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Geotechnical Investigation

Proposed Hi-Rise Complex
3-33 Selkirk Street and 2 Montreal Road
Ottawa, Ontario

Prepared For

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Report PG4915-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Main and Main Developments Inc. to conduct a geotechnical investigation for the subject site located at 3-33 Selkirk Street and 2 Montreal Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

☐ determine the subsurface soil and groundwater conditions based on borehole information.

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Environmental information is provided under a separate cover.

2.0 Proposed Development

Based on the current conceptual drawings, it is our understanding that several multi-storey high-rise buildings will be constructed over an underground parking structure with one or two levels below the existing grade and will occupy the majority of the site.

It is further expected that the proposed hi-rise complex will be municipally serviced with water and sewer.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the preliminary geotechnical investigation was conducted on April 3, 4 and 5, 2019. During that time, a total of 10 boreholes (BH 1 to BH 10) were drilled to a maximum depth of 8.3 m below existing ground surface. The test holes were located in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration of site features and underground utilities. The approximate locations of the test holes are shown in Drawing PG4915-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig and portable drilling equipment operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedure consisted of augering to the required depths at select locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

Standard Penetration Tests (SPT) were 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.

Diamond drilling was carried out at three borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

Groundwater

A 32 or 51 mm diameter PVC groundwater monitoring well was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ Slotted 32 or 51 mm diameter PVC screen at the base of the aforementioned boreholes.
- ☐ 32 or 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.
- ☐ A minimum of 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

The groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were determined by Paterson personnel taking into consideration of site features and underground utilities. The location and ground surface elevation at each test hole location was surveyed by Paterson personnel. The boreholes were surveyed with respect to a geodetic datum using a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located to the east of the subject site in front of 307 Montgomery Street. A geodetic elevation of 57.63 m was assigned to the TBM by Annis O'Sullivan Vollebek Ltd. The test hole locations and ground surface elevation at each test hole location are presented on Drawing PG4915-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a one storey commercial retail building with a slab-on-grade construction surrounded by asphalt covered parking areas and access lanes. The site is bordered by Montreal Road to the north, Montgomery Street to the east and Selkirk Street to the south followed by a mixture of residential, commercial and institutional structures. It should be further noted that the site is bordered to the west by North River Road followed by a municipal park and the Rideau River.

The site was observed to be relatively flat and approximately at grade with adjacent roadways and neighbouring properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an asphaltic pavement structure overlying varying thickness of fill material extending to depths between 1.2 to 5.6 m below existing ground surface. The fill consisted of a mixture of silty sand//sandy silt with gravel, shale fragments, trace clay, topsoil and brick. Glacial till, consisting of sandy silt to silty sand with gravel, trace clay was identified at BH 5, BH 9 and BH 10. A heavily fractured to fractured shale bedrock was encountered at all borehole locations between 2.0 and 5.8 m below existing ground surface within the exception of BH 9 where bedrock was not encountered. However, BH 9 was terminated in a compact to very dense grey sandy silt to silty sand with gravel and shale fragments at a depth of 8.2 m.

Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of a dark brown to black shale with laminations of calcareous silt stone of the Billings Formation and expected to be encountered at depths varying between 3 and 10 m.

4.3 Groundwater

Groundwater levels were measured in monitoring wells on April 12, 2019. The measured groundwater level (GWL) readings are presented in Table 1 below and further presented in the Soil Profile and Test Data sheets in Appendix 1. Long-term groundwater level can also be estimated based on the observed moisture levels, colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected between 6 to 7 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	56.08	6.02	50.06	April 12, 2019
BH 2	56.09	5.56	50.53	April 12, 2019
BH 3	56.47	5.94	50.53	April 12, 2019
BH 4	56.50	5.95	50.55	April 12, 2019
BH 5	56.55	5.98	50.57	April 12, 2019
BH 6	56.69	5.56	51.13	April 12, 2019
BH 7	56.75	6.22	50.53	April 12, 2019
BH 8	56.70	6.16	50.54	April 12, 2019
BH 9	56.66	4.04	52.62	April 12, 2019
BH 10	-	6.43	-	April 12, 2019

Notes: The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located to the east of the subject site in front of 307 Montgomery Street. A geodetic elevation of 57.63 m was assigned to the TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It's expected that the buildings will be founded on conventional spread footing foundations placed on a clean, surface sounded shale bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

For the subject development, it is expected that all the overburden will be removed to accommodate one or two levels of underground parking. Furthermore, all buildings and structures will be demolished and removed. Topsoil and deleterious fill, such as those containing organics or construction debris, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming and/or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

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

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

Rock Stabilization

Rock anchors and/or rock bolts in conjunction with steel mesh and/or shotcrete may be required to stabilize the vertical bedrock face during the excavation program. The requirement for rock stabilization will be evaluated during the excavation operations.

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 in the construction operations to maintain a cooperative environment with the residents.

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. Vibrations, 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). These guidelines are for current construction standards.

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

Protection of Potential Expansive Bedrock

It is possible that expansive shale will be encountered at the subject site. Although the effects of expansive shale will not affect the proposed building structure, it is possible that it will affect the proposed basement floor slabs founded close to the shale bedrock.

A potential for heaving and rapid deterioration of the shale bedrock exists at this site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed development footprint should be protected from excessive dewatering and exposure to ambient air.

To accomplish this, a 50 mm thick concrete mud slab should be placed on the exposed bedrock surface within a 48 hour period of being exposed. A 17 MPa sulphate resistant lean concrete is recommended for this purpose. As an alternative to the mud slab, keeping the shale surface covered with granular backfill is also acceptable.

Selected excavated vertical sides of the exposed bedrock can be protected using a sprayed elastomeric coating or shotcrete to seal the bedrock from exposure to air and dewatering.

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 to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted using 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 maximum 300 mm thick lifts and compacted by the tracks of the spreading equipment to minimize voids.

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

Excavated shale deteriorates upon exposure to air and is not generally suitable for re-use as an engineered fill. The use of imported granular fill is recommended.

5.3 Foundation Design

Bearing Resistance Values

Footings placed over a clean, surface sounded shale bedrock surface can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings placed on a sound bedrock surface can be designed using a factored bearing resistance value at ULS of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5, can be used if the bedrock is free of seams, fractures and voids within 1.5 m below the founding level.

It is anticipated that the weathered portion of the bedrock is a surficial layer with a thickness of 250 to 700 mm. Where encountered at the design underside of footing elevation, this layer is anticipated to be sufficiently fractured and weathered to permit removal by a hydraulic shovel excavator and minimal hoe-ramming. Once the heavily weathered bedrock layer is removed to expose the underlying sound bedrock, a bearing capacity of **3,000 kPa** (ULS), as noted above, may be considered for the design of footings. Prior to construction, the following provisions should be considered for attaining a sound bedrock surface by sub-excavating through the weathered bedrock during construction by considering either:

- ❏ in-filling the sub-excavated footing footprints with lean-concrete to the design underside of footing elevation as per the recommendations provided in the proceeding portion of this Subsection, or

- ❏ extending the footing such that it is placed directly on the sound bedrock surface.

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

Settlement

Footings bearing directly or indirectly on bedrock will be subjected to negligible 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 (OBC) 2012. The results of the shear wave velocity testing are attached to the present report.

Field Program

The seismic array location is presented on Drawing PG4915-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 18 horizontal geophones in a straight line in a roughly east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were 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 are located at the centre of the geophone array and 3, 4.5 and 15 m away from the first geophone and last geophones.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. 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 proposed building foundations. 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 shear wave velocity due to the increasing quality of 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 **240 m/s**, while the bedrock shear wave velocity is **2,782 m/s**. Provided the building will be founded partly directly and partly indirectly on the bedrock surface, the overburden shear wave velocity does not need to be considered for the calculation of Vs_{30}.

The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}782m/s}\right)}$$

$$V_{s30} = 2{,}782m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed buildings is **2,782 m/s** provided the footings are placed directly on bedrock surface and/or extended through the overburden soils to an approved bedrock bearing surface by means of near vertical, zero entry concrete in-filled trenches. Therefore, a **Site Class A** is applicable for the proposed buildings, 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

It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear 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. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Basement Wall

It is expected that the basement walls are to be poured against a waterproofing and/or drainage system, which will be placed against the shoring face and exposed bedrock face, where encountered. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). 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. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g 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.

