PATERSON
GROUP

Geotechnical Investigation

Proposed High-Rise Building

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

Prepared for Main and Main Developments Inc.

Report PG4915-2 dated August 8, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Main and Main Developments Inc. to conduct a geotechnical investigation for the proposed development to be 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 objective of the geotechnical investigation was to:

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

- 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.

Investigating the presence or potential presence of contamination on the subject site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the current conceptual drawings for Phase 3 of the proposed development, it is our understanding that a 28-storey high-rise building with a 2-storey podium level will be constructed over an underground parking structure with one basement level.

It is further expected that the proposed development will be municipally serviced with water and sewer services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was undertaken on May 11 and between July 4 and July 7, 2023. During that time, a total of 4 boreholes (BH 1-23 to BH 4-23) and 5 test pits (TP 1-23 to TP 5-23) were advanced to a maximum depth of 12.0 and 8.5 m below the existing ground surface, respectively. Previous investigations were undertaken between April 3, 2019 and February 3, 2022. During that time, a total of 5 boreholes (BH 7, BH 9, BH 10, BH 7-20, and BH 2-22) and 3 test pits (TP 13-21, TP 17-21, and TP 18-21) were advanced to a maximum depth of 9.4 m below the existing ground surface.

The test holes undertaken by Paterson were located in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration 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. The test pits were advanced using a hydraulic shovel. 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 and test pitting procedure consisted of advancing to the required depths at select locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the sidewalls of the test pits and from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment.

All samples were visually inspected and initially classified on site. The sidewall, auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the grab, auger, split spoon and rock core samples were recovered from the test holes are shown as G, AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented 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.

Diamond drilling was carried out at several 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 test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

During previous investigations 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 carried out by Paterson 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 and are referenced to a geodetic datum.

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by construction site trailers and associated equipment and material storage and laydown areas. The previously existing one-storey commercial plaza has been demolished as part of the proposed development and aforementioned site operations.

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 neighboring properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations throughout the subject site consisted of a layer of fill underlain by a deposit of glacial till which is further underlain by the bedrock formation. The fill layer was observed to extend to depths between 0.6 to 5.6 m below existing ground surface. The fill layer generally consisted of silty sand with some gravel and crushed stone. The underlying glacial till layer was found to consist of dark brown silty sand with clay, gravel, shale fragments, cobbles and boulders.

A heavily fractured weathered shale bedrock was encountered below the glacial till deposit at depths between 2.2 and 8.2 m throughout the subject site.

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 5 m. Reference can be made to Drawing PG4915-2 - Bedrock Contour Plan for the test hole locations and depth which bedrock had been encountered.

4.3 Groundwater

Groundwater levels were measured in monitoring wells on July 17, 2023. 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 to be 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 Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-23	56.67	6.50	50.17	July 17, 2023
BH 2-23	56.63	6.57	50.06	
BH 3-23	56.73	6.56	50.17	
BH 4-23	56.67	6.45	50.22	
TP 1-23	56.54	DRY	N/A	May 11, 2023
TP 2-23	56.80	8.00	48.80	
TP 3-23	56.62	7.50	49.12	
TP 4-23	56.45	7.50	48.95	
TP 5-23	56.48	7.50	48.98	
BH 2-22	57.05	7.01	50.04	March 2, 2022
BH 7	56.75	6.22	50.53	April 12, 2019
BH 9	56.66	4.04	52.62	
BH 10	57.07	6.43	50.64	

Notes: The test holes were surveyed with respect to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed high-rise building be founded on conventional spread footing foundations placed directly upon a clean, surface sounded shale bedrock and/or compact to dense, in-situ, native glacial till bearing surface. In some areas, over-sized footings and/or raft slabs may be required to provide sufficient foundation support for the design building loads. The associated bearing resistance and subgrade modulus values have been provided in Section 5.3 accordingly.

Foundations supporting the underground parking structure and overlying podium level located beyond the high-rise portion of the building may be supported by conventional spread footings placed on the aforementioned bedrock bearing mediums and/or a weathered bedrock bearing surface, an undisturbed, compact glacial till bearing medium or suitable and site-approved existing fill.

Where foundation loads exceed the bearing pressures provided herein for the existing fill layer, recommendations have been provided for the reinstatement of the bearing medium to attain a suitable bearing surface.

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 the proposed underground parking level. 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.

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

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming and/or controlled blasting will be required to remove sound 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 in conjunction with conventional excavation techniques, such as the use of a hydraulic excavator.

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.

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.

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 susceptible to degradation over time and can result in localized loss of support where it is placed below settlement sensitive structures.

5.3 Foundation Design

Bearing Resistance Values – High-Rise Building

Footings placed on a surface sounded shale bedrock 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.

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

Footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **750 kPa**, incorporating a geotechnical resistance factor of 0.5. For this case, the modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **500 kPa**.

Localized bearing resistance values of up to **590 kPa** (SLS) may be able to be considered on a footing- and loading-condition basis and as required and assessed by Paterson at the foundation design stage. For this case, a footing- and loading case-specific modulus of subgrade reaction was calculated to be **23.6 MPa/m** for a contact pressure of **590 kPa** will be able to be considered for these isolated zones and as based on review by Paterson at the time of detailed structural design.

Lean Concrete Filled Trenches

Alternatively, where bedrock is not encountered at the design underside of footing elevation for footings where a bedrock bearing resistance value and bearing medium is sought as part of the foundation design, consideration may be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**17 MPa** 28-day compressive strength).

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying sound bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. Given the highly weathered nature of the in-situ shale formation, it is expected the upper 1 to 2 m of shale bedrock will be removed readily using a hydraulic excavator. It would be recommended to sub-excavated a minimum depth of 600 mm below the bedrock surface using this methodology to attain a potentially suitable bedrock bearing surface.

It is recommended that Paterson personnel review the sub-excavation work prior to completing trenches to verify the confirm the depth of the bedrock surface to mitigate potential over-excavation into the shale formation. Once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Bearing Resistance Values – Parking Garage and Podium Level

Based on our review, footings supporting the remainder of the proposed structure may be founded upon a combination of the existing fill, in-situ glacial till and/or weathered bedrock bearing mediums. Consideration may be required to be given to replacing the existing fill layer with a suitably prepared pad of engineered fill where the footing loads exceed the bearing pressure provided herein for the existing fill layer. Further, footings founded upon glacial till and bedrock bearing mediums should be provided a bedrock-to-soil bearing medium transition treatment as described herein.

Footings placed on a surface sounded shale bedrock 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.

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

Footings placed on an undisturbed, dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **750 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed on the existing and site-approved fill bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **80 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **120 kPa**, incorporating a geotechnical resistance factor of 0.5.
All existing fill bearing surfaces should be proof-rolled at the time of construction. Proof-rolling of the existing fill should be reviewed at the time of construction by Paterson personnel.

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

Lean Concrete Filled Trenches

Alternatively, where bedrock is not encountered at the design underside of footing elevation for footings where a bedrock bearing resistance value and bearing medium is sought as part of the foundation design, consideration may be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**17 MPa** 28-day compressive strength).

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying sound bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. Given the highly weathered nature of the in-situ shale formation, it is expected the upper 1 to 2 m of shale bedrock will be removed readily using a hydraulic excavator. It would be recommended to sub-excavated a minimum depth of 600 mm below the bedrock surface using this methodology to attain a potentially suitable bedrock bearing surface.

It is recommended that Paterson personnel review the sub-excavation work prior to completing trenches to verify the confirm the depth of the bedrock surface to mitigate potential over-excavation into the shale formation. Once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Soil/Bedrock Transition

It is anticipated the majority of the footings supporting will be founded on glacial till or approved fill. However, where a footing may be founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on a soil bearing medium to reduce the potential for long-term total and differential settlements.

At the soil/bedrock transitions, it is recommended that a minimum depth of 300 mm of bedrock be removed from below the founding elevation for a minimum length of 2.0 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or OPSS Granular B Type II crushed stone and compacted to a minimum of 98% of the materials SPMDD.

