

Datum: Approximate Elevation

Logged by: MAD   Checked by: SMP

Figure No. 4

Page. 1 of 1

Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢⊖
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Depth (m)	Approximate Elevation (m)	SOIL DESCRIPTION	SPT N Value	Combustible Vapour Reading (ppm)	Natural Moisture Content %	Samples	Natural Unit Wt. kN/m³
0	57.95						
	57.8	**ASPHALTIC CONCRETE** ~ 150 mm					
	57.7	**GRANULAR FILL (BASE)** ~ 125 mm Crushed gravel with silt and sand, grey, damp, no odours, no stains	17	10	✕	AS1/SS2	
1	56.6	**FILL** Silty sand with gravel, roots, reddish brown mottling, brown, moist, no odours, no stains, (compact)	12	5	✕	SS3	
2		**GLACIAL TILL** Silty sand with gravel, cobbles, boulders, dark brown to black, moist, no odours, no stains, (dense)	50	25	✕	SS4	22.5
			50/125 mm	15	✕	SS5	
3	54.75		70/200 mm	10	✕	SS6	
		DIfficult augering (augers grinding) from 3.2 m to 3.6 m depths					
4		Cobbles and boulders from 3.6 m to 5.2 m depths				Run 1	
5	52.8	**SHALE BEDROCK** Weathered, dark grey to black, (very poor quality)				Run 2	
6	51.8						

Borehole Terminated at 6.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in the borehole as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252128-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
12 days	3.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.6 - 3.9		
2	3.9 - 6.2	43	5

Log of Borehole BH3

Project No: OTT-00252128-A0

Project: Apartment Building

Location: 337 Montgomery Street, Ottawa, Ontario

Figure No. 5

Page. 1 of 1

Date Drilled: 'April 5, 2019

Drill Type: CME45 Track Mounted Drill Rig

Datum: Approximate Elevation

Logged by: MAD Checked by: SMP

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢⊖
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

Approximate Elevation (m)	Depth (m)	SOIL DESCRIPTION	SPT N Value	Combustible Vapour Reading (ppm)	Samples
57.82	0				
57.7		**TOPSOIL** ~ 150 mm			
		FILL Silty sand with gravel, roots, brown to dark brown, moist, no odours, no stains, (loose)	7	10	AS1/SS1
57.1		**GLACIAL TILL** Silty sand with gravel, cobbles, boulders, dark brown to black, moist, no odours, (loose to compact)	7	15	SS2
			24	15	SS3
		Difficult augering (augers grinding) from 2.4 m to 3.1 m depths	59/280mm	10	SS4
54.62		Cobbles and boulders from 3.1 m to 6.0 m depths			Run 1
					Run 2
					Run 3
51.8					Run 4
51.6		**SHALE BEDROCK** Weathered, dark grey to black			

Borehole Terminated at 6.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in the borehole as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252128-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
11 days	3.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.1 - 3.8		
2	3.8 - 4.6		
3	4.6 - 5.4		
4	5.4 - 6.2		

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 33141

Report Date: 20-Sep-2021

Order Date: 14-Sep-2021

Project Description: PG5952

	Client ID:	BH4-21 SS5	-	-	-
	Sample Date:	10-Sep-21 12:00	-	-	-
	Sample ID:	2138240-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	95.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.87	-	-	-
Resistivity	0.10 Ohm.m	36.3	-	-	-
Anions					
Chloride	5 ug/g dry	57	-	-	-
Sulphate	5 ug/g dry	45	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5952-1 – TEST HOLE LOCATION PLAN