Design Parameters

According to the latest version of the Canadian Foundation Manual, a load resistance factored design (LRFD) should be implemented. As such, the coefficient of friction factor for concrete on bedrock bearing surface can be taken as 0.7. A sliding resistance factor of 0.8 should be utilized as per the Canadian Foundation Manual.

5.7 Rock Anchor Design

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

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

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

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

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

Grout to Rock Bond

The unconfined compressive strength of shale bedrock ranges between 40 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183 and 0.00009**, respectively. For design purposes, all rock anchors were assumed to be placed at least 1.2 m apart to reduce group anchor effects..

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

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) - Fair Quality Shale Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

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	3.0	1.5	4.5	250
	4.2	2.2	6.4	500
	6.5	2.6	9.1	1000
	10	3.5	13.5	2000
125	2.8	1.5	4.3	250
	3.5	2.4	5.9	500
	5.5	2.8	8.3	1000
	8	3.8	11.8	2000

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and flushed clean with water 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. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Structure

The recommended pavement structures for the subject site are shown in Tables 4, 5 and 6.

Table 4 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 5 - 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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 6 - Recommended Rigid Pavement Structure - Lower Parking Level

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage
One Level of Underground Parking

It’s recommended that a perimeter foundation drainage system be provided for the proposed structure if only one level of underground parking is being considered. It’s expected that insufficient room will available for exterior backfill and most likely will be a blind pour against a shoring system. It is suggested that a drainage system would consist of the following:

❑ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the shoring system and bedrock excavation face from the surface to the founding elevation.

❑ It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

❑ Underfloor drainage may be required to control water infiltration below the underground parking level slab. For preliminary design purposes, it’s recommended that a 150 mm diameter perforated pipe be placed in each bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Water Suppression System and Foundation Drainage
Two Levels of Underground parking

To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls and underfloor drainage:

- ❏ The concrete mud slab will create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation. The thickness of the concrete mud slab will be determined during the excavation program when realistic groundwater infiltration can be properly assessed. However for preliminary design purposes, it is recommended that the concrete mud slab be design at a minimum thickness of 150 mm.

- ❏ A waterproofing membrane will be required to lessen the effect of water infiltration for the lower underground parking level starting at 4 m below finished grade. The waterproofing membrane will consist of a bentonite waterproofing such as Tremco Paraseal or equivalent securely fastened to the temporary shoring system or the vertical bedrock surface. The membrane should extend to the bottom of the excavation at the founding level and extend horizontally over the concrete mud slab a minimum of 300 mm prior to the placement of the footings.

- ❏ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as DeltaDrain 6000, MiraDrain G100N or equivalent) extend down to the bottom of the foundation. It's expected that 150 mm diameter sleeves placed at 3 m centres be cast concrete or in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration from the upper 3 m which is above the vertical waterproofed area. The second source will be water breaching the waterproofing membrane.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume of less than 150,000 L/day plus any surface water infiltration following a precipitation event. The initial influx will be greater once the excavation extends below the long term groundwater level. The zone of influence associated with the temporary dewatering during construction excavation for 2 levels of underground parking will be approximately 5 m. Based on the proposed water suppression system, it's expected that long term groundwater infiltration will be significantly reduced during post-construction. With a properly implemented water suppression system, it's expected that post-construction volumes will be less than 10,000 L/day.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier (minimum 150 mm thick concrete mud slab). For design purposes, it's recommended that a 150 mm diameter perforated pipe be placed in each bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

For areas where sufficient space is available for backfill against the exterior sides of the foundation walls, the backfill material should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

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

The underground parking area should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation

Temporary 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 insufficient room will be available for the greater part 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 subsurface soil 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.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring 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 potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 7 - Soil Parameters for Shoring System Design

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

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

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

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.
The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

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 bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) Category 3 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 MOECC.

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

Long-term Groundwater Control

Our preliminary recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building’s perimeter or sub-slab drainage system will be directed to the proposed building’s cistern/sump pit.

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighbouring structures are expected to be founded within the glacial till deposit and/or over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the low permeability of the native soils.

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.

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

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Review of the geotechnical aspects of the excavating contractor's shoring design and bedrock excavation face protection system, prior to construction.

☐ Review the water suppression system design and implementation.

☐ Review proposed foundation drainage design and requirements.

☐ 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.

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

A report confirming the work has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are preliminary in nature and are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 its 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 Main and Main Developments Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
June 2, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

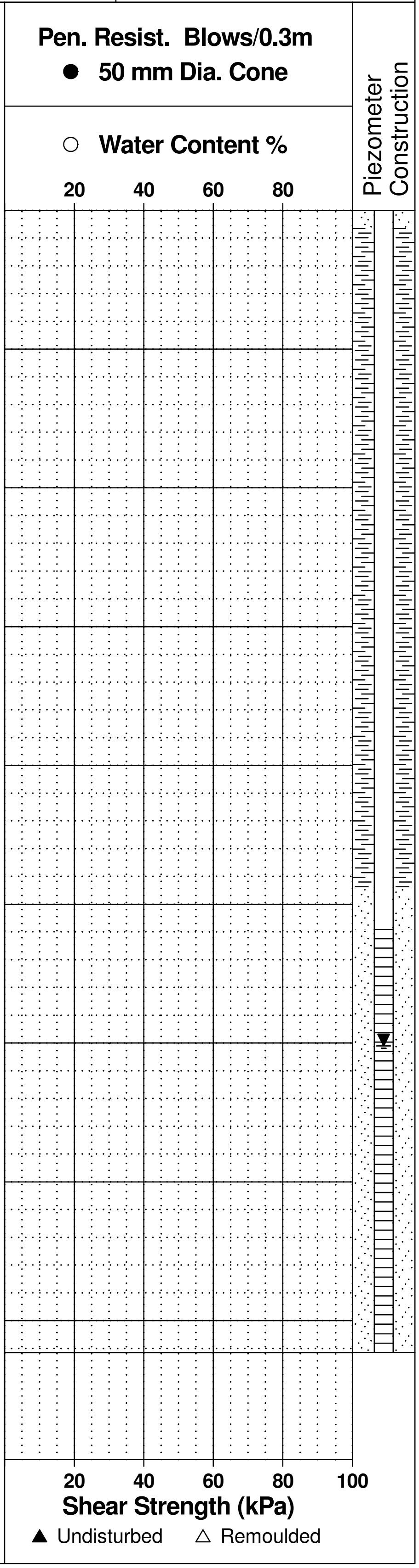
DATE April 3, 2019

FILE NO. PG4915

HOLE NO. BH 1

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Brown silty clay with sand, gravel, trace brick	0.51	AU	1		
FILL: Light brown silty sand with gravel	1.52	SS	2	79	23
FILL: Brown sandy silt to silty sand, trace shale fragments and clay - shale fragments increasing with depth	3.05	SS	3	83	37
		SS	4	88	27
BEDROCK: Heavily fractured to fractured, black shale	8.23	SS	5	83	45
		SS	6	100	50+
		SS	7	88	87
		SS	8	78	50+
		SS	9	71	43
			10	90	50+
		SS	11	75	11
End of Borehole (GWL @ 6.02m - April 12, 2019)					

DEPTH (m)	ELEV. (m)
0	56.08
1	55.08
2	54.08
3	53.08
4	52.08
5	51.08
6	50.08
7	49.08
8	48.08



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger **DATE** April 3, 2019

FILE NO. PG4915

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	56.09
Asphaltic concrete 0.13						
FILL: Brown silty sand with crushed stone 0.60	AU	1				
FILL: Brown silty sand, some clay, gravel and shale fragments	SS	2	88	22	1	55.09
1.98	SS	3	100	50	2	54.09
BEDROCK: Heavily fractured to fractured, black shale	SS	4	88	50+		
	SS	5	80	50+	3	53.09
	SS	6	87	45	4	52.09
	SS	7	71	50+	5	51.09
	SS	8	30	36	6	50.09
	RC	1	100	44		
8.18	RC	2	35	8		
End of Borehole						

(GWL @ 5.56m - April 12, 2019)