Proof Rolling and Subgrade Improvement for Unsuitable Existing Fill

Where the existing fill is encountered at the design founding elevation for footings, the existing fill should be sub-excavated to a depth of 1.0 m below USF (or shallower if native, in-situ, undisturbed soil or bedrock is encountered) and reinstated with a suitable engineered fill pad. The sub-excavation should extend a minimum of 500 mm beyond all faces of the affected footings, where required. The surface of the sub-excavation should be reviewed and approved by Paterson personnel and covered with a woven geotextile liner, such as Terrafix 200W, and further by a bi-axial geogrid layer, such as Terrafix TBX2000, once approved.

The geotextile and geogrid layers should be overlapped over the subgrade surface as specified by the manufacturer and reviewed and approved by Paterson prior to being covered with stone fill. The contractor should provide sufficient extensions of these layers to wrap the geotextile and geogrid layers around the stone layer.

The sub-excavated area should be backfilled to USF with granular fill, consisting of a Granular B Type II or Granular A crushed stone placed in maximum 300 mm loose lifts and compacted to 98% of its SPMDD. The geogrid and geotextile liners should be wrapped around the compacted granular fill with a minimum overlap of 500 mm along the top of stone layer as per manufacturer's recommendations.

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 or soil bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings placed on the glacial till deposit using the above-noted values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on an acceptable bedrock bearing surface and designed using 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 a portion of the subject site as part of the 2011 investigation 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}.

Based on our review, it is anticipated the finished floor elevation throughout the basement level will be the same as Phase 1 (i.e., 52.8 m). The lowest elevation the bedrock surface was encountered throughout the proposed building footprints was at a geodetic elevation of 49.2 m. It is expected footings will be founded on a combination of bedrock and in-situ soil. Based on the above-noted information, it is expected up to 3 m of overburden may be present between USF and the bedrock surface. The Vs_{30} was calculated considering the above-noted methodology and using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\,m}{\left(\frac{3\,m}{240\,m/s} + \frac{27\,m}{2{,}782\,m/s}\right)}$$

$$V_{s30=}\ 1{,}351\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed building is **1,351 m/s**. Therefore, a **Site Class B** is applicable for 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 fill within the footprint of the proposed building, the in-situ soil and/or bedrock surfaces will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. 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 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.

Consideration could be given to re-using site-generated shale to raise the subgrade level throughout the basement level, however, eexcavated shale deteriorates upon exposure to air and is susceptible to degradation over time. This can result in localized loss of support where it is placed below settlement sensitive structures. If considered for re-use below the proposed floor slab in areas that will not be used for parking, it is recommended the shale layer be capped with a minimum 200 mm thick layer of OPSS Granular A compacted to a minimum of 98% of the materials SPMDD.

A sub-slab drainage system consisting of lines of perforated drainage pipes should be connected to a sump pump located within the lowest basement level. The spacing and layout of the sub-slab drainage system should be provided by Paterson once the foundation layout has been finalized.

The spacing may be subject to change based on groundwater conditions encountered at the time of construction and as reviewed by the geotechnical consultant.

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.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

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.

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.

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.

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

5.8 Pavement Design

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 - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Table 5 - Recommended Pavement Structure – Local Residential Roadways, 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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 6 - Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or 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 100% of the SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

❏ For blind-sided pours, a composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the shoring system and bedrock excavation face from the finished ground surface to the top of the footing.

❏ For double-sided pours, a composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the building face with the HDPE face applied to the exterior concrete foundation wall face from the finished ground surface to the top of the footing.

❏ It is recommended that 150 mm diameter sleeves at 3 m centers 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 sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

For design purposes, it is recommended the underfloor drainage system consist of 150 mm diameter perforate pipes surrounded by a geosock and a 150 mm thick layer of 19 mm clear crushed stone on all of its sides. Several north-south and east-west lines of pipes will be placed throughout the basement level, and as directed by Paterson, to direct water from the foundation drainage and perimeter subdrain systems to the sump pump system.

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.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch’n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall.

The Colphene Torch’n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer’s specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

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 heated structures. Such exterior structures require additional frost protection, such as 2.1 m of soil cover, or a reduced thickness of soil cover if rigid insulation is used.

The parking garage 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 Side Slopes

Unsupported Excavations

Excavation side slopes above the groundwater level extending to a maximum vertical height 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.

Consideration may also be given to providing a near-vertical 1 m high wedge at the bottom of the temporary side-slope in overburden materials and directly above the soil-bedrock interface. In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors or other means determined by the geotechnical consultant at the time of construction.

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.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 2 m centres.

Soil stockpiles, debris, and other forms of weight should not be considered for the purpose of securing the tarpaulins along the top of the slope. However, consideration may be given to restraining the tarpaulins with soil, sandbags, stone, etc. along the bottom of the side-slope. The tarpaulins should extend beyond the overburden and onto the bedrock surface. Reference should be made to Drawing PG4915-3 – Cross Section for additional information for carrying temporary open-cut excavations along the perimeter of the proposed building footprint.

Efforts should also be made to maintain dry surfaces at the bottom of the excavation footprints and along the bottom of side slopes.

Due to the extension of the excavation, multiple tarps are expected to be stitched in order to cover the entire excavation face. Therefore, a minimum horizontal and vertical overlapping of 600 mm will be required between tarp sections. The tarpaulins should be overlapped such that the top end-lap of lower tarps are placed a minimum of 600 mm below the bottom end-lap of higher tarps to promote sheet drainage along the tarps and away from directly onto the underlying overburden.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A minimum of 1 m horizontal ledge should remain between the unsupported excavation and bedrock surface.

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 Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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. Further, the bonded portion of the rock anchor should be fully extended below the sound bedrock surface.

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 be increased to a minimum thickness of 300 mm where located over a bedrock 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. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material's SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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, Conservation and Parks (MECP) 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 MECP.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

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

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

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

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

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

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 proposed foundation drainage design and requirements, including the implementation of the system and associated underfloor drainage system.
- Review of structural drawings from a geotechnical perspective.
- 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.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 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.

LICENSED PROFESSIONAL ENGINEER
August 8, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Main and Main Developments Inc. (1 email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 5, 2023

FILE NO. PG4915

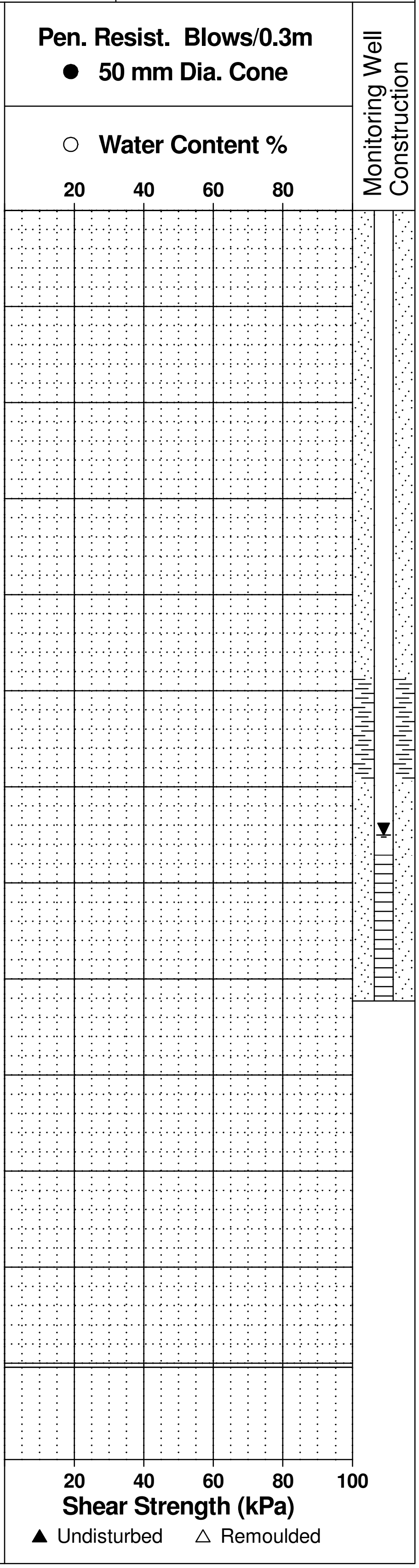
HOLE NO. BH 1-23

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.04				
FILL: Crushed stone with brown silty sand and gravel	0.28	AU	1		
FILL: Brown silty sand with gravel - trace organics to 1.5m depth - trace shale fragments from 0.9m depth		SS	2	100	16
		SS	3	83	23
	2.51	SS	4	44	50+
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles, boulders and shale		SS	5	100	50+
		RC	1	35	
		RC	2	24	
	6.15	SS	6	0	50+
BEDROCK: Fair to poor quality, black shale	7.50	RC	3	77	41
BEDROCK: Fair to excellent quality limestone with interbedded shale		RC	4	90	27
		RC	5	78	68
	12.04	RC	6	100	100