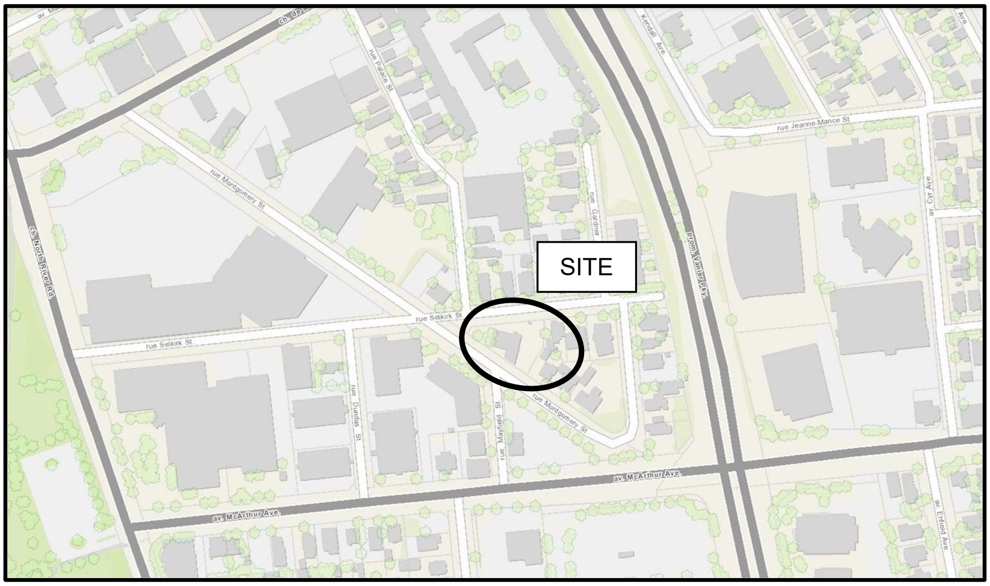


FIGURE 1

KEY PLAN

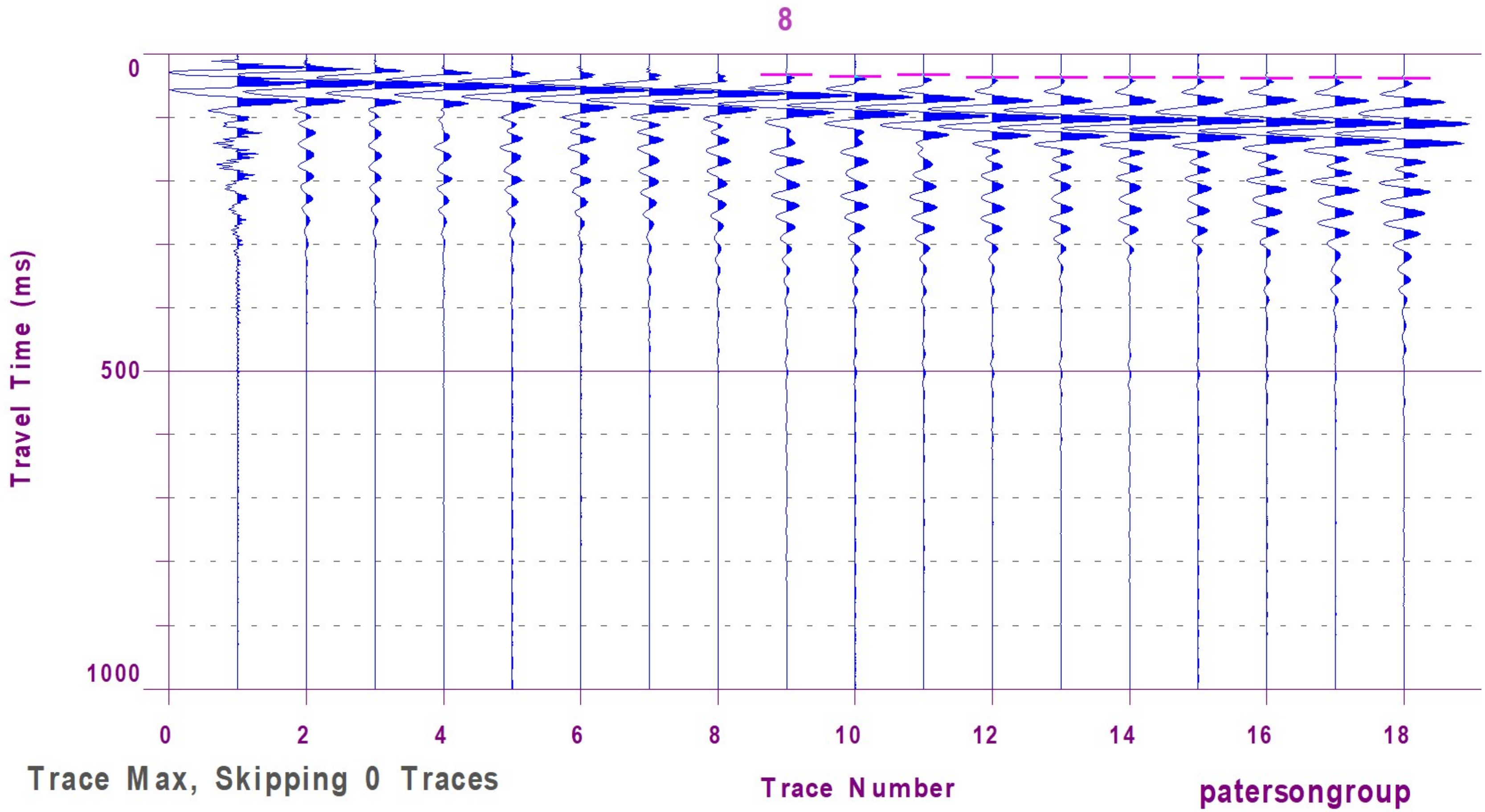


Figure 2 – Shear Wave Velocity Profile at Shot Location -1 m

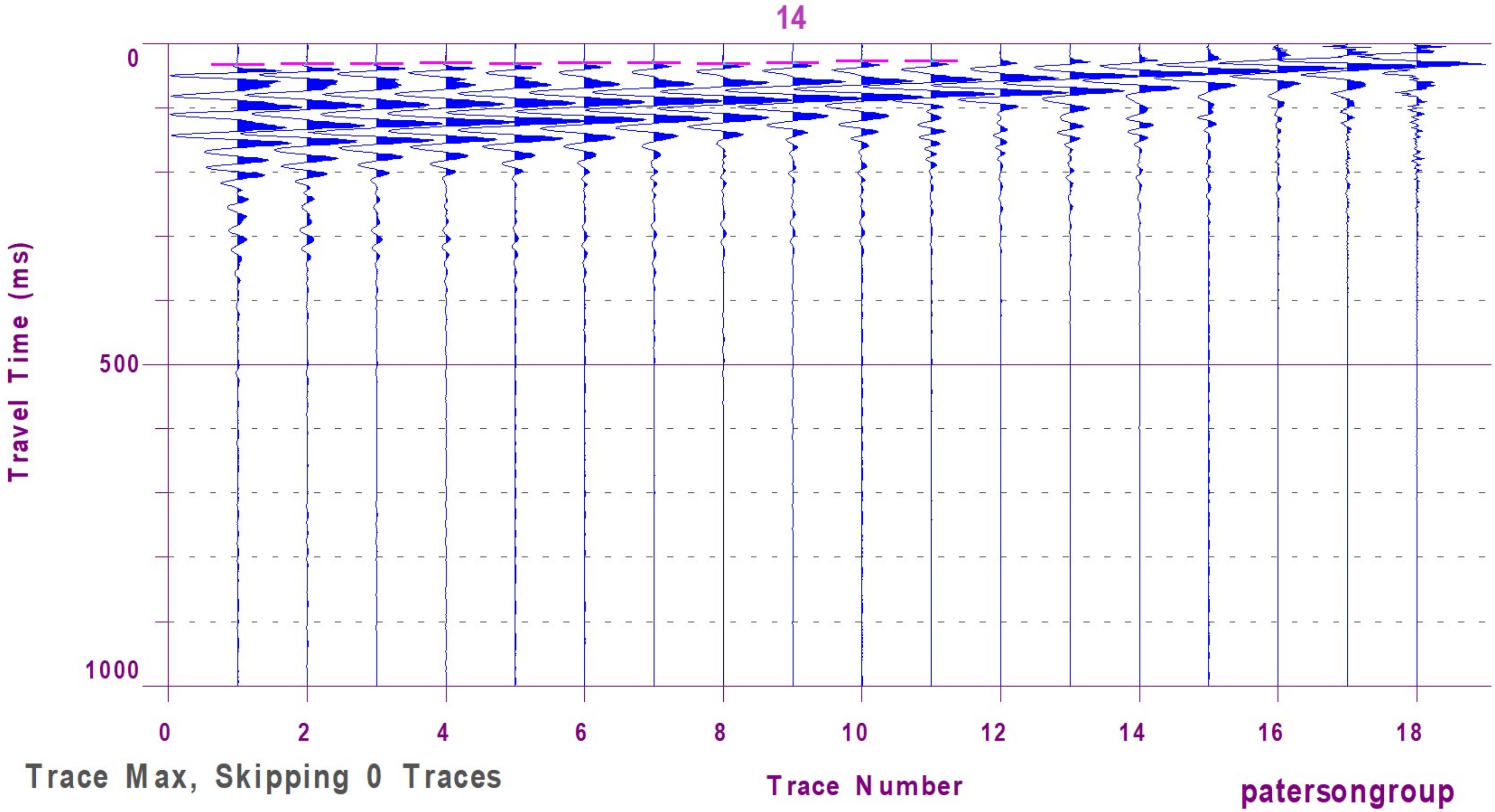
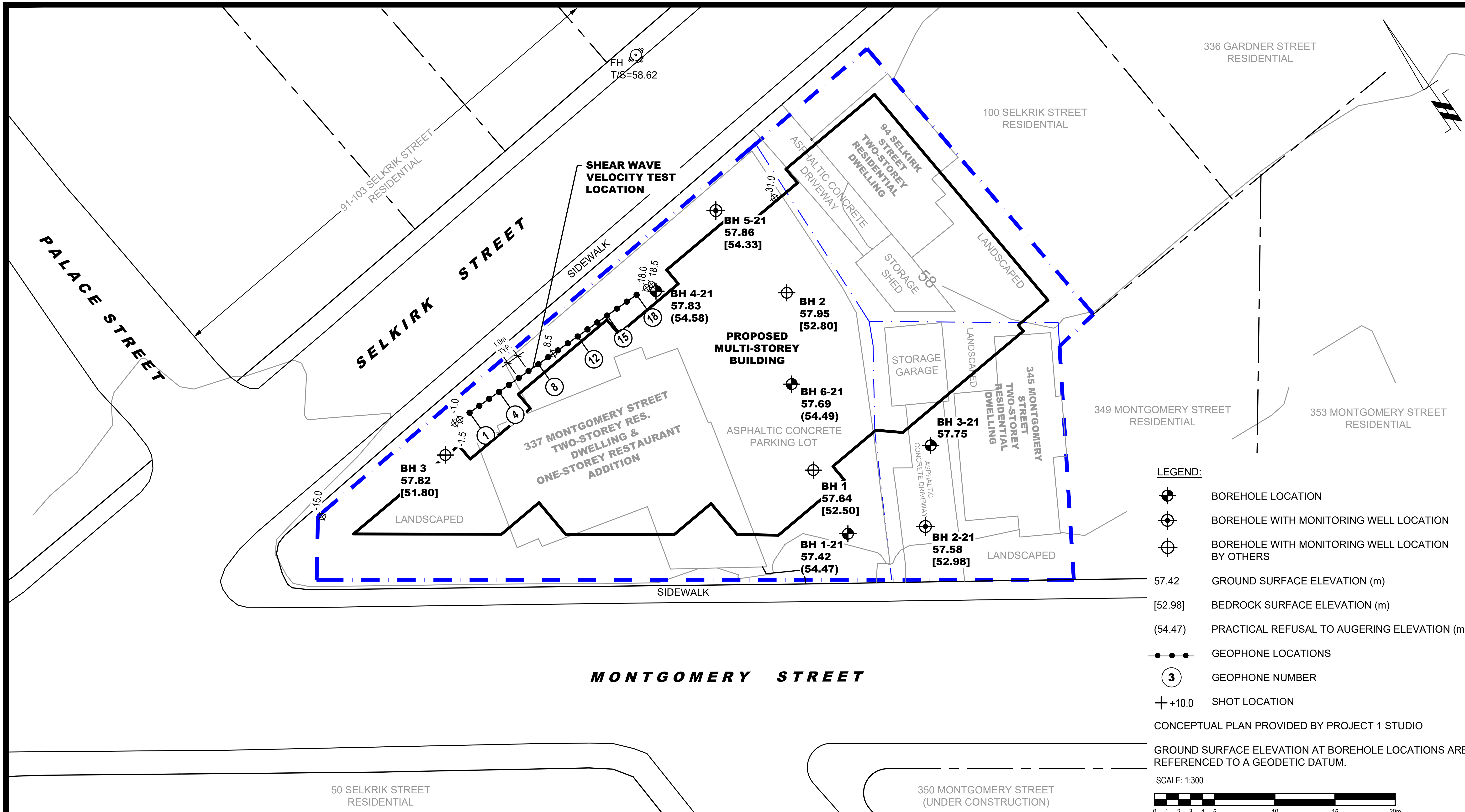


Figure 3 – Shear Wave Velocity Profile at Shot Location 1.5 m



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NO.	REVISIONS	DATE	INITIAL

SERCO REALTY GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
OTTAWA, 337 AND 345 MONTGOMERY STREET AND 94 SELKIRK STREET ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 09/2021
Drawn by: YA	Report No.: PG5952-1
Checked by: FC	Dwg. No.: PG5952-1
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg59xx\pg5952\pg5952-1-test hole location plan.dwg