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

DATE April 5, 2019

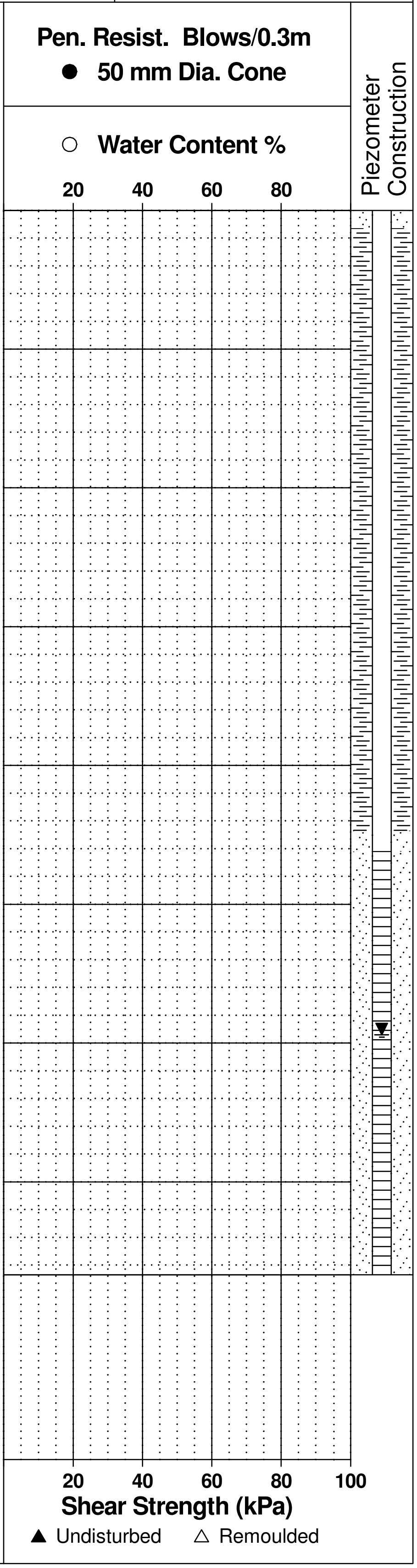
FILE NO. PG4915

HOLE NO. BH 3

SOIL DESCRIPTION	Bottom depth (m)
GROUND SURFACE	
Asphaltic concrete	0.15
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty sand with clay, gravel, trace plastic and topsoil	2.44
FILL: Brown silty sand with gravel and crushed stone, trace clay	4.09
BEDROCK: Heavily fractured to fractured, black shale	7.67
End of Borehole (GWL @ 5.94m - April 12, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	5
SS	3	62	11
SS	4	67	51
SS	5	79	78
SS	6	70	50+
SS	7	60	50+
SS	8	100	50+
SS	9	100	50+
SS	10	100	50+
SS	11	0	50+

DEPTH (m)	ELEV. (m)
0	56.47
1	55.47
2	54.47
3	53.47
4	52.47
5	51.47
6	50.47
7	49.47



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

DATE April 5, 2019

FILE NO. PG4915

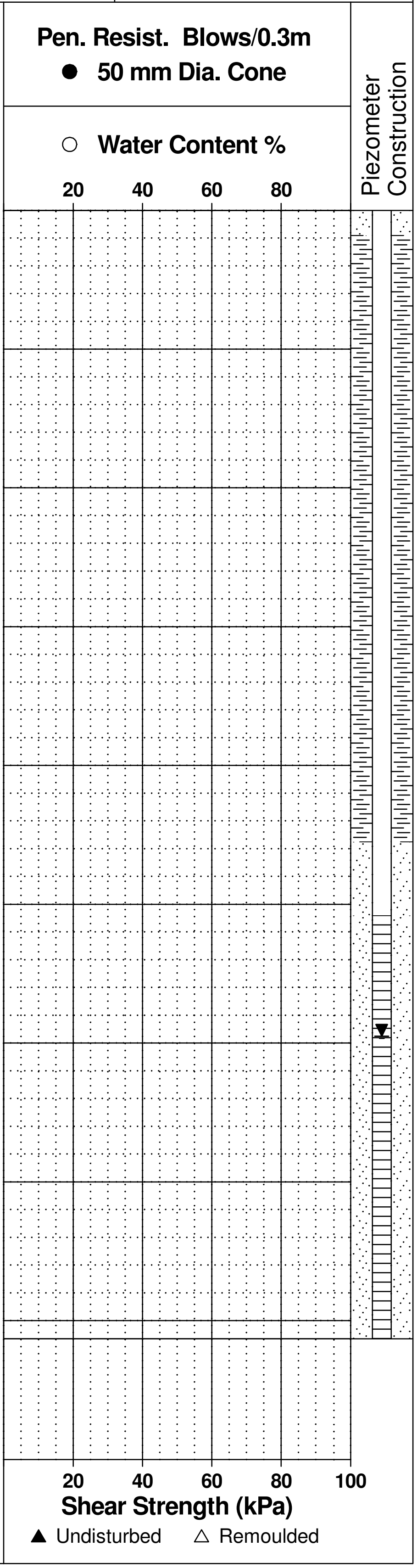
HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.15				
FILL: Crushed stone with sand	0.60	AU	1		
FILL: Brown silty clay, some sand, gravel, shale fragments, trace topsoil	1.45	SS	2	46	5
FILL: Dark brown silty sand with shale fragments and gravel	3.96	SS	3	42	24
		SS	4	54	71
		SS	5	100	50+
BEDROCK: Heavily fractured to fractured, black shale	8.13	SS	6	100	50+
		SS	7	100	50+
		SS	8	100	50+
		SS	9	67	50+
		SS	10	0	50+
		SS	11	0	50+

End of Borehole

(GWL @ 5.95m - April 12, 2019)

DEPTH (m)	ELEV. (m)
0	56.50
1	55.50
2	54.50
3	53.50
4	52.50
5	51.50
6	50.50
7	49.50
8	48.50



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

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REMARKS

BORINGS BY CME 45 Power Auger

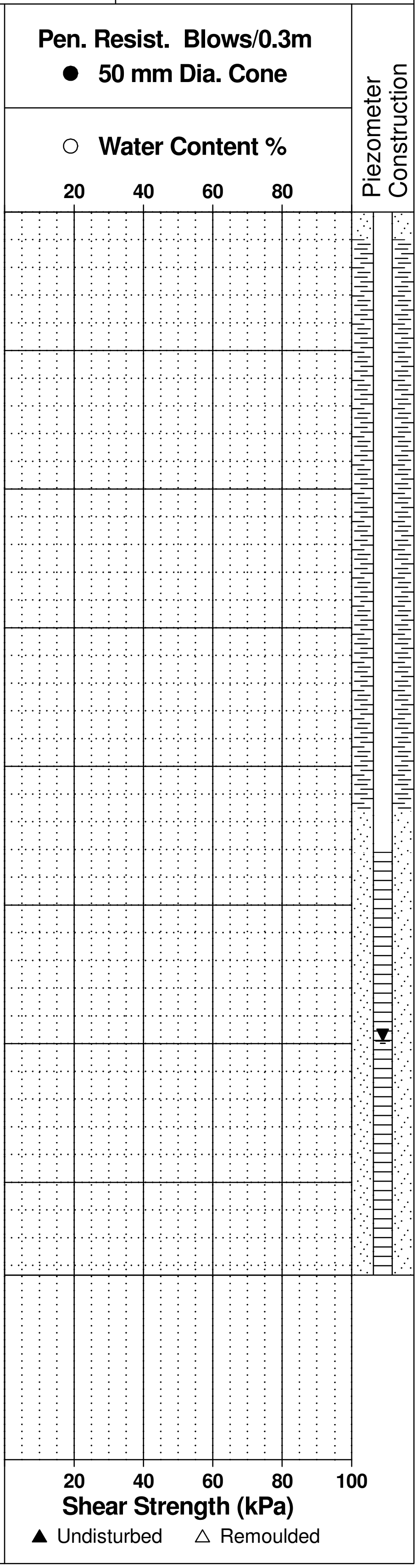
DATE April 5, 2019

FILE NO. PG4915

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.18				
FILL: Dark brown silty clay with sand and gravel, some topsoil and shale fragments	AU	1		
	SS	2	50	7
- clay content decreasing with depth 2.21	SS	3	54	29
FILL: Brown silty sand with crushed stone, gravel and shale fragments, trace clay 2.90	SS	4	62	56
GLACIAL TILL: Very dense, grey sandy silt to silty sand, some gravel, trace clay	SS	5	100	65
	SS	6	83	46
5.26	SS	7	100	68
BEDROCK: Heavily fractured to fractured, black shale	SS	8	33	50+
	SS	9	44	50+
7.67	SS	10	0	50+
End of Borehole (GWL @ 5.98m - April 12, 2019)				