End of Borehole

(GWL @ 6.50m - July 17, 2023)

DEPTH (m)	ELEV. (m)
0	56.67
1	55.67
2	54.67
3	53.67
4	52.67
5	51.67
6	50.67
7	49.67
8	48.67
9	47.67
10	46.67
11	45.67
12	44.67



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 5, 2023

FILE NO. PG4915

HOLE NO. BH 1A-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						0	56.67		
						1	55.67		
OVERBURDEN						2	54.67		
						3	53.67		
3.81									
GLACIAL TILL: Very dense, dark brown silty sand with gravel, cobbles, boulders and shale	^^^	SS	1	100	50+	4	52.67		
4.83	^^^	SS	2	100	50+				
End of Borehole Practical refusal to augering at 4.83m depth.									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

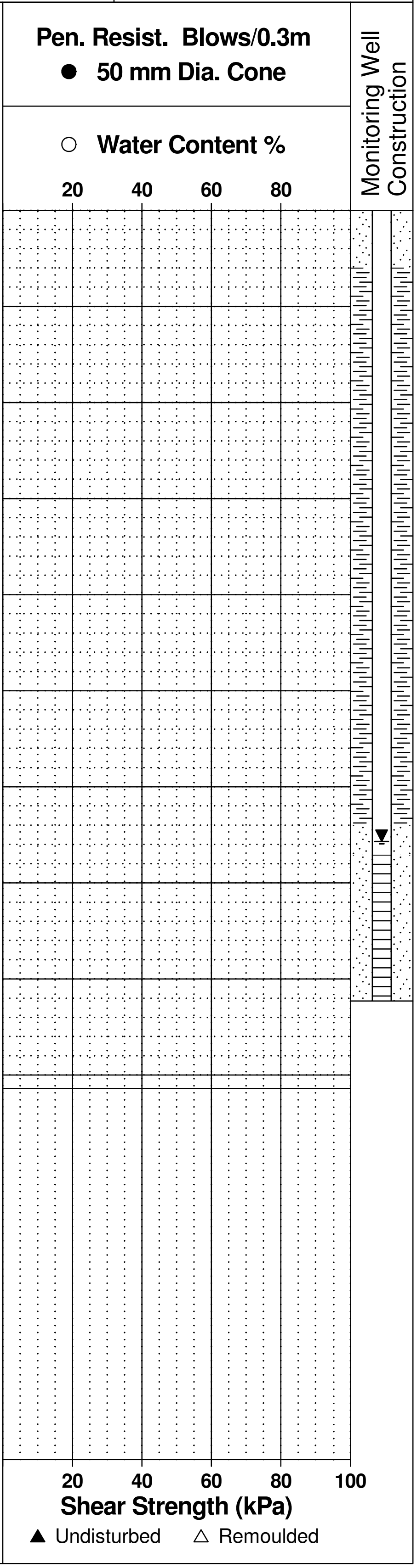
DATE July 4, 2023

FILE NO. **PG4915**

HOLE NO. **BH 2-23**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand, some gravel and crushed stone	AU	1		
	AU	2		
0.99	SS	3	83	11
GLACIAL TILL: Compact to very dense, dark brown silty sand, some to trace clay, gravel, occasional cobbles and boulders	SS	4	100	20
	SS	5	100	9
- trace shale fragments from 1.5m depth	SS	6	83	28
- shale content increasing with depth	SS	7	80	50+
4.42	SS	8	93	50+
BEDROCK: Fair to good quality, black shale	SS	9	33	50+
	RC	1	78	57
6.93	RC	2	98	78
BEDROCK: Fair to good quality, grey limestone with interbedded shale	RC	3	91	53
9.14				
End of Borehole (GWL @ 6.57m - July 17, 2023)				

DEPTH (m)	ELEV. (m)
0	56.63
1	55.63
2	54.63
3	53.63
4	52.63
5	51.63
6	50.63
7	49.63
8	48.63
9	47.63



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 7, 2023

FILE NO. PG4915

HOLE NO. BH 2A-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	56.63		
						1	55.63		
OVERBURDEN						2	54.63		
						3	53.63		
3.86									
BEDROCK: Fair quality, black shale	▤	RC	1	100	64	4	52.63		
4.93									
End of Borehole									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
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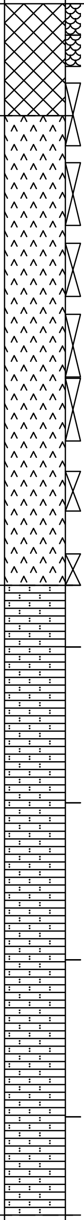
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE July 5, 2023
FILE NO. PG4915
HOLE NO. BH 3-23

GROUND SURFACE

SOIL DESCRIPTION	Depth to base (m)
FILL: Brown silty sand, some gravel and crushed stone	1.07
GLACIAL TILL: Compact to very dense, dark brown silty sand with gravel, cobbles, boulders and shale, trace clay	5.56
BEDROCK: Very poor to good quality, black shale - some interbedded grey limestone by 8.25m depth	11.58

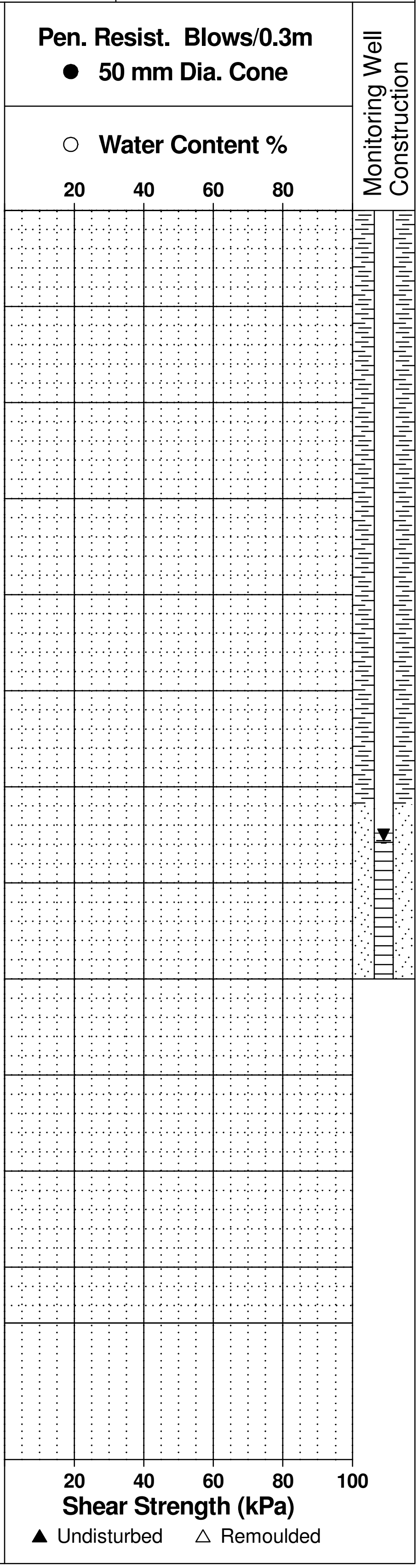
End of Borehole

(GWL @ 6.56m - July 17, 2023)



TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	100	25
SS	4	67	16
SS	5	100	42
SS	6	67	50+
SS	7	75	50+
SS	8	67	50+
SS	9	67	50+
RC	1	52	0
RC	2	59	19
RC	3	53	22
RC	4	92	80
RC	5	100	89

DEPTH (m)	ELEV. (m)
0	56.73
1	55.73
2	54.73
3	53.73
4	52.73
5	51.73
6	50.73
7	49.73
8	48.73
9	47.73
10	46.73
11	45.73



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 6, 2023

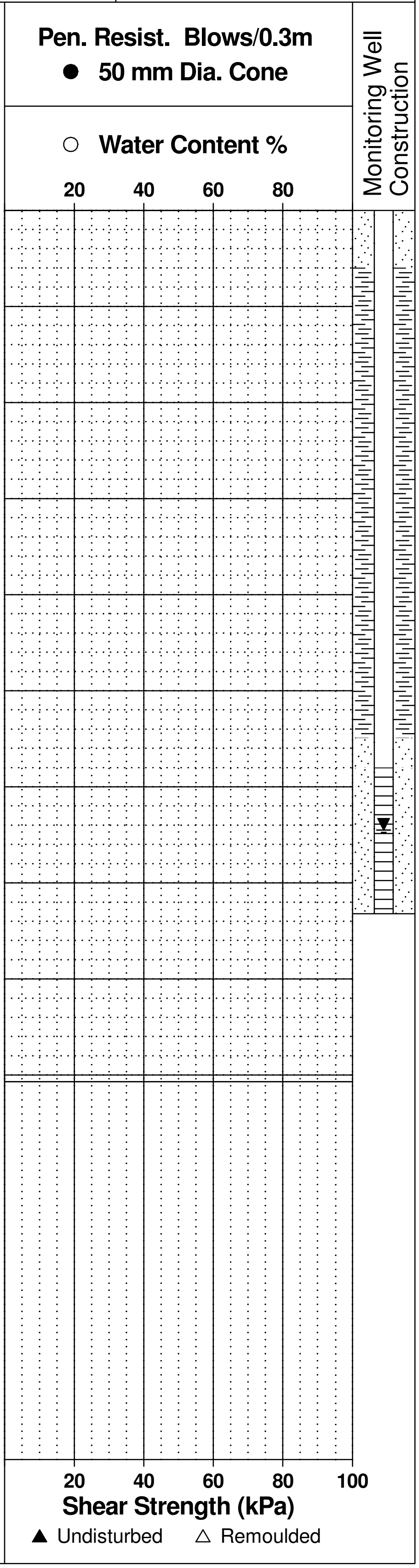
FILE NO. PG4915

HOLE NO. BH 4-23

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Crushed stone with silty sand and gravel	0.30
FILL: Brown silty sand, some gravel, crushed stone and concrete	3.05
GLACIAL TILL: Very dense, dark brown silty sand with gravel, cobbles, boulders and shale	4.70
BEDROCK: Very poor to good quality, black shale	5.16
BEDROCK: Good quality, grey limestone with interbedded shale	9.07
End of Borehole (GWL @ 6.45m - July 17, 2023)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	12	10
SS	4	50	8
SS	5	75	22
SS	6	100	50+
SS	7	100	50+
SS	8	100	50+
RC	1	60	0
RC	2	94	86
RC	3	78	64
RC	4	81	61

DEPTH (m)	ELEV. (m)
0	56.67
1	55.67
2	54.67
3	53.67
4	52.67
5	51.67
6	50.67
7	49.67
8	48.67
9	47.67



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 4, 2023

FILE NO. **PG4915**

HOLE NO. **BH 4A-23**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE						0	56.67		
						1	55.67		
OVERBURDEN						2	54.67		
						3	53.67		
3.81		SS	1	100	50+	4	52.67		
GLACIAL TILL: Very dense, dark brown silty sand with gravel, cobbles, boulders and shale		SS	2	100	50+	5	51.67		
		SS	3	100	50+				
6.02						6	50.67		
Weathered shale **BEDROCK**		SS	4	25	18				
7.26		SS	5	46	50+	7	49.67		
End of Borehole Practical refusal to augering at 7.26m depth.									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE May 11, 2023

FILE NO. PG4915

HOLE NO. TP 1-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	56.54
		G	1				
FILL: Brown silty sand, trace gravel and debris						1	55.54
		G	2				
2.10						2	54.54
		G	3				
						3	53.54
		G	4				
						4	52.54
		G	5				
GLACIAL TILL: Brown silty sand with gravel, trace clay, occasional cobbles and boulders						5	51.54
		G	6				
						6	50.54
		G	7				
						7	49.54
		G	8				
						8	48.54
		G	9				
8.50							
End of Test Pit							

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE May 11, 2023

FILE NO. PG4915

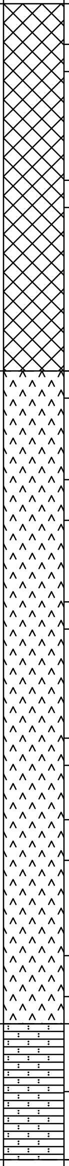
HOLE NO. TP 2-23

SOIL DESCRIPTION	Depth to base (m)
Ground Surface	
FILL: Brown silty sand, trace gravel	2.70
GLACIAL TILL: Brown silty sand some gravel, occasional cobbles and boulders -trace shale fragments by 6.0m depth	7.50
BEDROCK: Weathered shale, trace sand	8.50

End of Test Pit

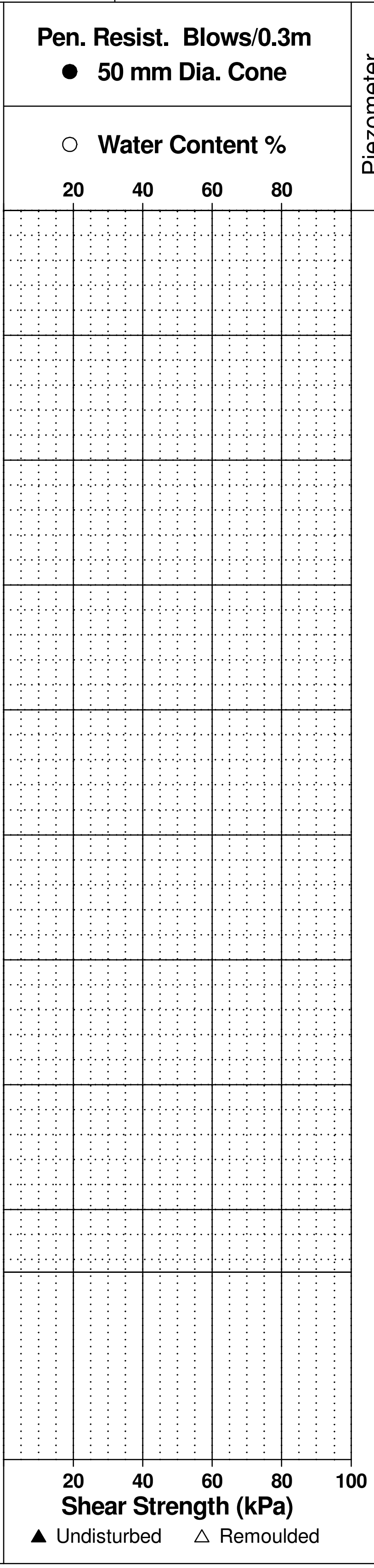
Test Pit terminated on bedrock surface.

(GW infiltration at 8.0m depth)



SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
G	1		
G	2		
G	3		
G	4		
G	5		
G	6		
G	7		
G	8		
G	9		

DEPTH (m)	ELEV. (m)
0	56.80
1	55.80
2	54.80
3	53.80
4	52.80
5	51.80
6	50.80
7	49.80
8	48.80



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE May 11, 2023

FILE NO. PG4915

HOLE NO. TP 3-23

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	56.62
FILL: Brown silty sand, trace gravel	G	1				
					1	55.62
	G	2				
					2	54.62
	G	3				
3.10					3	53.62
GLACIAL TILL: Brown silty sand, trace gravel and clay, occasional cobbles and boulders - trace shale fragments at 5.3m depth	G	4				
					4	52.62
	G	5				
					5	51.62
	G	6				
					6	50.62
	G	7				
					7	49.62
7.40						
BEDROCK: Weathered shale	G	8				
					8	48.62
8.30						

End of Test Pit

Test pit terminated on bedrock surface

(GW infiltration at 7.5m depth)

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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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE May 11, 2023

FILE NO. PG4915

HOLE NO. TP 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	56.45
		G	1				
FILL: Brown silty sand, with gravel, trace debris						1	55.45
		G	2				
						2	54.45
		G	3				
2.70							
						3	53.45
		G	4				
						4	52.45
		G	5				
GLACIAL TILL: Brown silty sand with gravel, trace clay, occasional cobbles and boulders						5	51.45
		G	6				
						6	50.45
- trace shale fragments at 6.0m depth							
		G	7				
						7	49.45
7.20		G	8				
BEDROCK: Weathered shale		G	9				
						8	48.45
8.20							
End of Test Pit Test pit terminated on bedrock surface. (GW infiltration at 7.5m depth)							

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, Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Excavator
DATE May 11, 2023
FILE NO. PG4915
HOLE NO. TP 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	56.48		
FILL: Brown silty sand, trace gravel and debris - trace gravel and clay by 0.8m depth 2.60	⊠	G G G	1 2 3			1 2	55.48 54.48		
GLACIAL TILL: Brown silty sand trace gravel, clay, occasional cobbles and boulders - trace shale fragments at 6.0m depth 7.20	^	G G G G G	4 5 6 7 8			3 4 5 6 7	53.48 52.48 51.48 50.48 49.48		
BEDROCK: Weathered shale 8.20	☰	G	9			8	48.48		

End of Test Pit

Test pit terminated on bedrock surface.