DEPTH (m)	ELEV. (m)
0	56.55
1	55.55
2	54.55
3	53.55
4	52.55
5	51.55
6	50.55
7	49.55



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

DATE April 4, 2019

FILE NO. PG4915

HOLE NO. BH 6

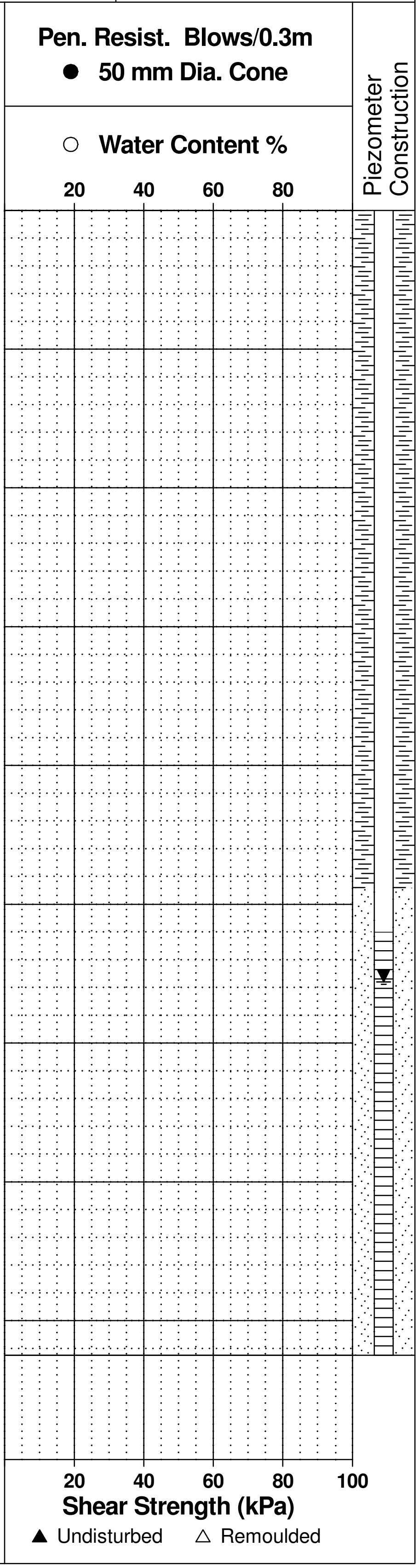
GROUND SURFACE

SOIL DESCRIPTION	Base depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Asphaltic concrete	0.13				
FILL: Brown silty sand with crushed stone, some clay	0.60	AU	1		
FILL: Light brown silty clay with sand and gravel, trace topsoil	1.37	SS	2	67	21
FILL: Brown silty sand with gravel and shale fragments, trace concrete	4.72	SS	3	88	
		SS	4	100	46
		SS	5	92	72
		SS	6	92	70
BEDROCK: Heavily fractured to fractured, black shale	8.25	SS	7	67	50+
		SS	8	100	58
		RC	1	78	31
		RC	2	79	0

End of Borehole

(GWL @ 5.56m - April 12, 2019)

DEPTH (m)	ELEV. (m)
0	56.69
1	55.69
2	54.69
3	53.69
4	52.69
5	51.69
6	50.69
7	49.69
8	48.69



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 45 Power Auger **DATE** April 5, 2019

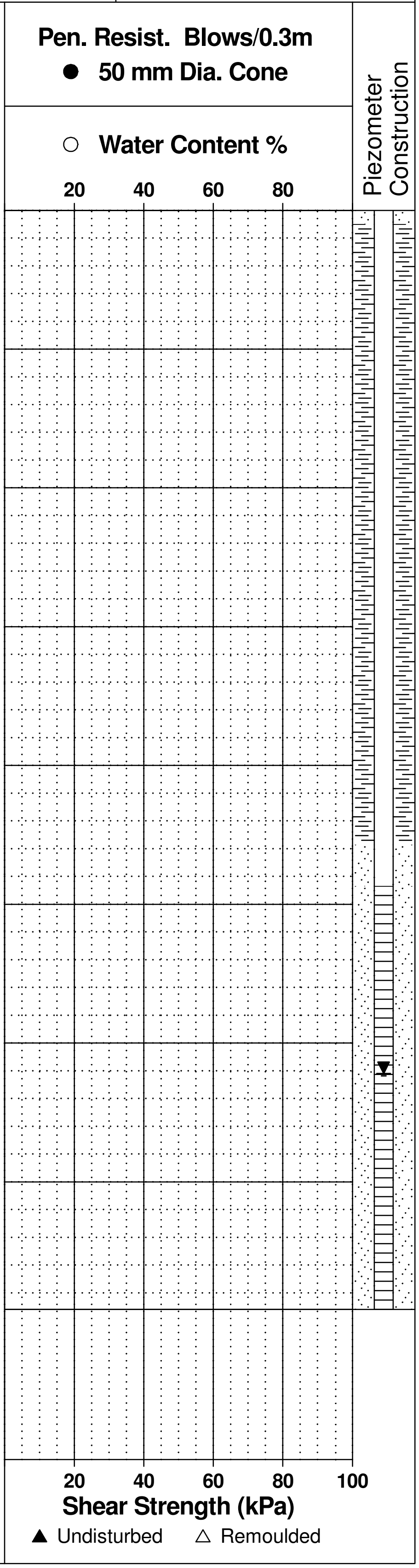
FILE NO. PG4915

HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.13
FILL: Brown silty sand to sandy silt with crushed stone	0.46
FILL: Dark brown silty clay with sand and gravel, some topsoil, trace organics and shale fragments - clay content decreasing with depth	2.21
FILL: Brown silty sand with crushed stone and gravel, some shale fragments	4.85
BEDROCK: Heavily fractured to fractured, black shale	7.92
End of Borehole (GWL @ 6.22`m - April 12, 2019)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	15
SS	3	33	19
SS	4	58	61
SS	5	100	50+
SS	6	0	50+
SS	7	82	50+
SS	8	12	94
SS	9	62	51
SS	10	8	8
SS	11	0	50+

DEPTH (m)	ELEV. (m)
0	56.75
1	55.75
2	54.75
3	53.75
4	52.75
5	51.75
6	50.75
7	49.75



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SOIL PROFILE AND TEST DATA

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3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

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REMARKS

BORINGS BY CME 45 Power Auger

DATE April 3, 2019

FILE NO. PG4915

HOLE NO. BH 8

GROUND SURFACE

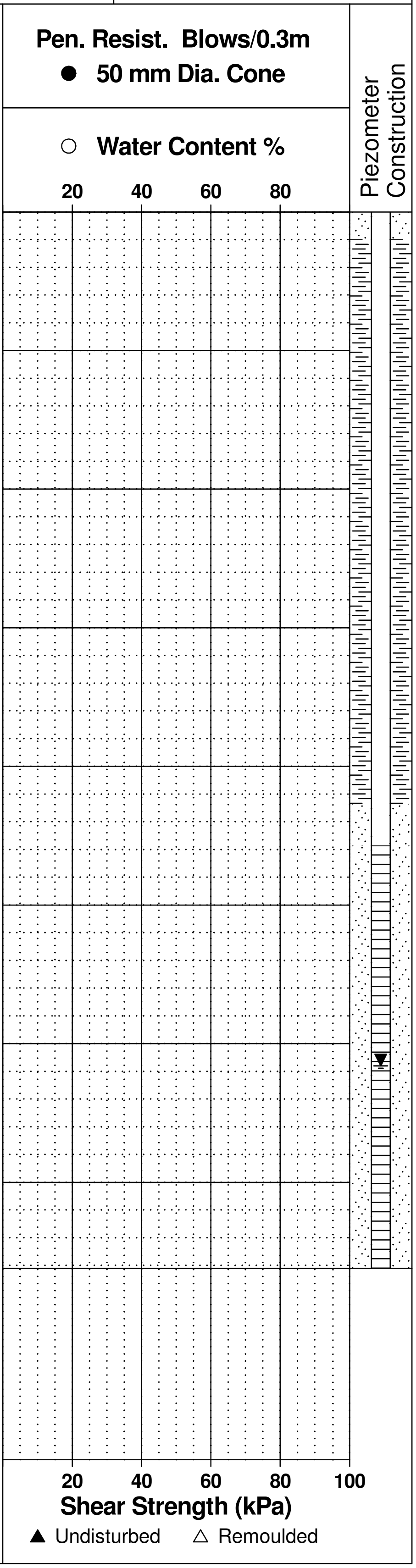
SOIL DESCRIPTION	Depth to base (m)
Asphaltic concrete	0.15
FILL: Brown silty sand with crushed stone	0.51
FILL: Brown silty sand, trace clay, gravel and brick	3.66
BEDROCK: Heavily fractured to fractured, black shale	7.62