(GW infiltration at 7.5m depth)

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 4, 2022

FILE NO. PG4915

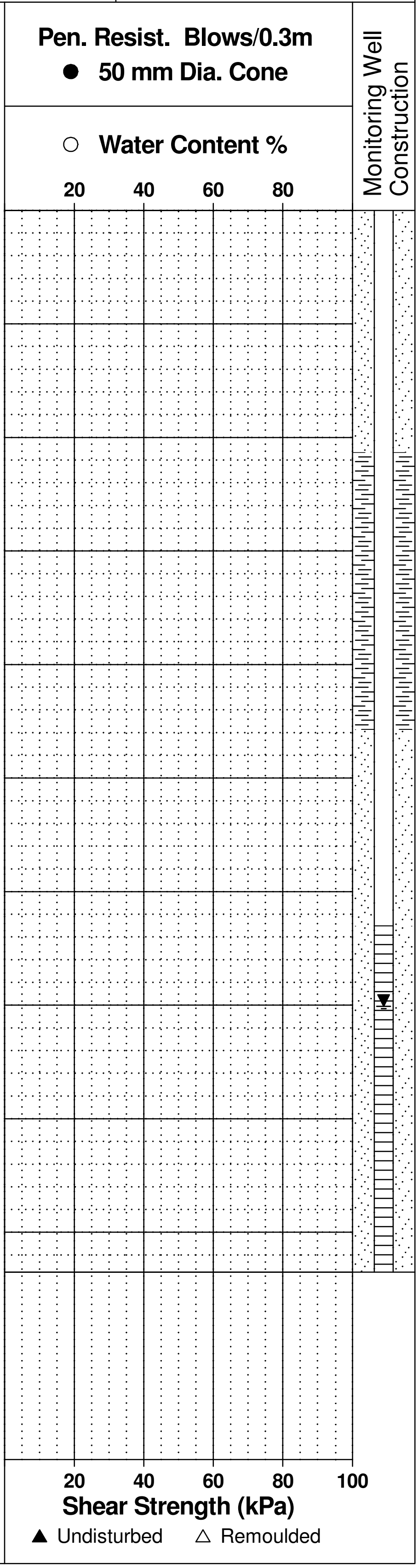
HOLE NO. BH 2-22

SOIL DESCRIPTION	Depth to bottom of layer (m)
GROUND SURFACE	
Concrete slab	0.21
FILL: Crushed stone	0.34
FILL: Brown silty sand, some gravel	1.52
GLACIAL TILL: Brown silty clay, some sand, gravel, trace cobbles and boulders	5.97
Weathered shale **BEDROCK**	6.70
BEDROCK: Poor to good quality, grey limestone with interbedded shale	9.35
End of Borehole	

(GWL @ 7.01m - March 2, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
G	1		
SS	2	81	
SS	3	71	
SS	4	74	
SS	5	100	
SS	6	85	
SS	7	0	
SS	8	0	
SS	9	0	
RC	1	68	0
RC	2	89	0
RC	3	100	100
RC	4	82	51
RC	5	87	18

DEPTH (m)	ELEV. (m)
0	57.05
1	56.05
2	55.05
3	54.05
4	53.05
5	52.05
6	51.05
7	50.05
8	49.05
9	48.05



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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, Ottawa, Ontario

DATUM Geodetic

REMARKS

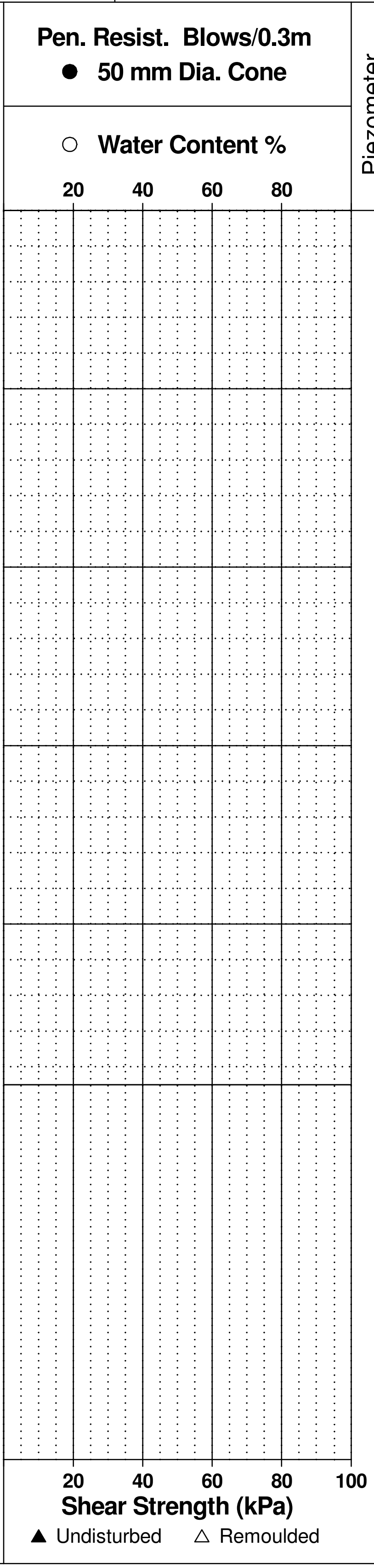
BORINGS BY Excavator

DATE 2021 December 22

FILE NO. PG4915

HOLE NO. TP13-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.72
Asphalt 0.08							
FILL: Brown silty sand with gravel and crushed stone 0.49		G	1				
FILL: Brown silty sand, some gravel and cobbles		G	2			1	55.72
1.10							
FILL: Brown silty clay some rock fragments, gravel and debris 1.50		G	3				
Brown **SILTY SAND** with rock fragments		G	4			2	54.72
						3	53.72
3.50							
GLACIAL TILL: Brown silty sand with rock fragments, gravel, trace cobbles and boulders		G	5				
		G	6			4	52.72
4.50							
Weathered shale **BEDROCK** 4.90							
End of Test Pit TP terminated on bedrock surface at 4.9 m depth							



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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, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG4915

REMARKS

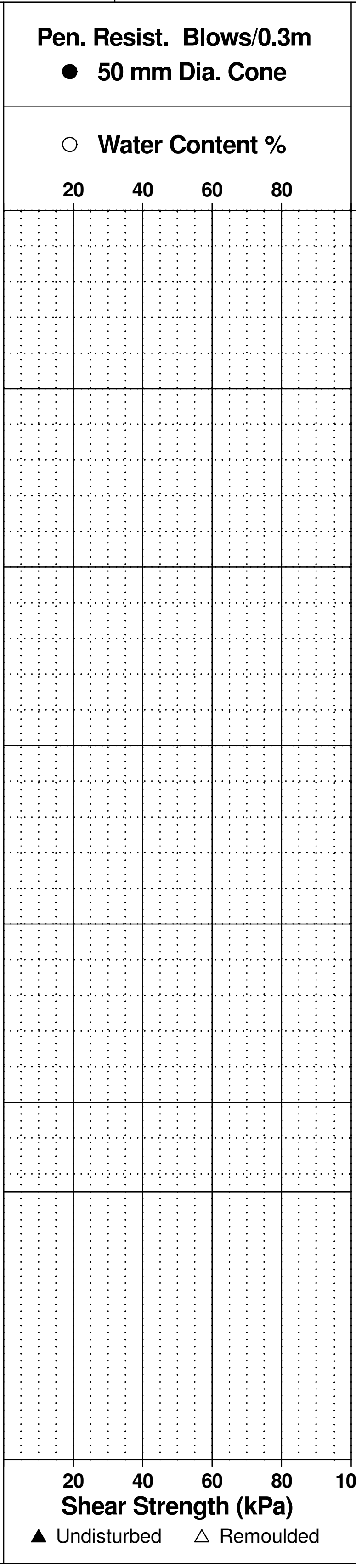
HOLE NO. TP17-21

BORINGS BY Excavator

DATE 2021 December 22

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphalt	0.08	G	1		
FILL: Brown silty sand with gravel and crushed stone	0.20	G	2		
FILL: Brown silty sand with rock fragments, trace clay, gravel and cobbles		G	3		
		G	4		
		G	5		
	5.00				
GLACIAL TILL: Grey silty sand trace clay, gravel, cobbles and boulders		G	6		
	5.50				
End of Test Pit					