End of Borehole

(GWL @ 6.16m - April 12, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	8
SS	3	46	9
SS	4	75	17
SS	5	58	62
SS	6	100	87
SS	7	88	53
SS	8	0	50+
SS	9	25	14
SS	10	38	33

DEPTH (m)	ELEV. (m)
0	56.70
1	55.70
2	54.70
3	53.70
4	52.70
5	51.70
6	50.70
7	49.70



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

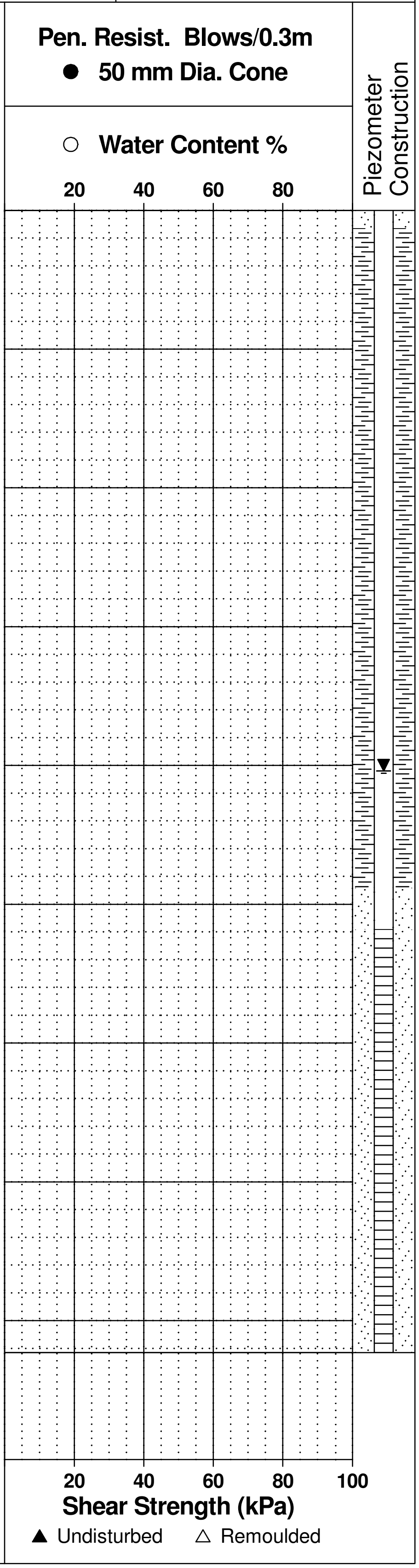
DATE April 4, 2019

FILE NO. PG4915

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete (0.10)				
FILL: Brown silty sand with crushed stone (0.60)	AU	1		
FILL: Dark brown to black silty clay with gravel, cobbles, sand and shale fragments, trace topsoil (5.64)	SS	2	88	10
	SS	3	54	12
	SS	4	83	12
	SS	5	100	12
	SS	6	12	23
	SS	7	33	13
	SS	8	75	25
GLACIAL TILL: Compact to very dense, grey sandy silt to silty sand with gravel and shale fragments (8.23)	SS	9	100	56
	SS	10	83	91
	SS	11	88	67
End of Borehole				
(GWL @ 4.04m - April 12, 2019)				

DEPTH (m)	ELEV. (m)
0	56.66
1	55.66
2	54.66
3	53.66
4	52.66
5	51.66
6	50.66
7	49.66
8	48.66



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street & 2 Montreal Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 307 Montgomery Street. Geodetic elevation = 57.63m.

REMARKS

BORINGS BY CME 45 Power Auger

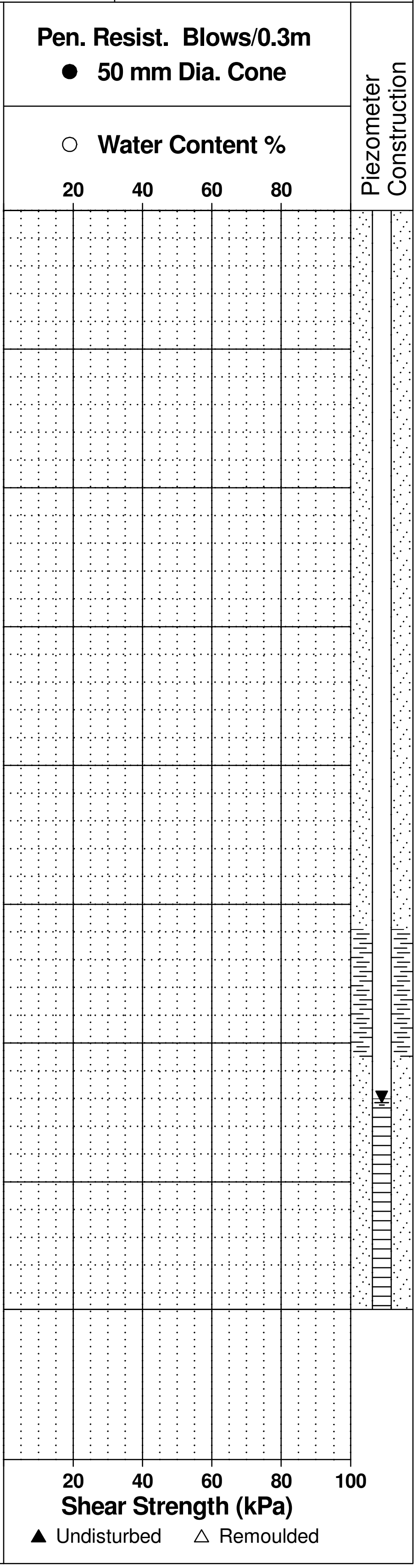
DATE May 4, 2019

FILE NO. PG4915

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete slab 0.13		AU	1				
FILL: Brown silty sand with gravel		SS	2	67			
1.22						1	
GLACIAL TILL: Dark brown silty sand with gravel, cobbles and shale fragments		AU	3			2	
		SS	4	50		3	
		SS	5	12			
		SS	6	17		4	
		SS	7	8		5	
5.79		SS	8	10			
BEDROCK: Heavily fractured to fractured, black shale		RC	1	44	0	6	
		RC	2	100	0		
		RC	3	74	0	7	
7.92		RC	4	58	28		
End of Borehole							

(GWL @ 6.43m - April 12, 2019)



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)

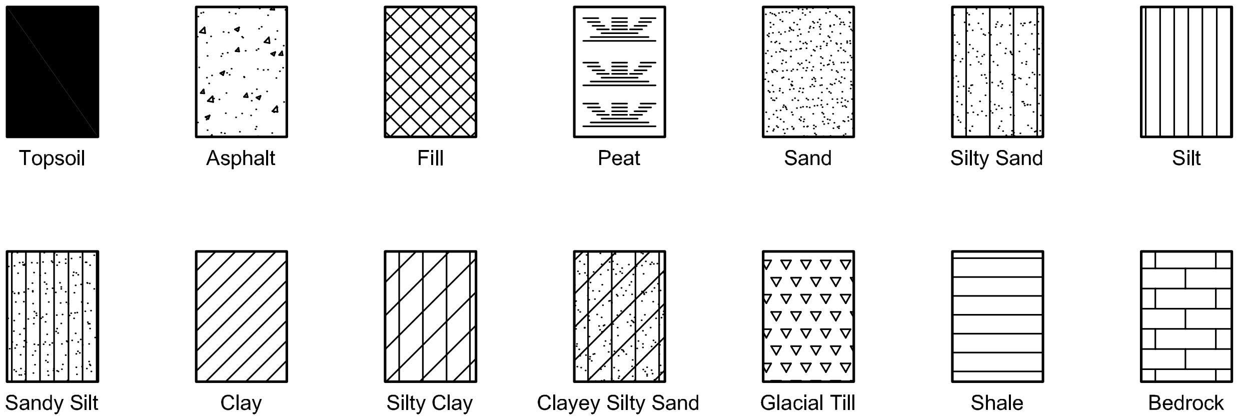
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