DEPTH (m)	ELEV. (m)
0	56.01
1	55.01
2	54.01
3	53.01
4	52.01
5	51.01



Piezometer Construction

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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, Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG4915**

REMARKS

HOLE NO. **TP18-21**

BORINGS BY Excavator

DATE 2021 December 22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	56.56		
Asphalt 0.08									
FILL: Brown silty sand with gravel and crushed stone 0.29		G	1						
FILL: Brown silty sand with rock fragments, gravel and cobbles		G	2						
		G	3			1	55.56		
- Trace clay by 1.2 m depth									
		G	4			2	54.56		
						3	53.56		
		G	5						
						4	52.56		
		G	6						
						5	51.56		
6.00		G	7			6	50.56		
End of Test Pit									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
3-33 Selkirk Street and 2 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 18, 2020

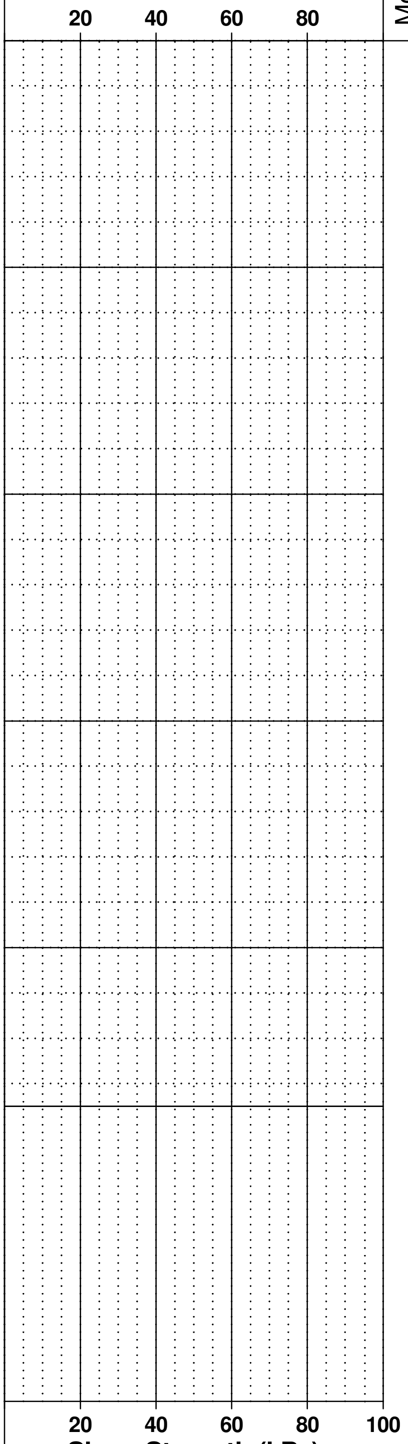
FILE NO. PG4915

HOLE NO. BH 7-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0	56.31	
Asphaltic concrete 0.08								
FILL: Brown silty sand with crushed stone 0.56		AU	1					
FILL: Dark brown silt, trace sand and gravel		SS	2	58	21	1	55.31	
		SS	3	79	30	2	54.31	
2.29								
GLACIAL TILL: Dense to very dense, brown silty sand-gravel and shale fragments		SS	4	88	48	3	53.31	
		SS	5	71	64			
		SS	6	70	50+	4	52.31	
4.70		SS	7	20	50+			
End of Borehole								

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

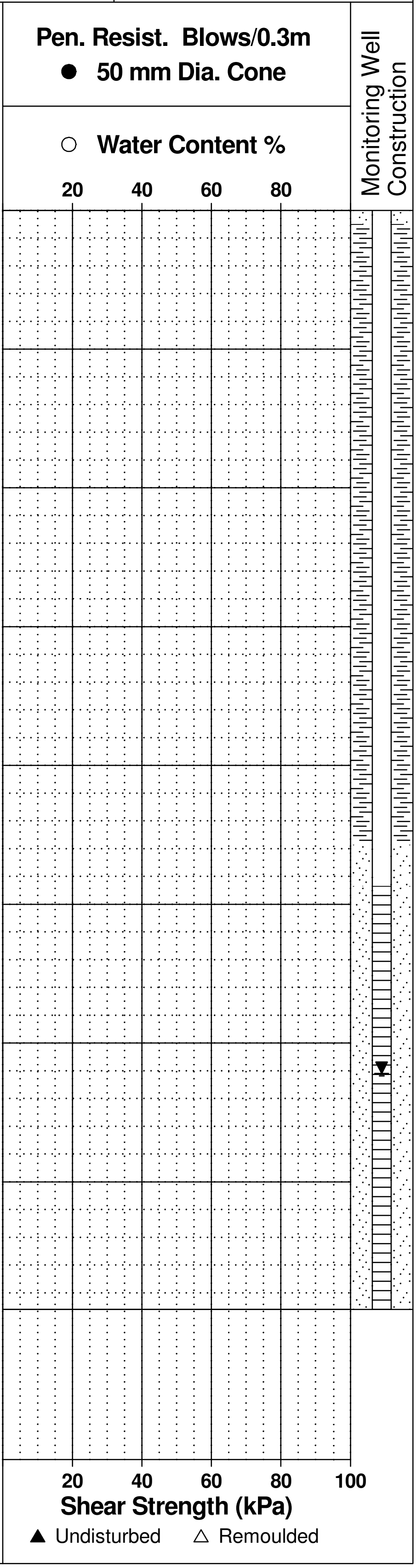
DATE April 5, 2019

FILE NO. PG4915

HOLE NO. BH 7

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.13				
FILL: Brown silty sand to sandy silt with crushed stone 0.46	AU	1		
FILL: Dark brown silty clay with sand and gravel, some topsoil, trace organics and shale fragments - clay content decreasing with depth	SS	2	42	15
2.21	SS	3	33	19
FILL: Brown silty sand with crushed stone and gravel, some shale fragments	SS	4	58	61
	SS	5	100	50+
	SS	6	0	50+
4.85	SS	7	82	50+
BEDROCK: Heavily fractured to fractured, black shale	SS	8	12	94
	SS	9	62	51
	SS	10	8	8
7.92	SS	11	0	50+
End of Borehole (GWL @ 6.22`m - April 12, 2019)				