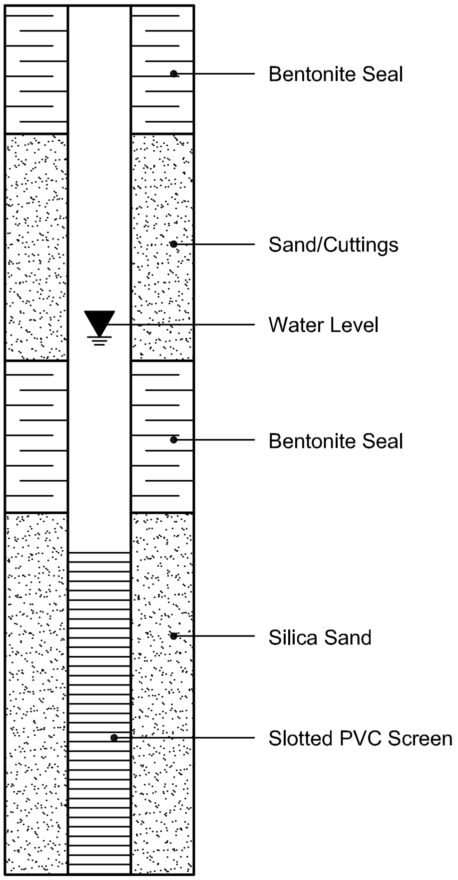
SYMBOLS AND TERMS (continued)

STRATA PLOT

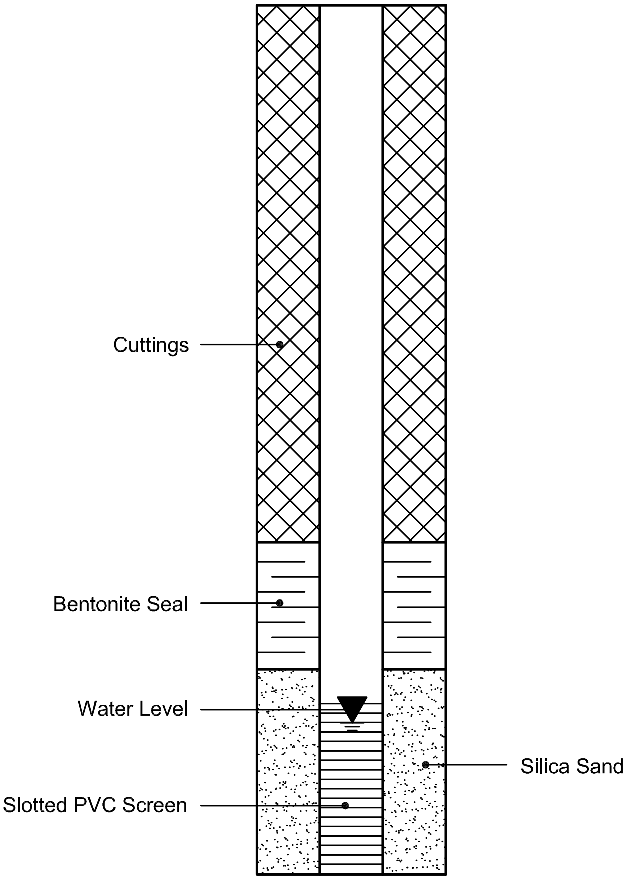


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - GROUNDWATER SUPPRESSION SYSTEM

FIGURES 3 & 4 - SHEAR WAVE VELOCITY TEST PROFILES

DRAWING PG4915-1 - TEST HOLE LOCATION PLAN

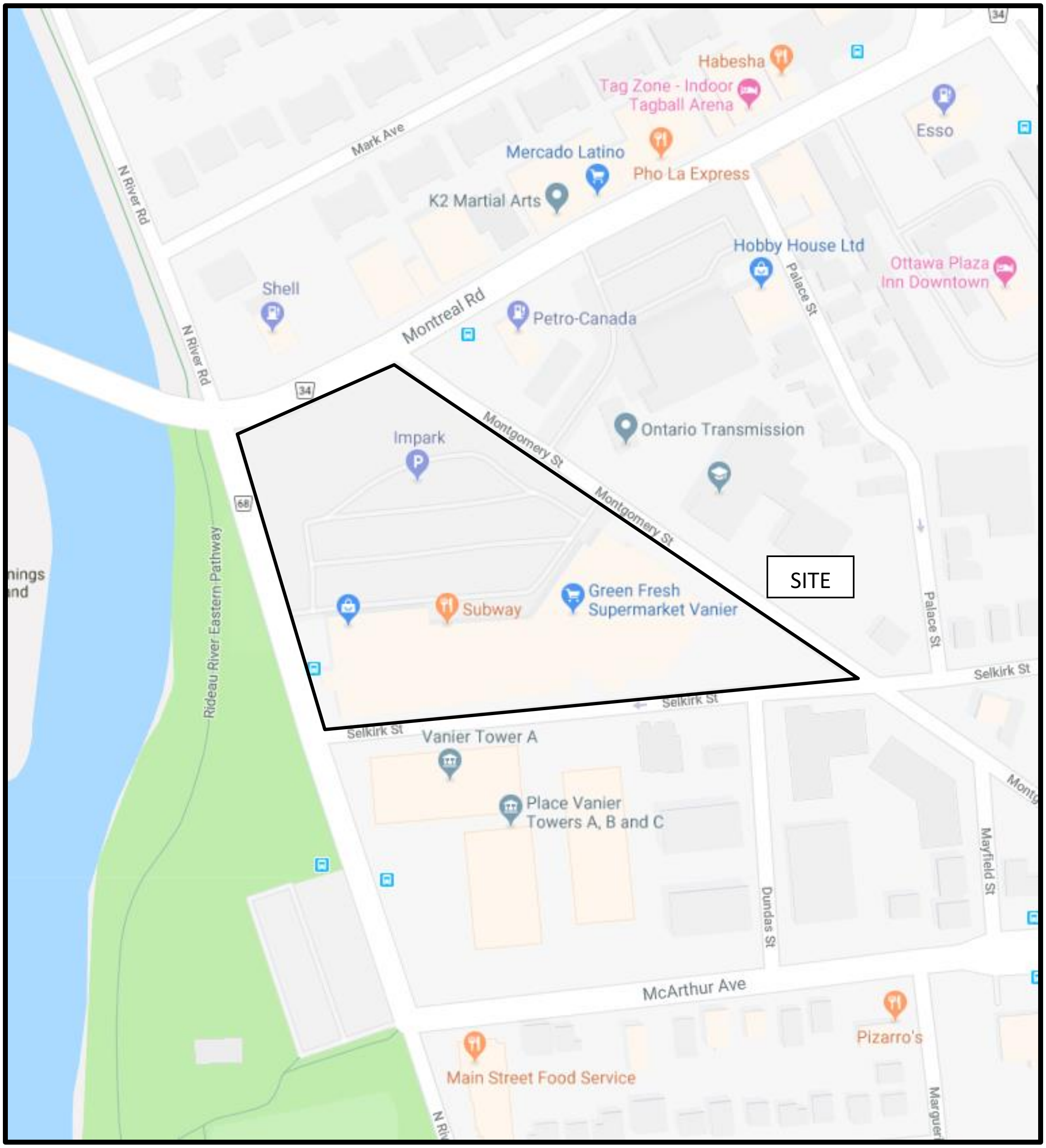
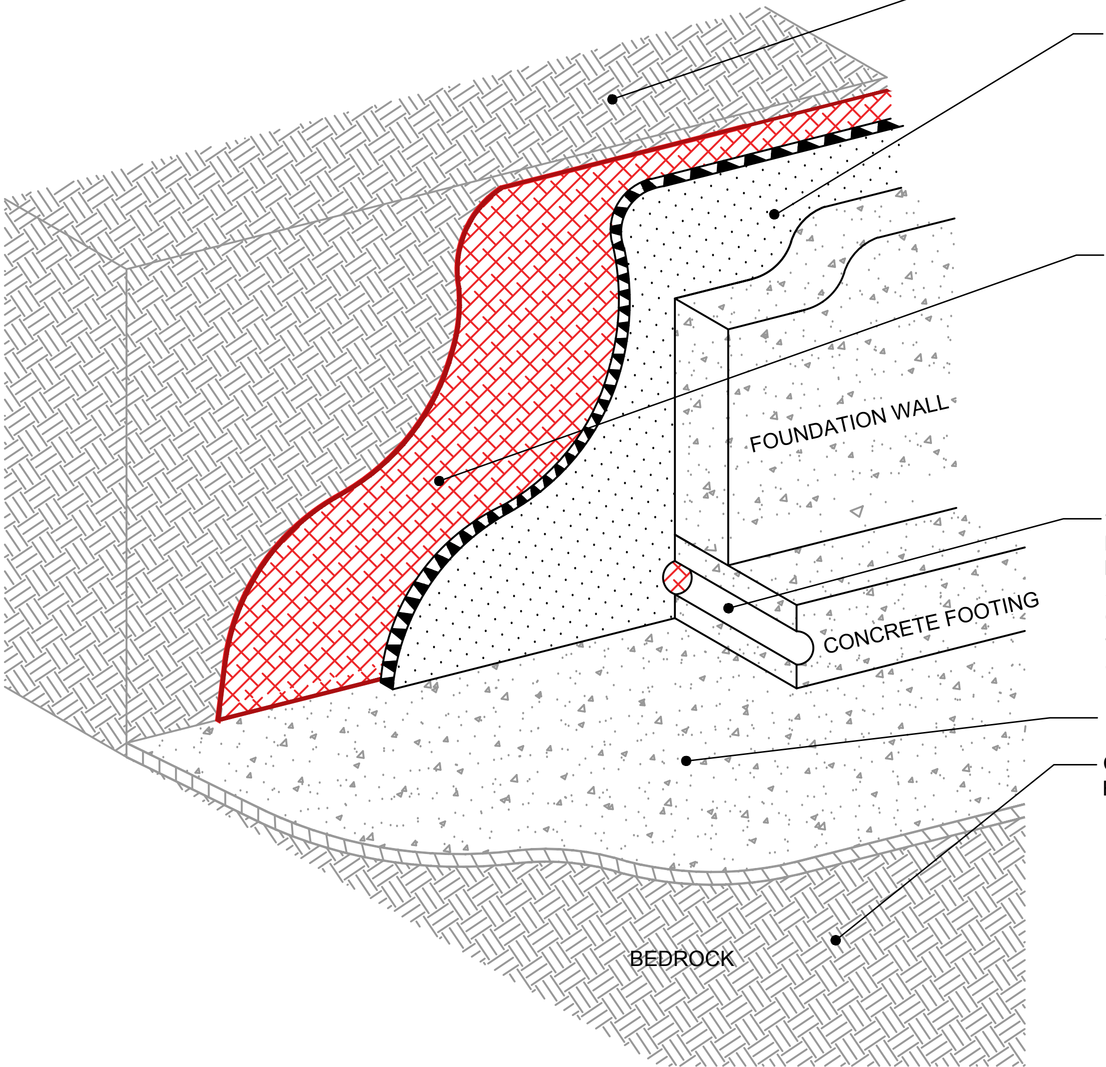


FIGURE 1

KEY PLAN



SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

COMPOSITE FOUNDATION DRAINAGE MEMBRANE (SUCH AS DELTA DRAIN 6000, MIRADRAIN G100N OR EQUIVALENT). INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

WATERPROOFING MEMBRANE WILL CONSIT OF 150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE SUCH AS TREMCO PARASEAL OR EQUIVALENT. TREMCO PARASEAL TO BE INSTALLED IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO EXTEND FROM 3.0m BELOW FINISHED GRADE TO UNDERSIDE OF FOOTING

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE FOOTING AT APPROXIMATELY 3m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE. THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

150mm THICK, 15 MPa CONCRETE MUD SLAB (OPTIONAL)

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEAOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE WATER RETARDATION SYSTEM .

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MAIN AND MAIN DEVELOPMENTS INC.

PROPOSED HI-RISE COMPLEX

3-33 SELKRIK STREET AND 2 MONTREAL ROAD

OTTAWA, ONTARIO

Title:

GROUND WATER SUPPRESSION SYSTEM

Scale: N.T.S.	**Date:** 05/2019
Drawn by: RCG	**Report No.:** PG4915-1
Checked by: RG	**Drawing No.:** FIGURE 2
Approved by: DG	**Revision No.:**

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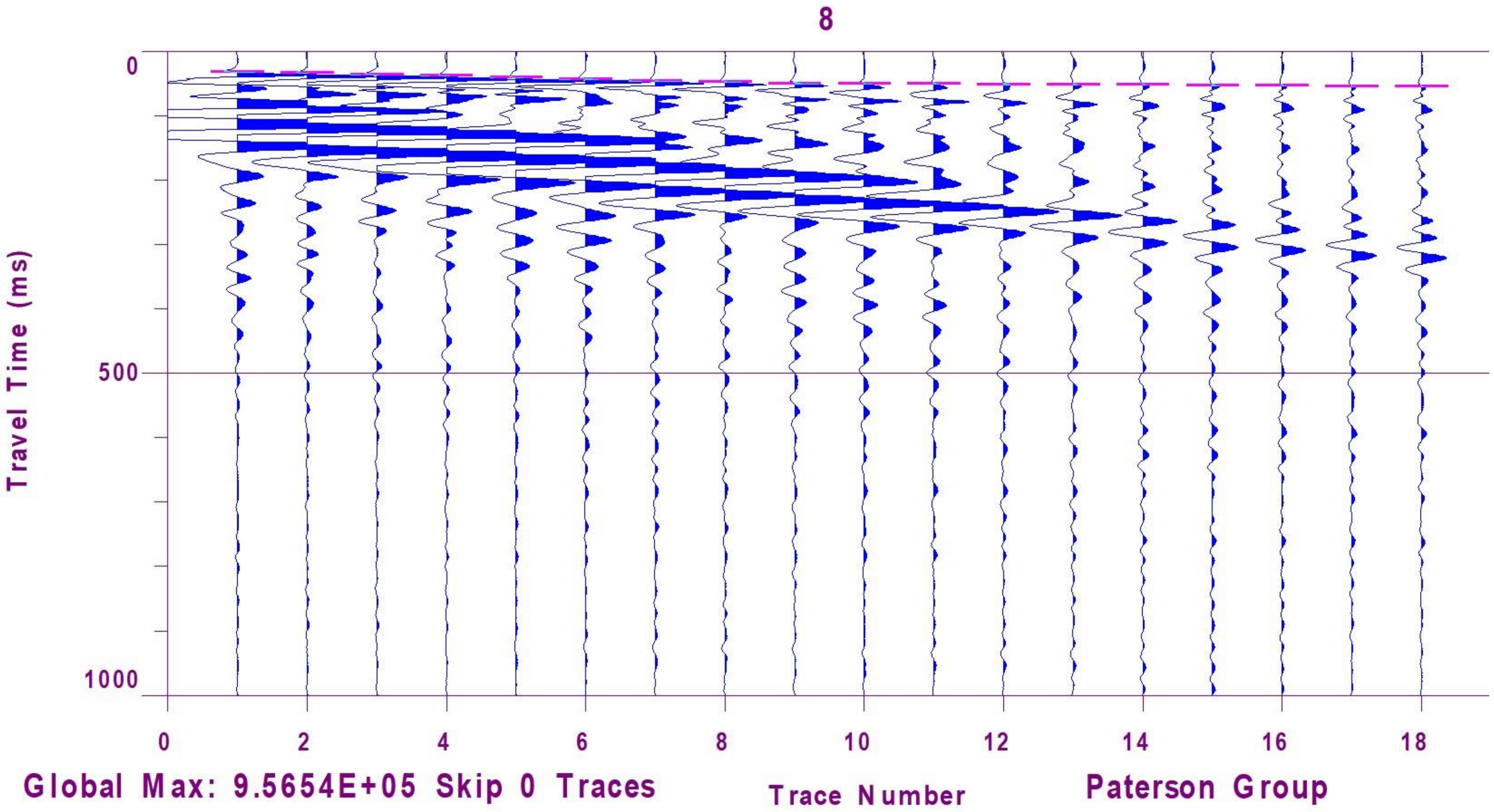