DEPTH (m)	ELEV. (m)
0	56.74
1	55.74
2	54.74
3	53.74
4	52.74
5	51.74
6	50.74
7	49.74



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

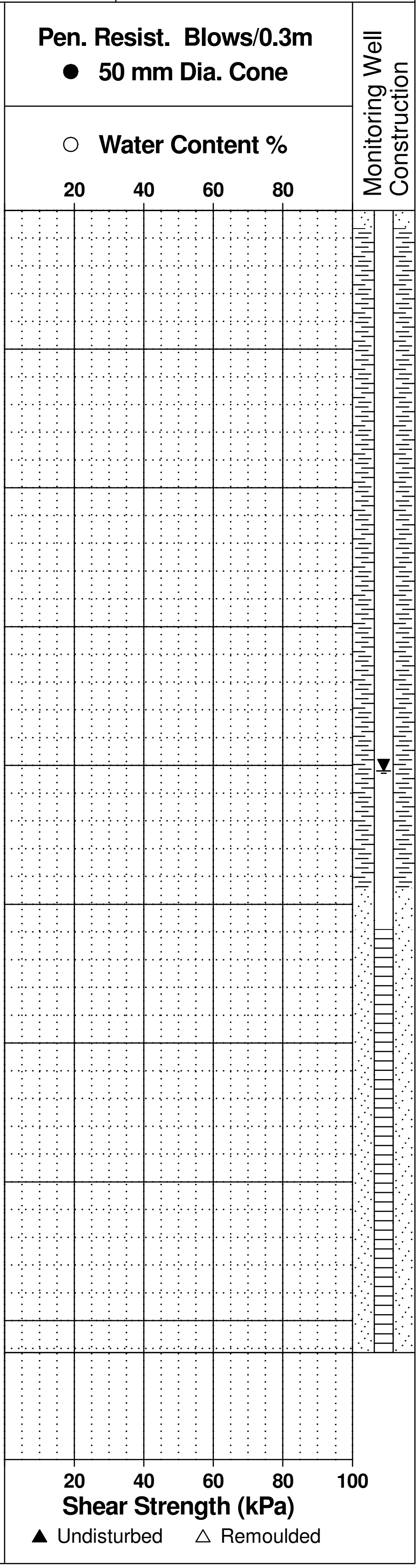
BORINGS BY CME 45 Power Auger

DATE April 4, 2019

FILE NO. PG4915

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.10					0	56.66
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	SS	2	88	10	1	55.66
	SS	3	54	12		
					2	54.66
	SS	4	83	12		
					3	53.66
	SS	5	100	12		
	SS	6	12	23	4	52.66
5.64	SS	7	33	13	5	51.66
GLACIAL TILL: Compact to very dense, grey sandy silt to silty sand with gravel and shale fragments	SS	8	75	25		
					6	50.66
	SS	9	100	56		
	SS	10	83	91	7	49.66
8.23	SS	11	88	67	8	48.66
End of Borehole						
(GWL @ 4.04m - April 12, 2019)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High-Rise Complex
3-33 Selkirk Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

DATE May 4, 2019

FILE NO. PG4915

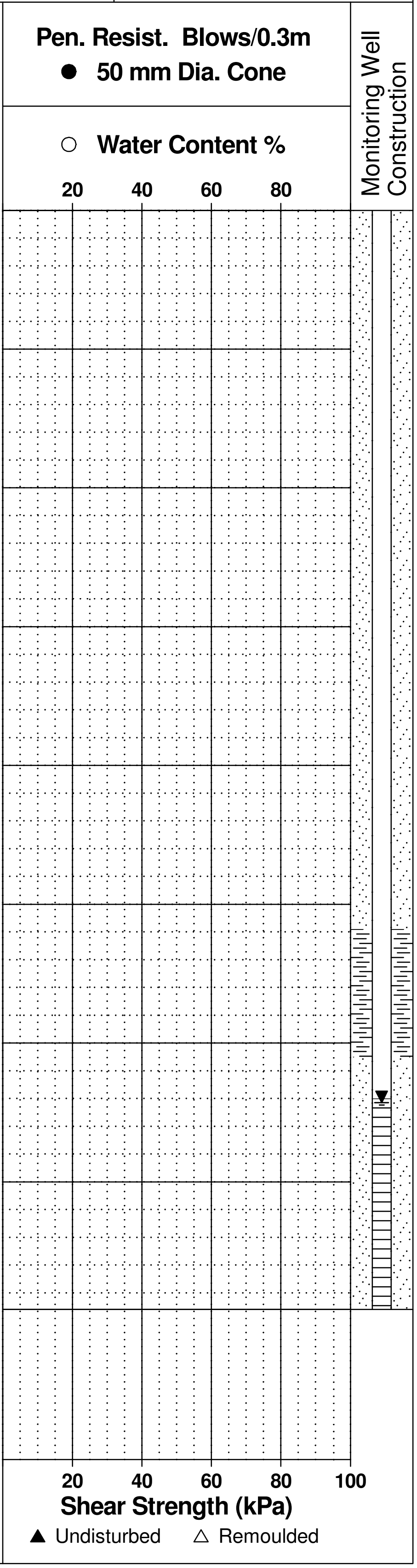
HOLE NO. BH10

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

DEPTH (m)	ELEV. (m)
0	57.07
1	56.07
2	55.07
3	54.07
4	53.07
5	52.07
6	51.07
7	50.07

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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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 = $(D30)^2 / (D10 \times D60)$
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)

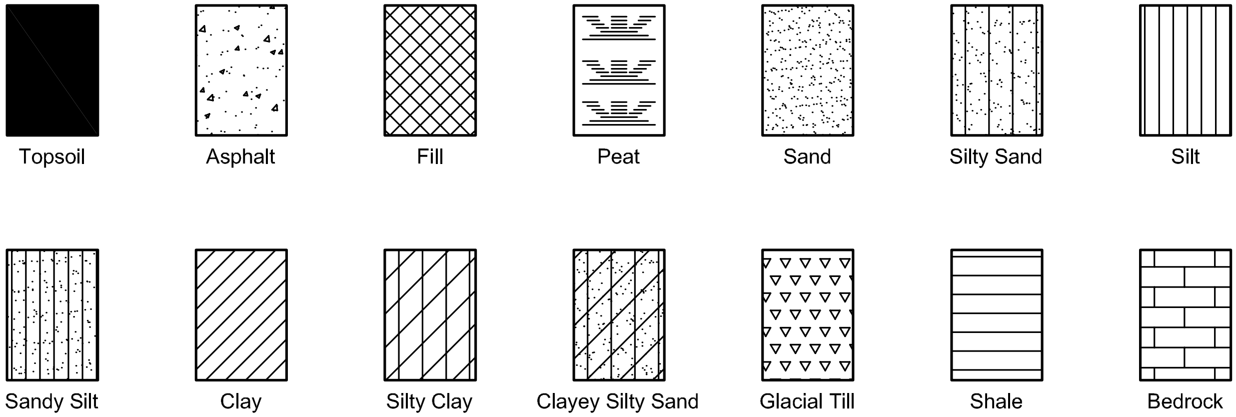
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

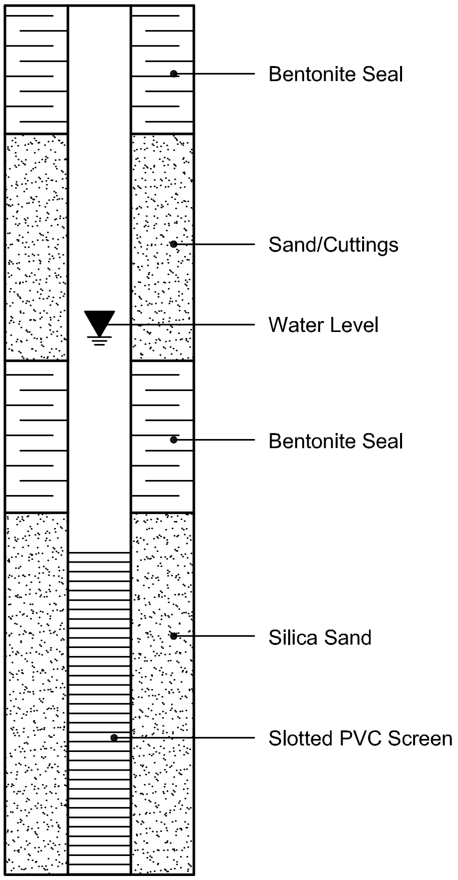
SYMBOLS AND TERMS (continued)