Figure 3 – Shear Wave Velocity Profile at Shot Location -15 m

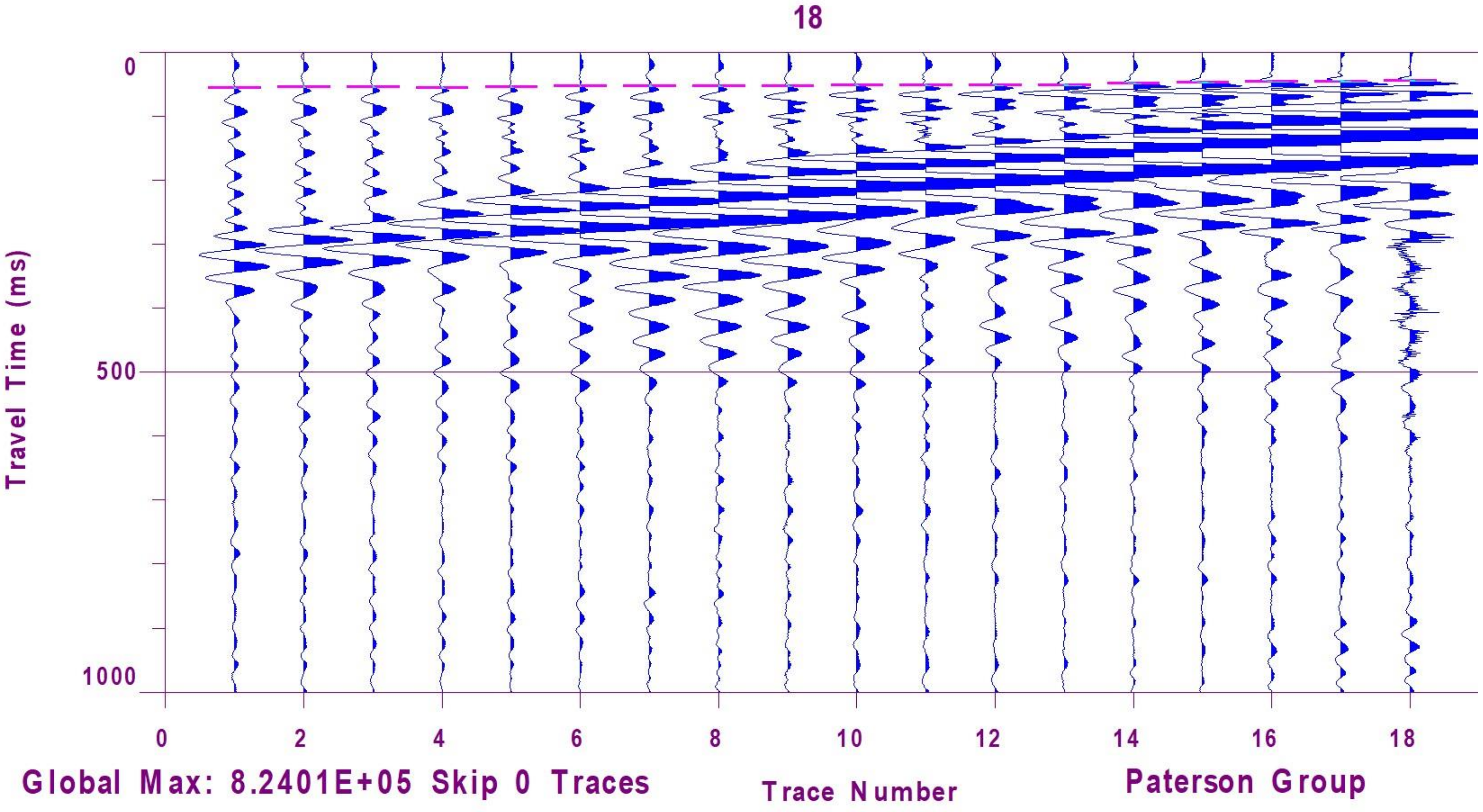
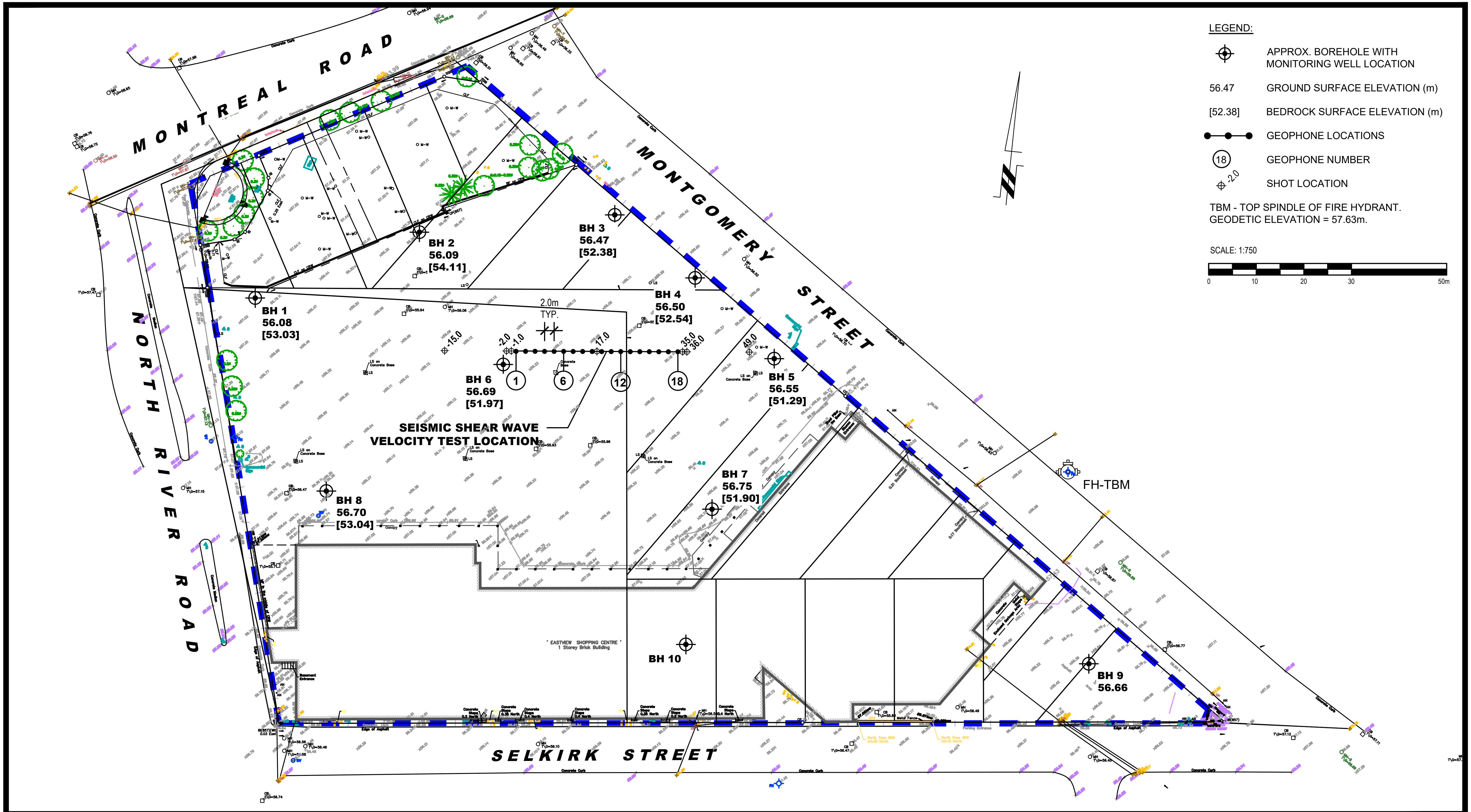


Figure 4 – Shear Wave Velocity Profile at Shot Location +49 m



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NO.	REVISIONS	DATE	INITIAL
1	BASE PLAN REVISED & SEISMIC SURVEY ADDED	20/05/2020	DP

MAIN AND MAIN DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
3-33 SELKIRK STREET AND 2 MONTREAL ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 05/2019
Drawn by: RCG
Report No.: PG4915-1
Checked by: RG
Approved by: DJG
PG4915-1
Revision No.: 1

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