STRATA PLOT

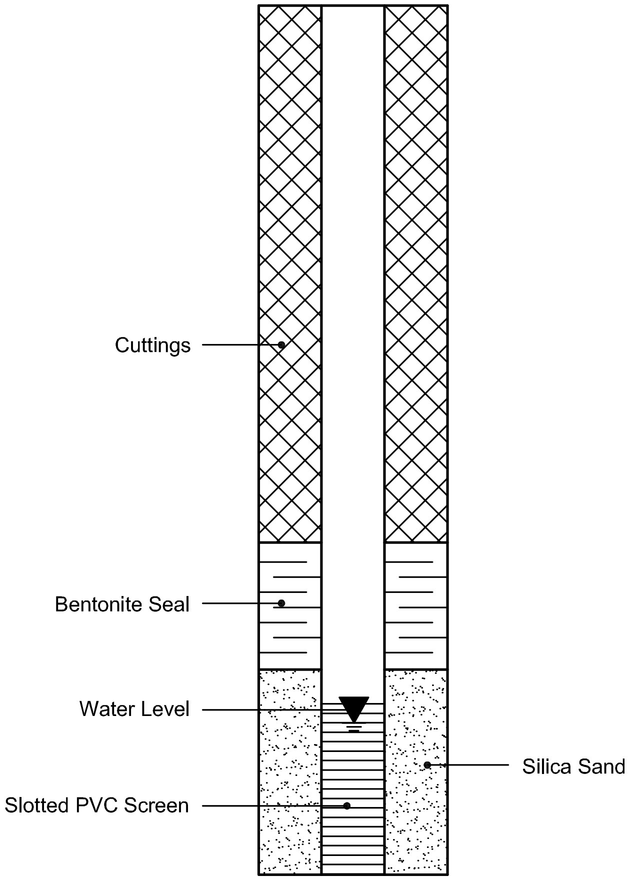


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 AND FIGURE 3 – SHEAR WAVE VELOCITY TESTING PROFILES

DRAWING PG4915-1 – TEST HOLE LOCATION PLAN

DRAWING PG4915-2 – BEDROCK CONTOUR PLAN

DRAWING PG4915-3 – CROSS SECTION

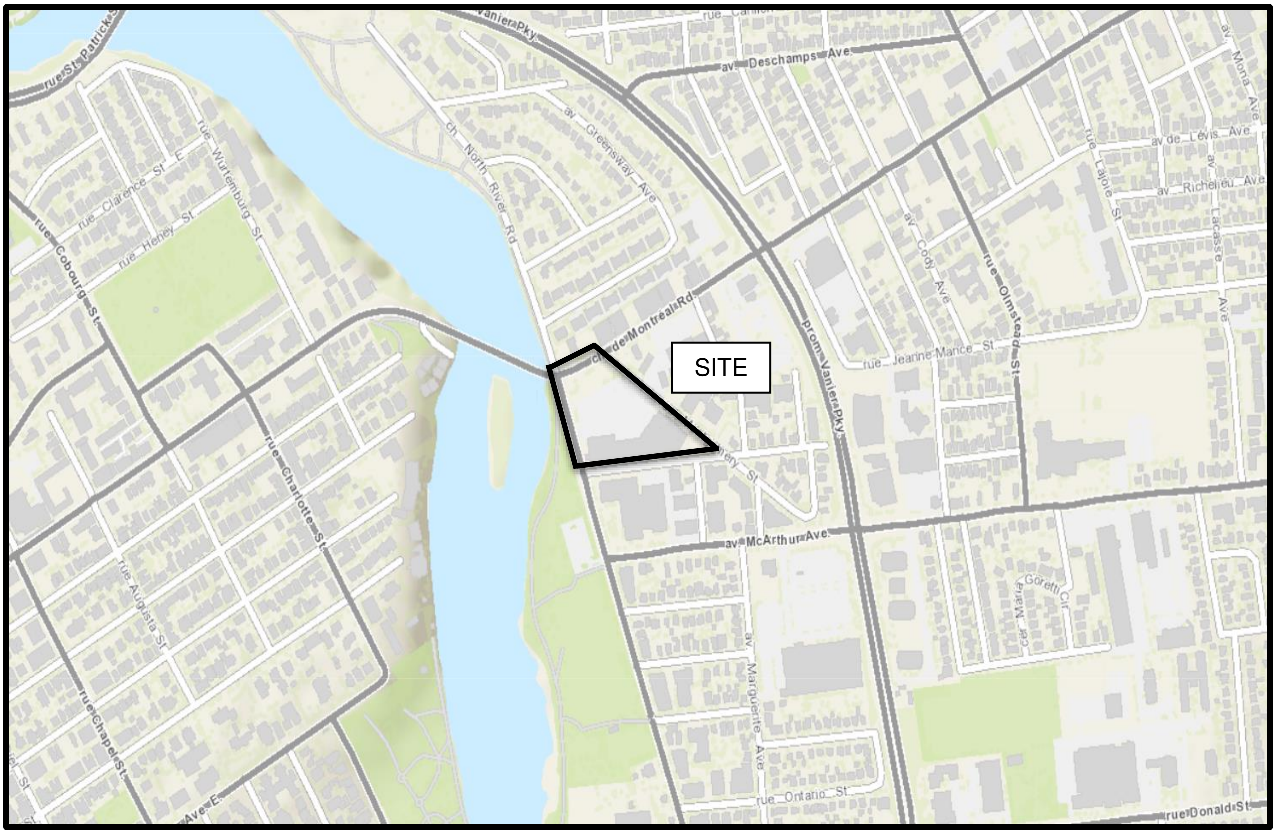


FIGURE 1

KEY PLAN

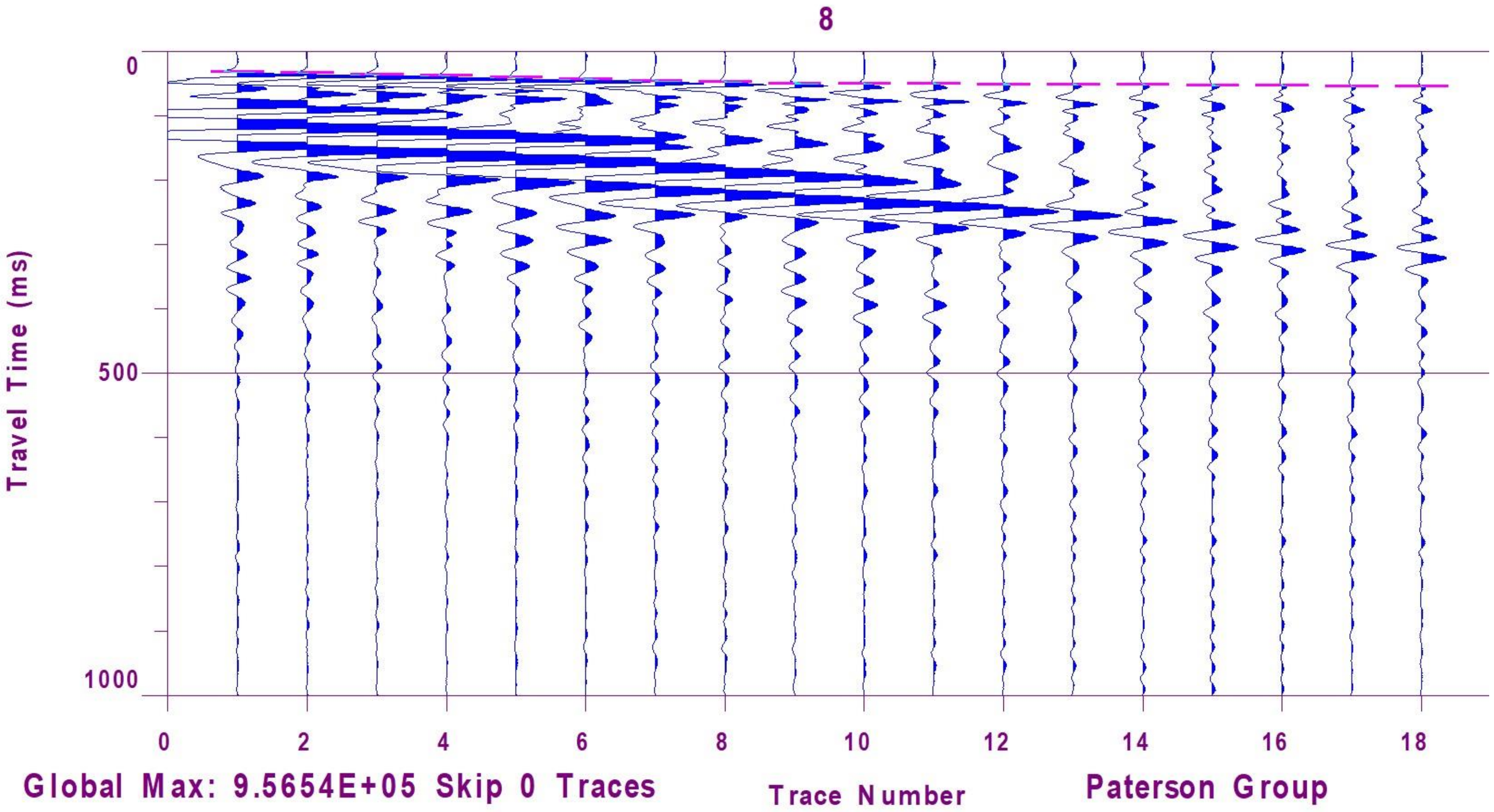


Figure 3 – Shear Wave Velocity Profile at Shot Location -15 m

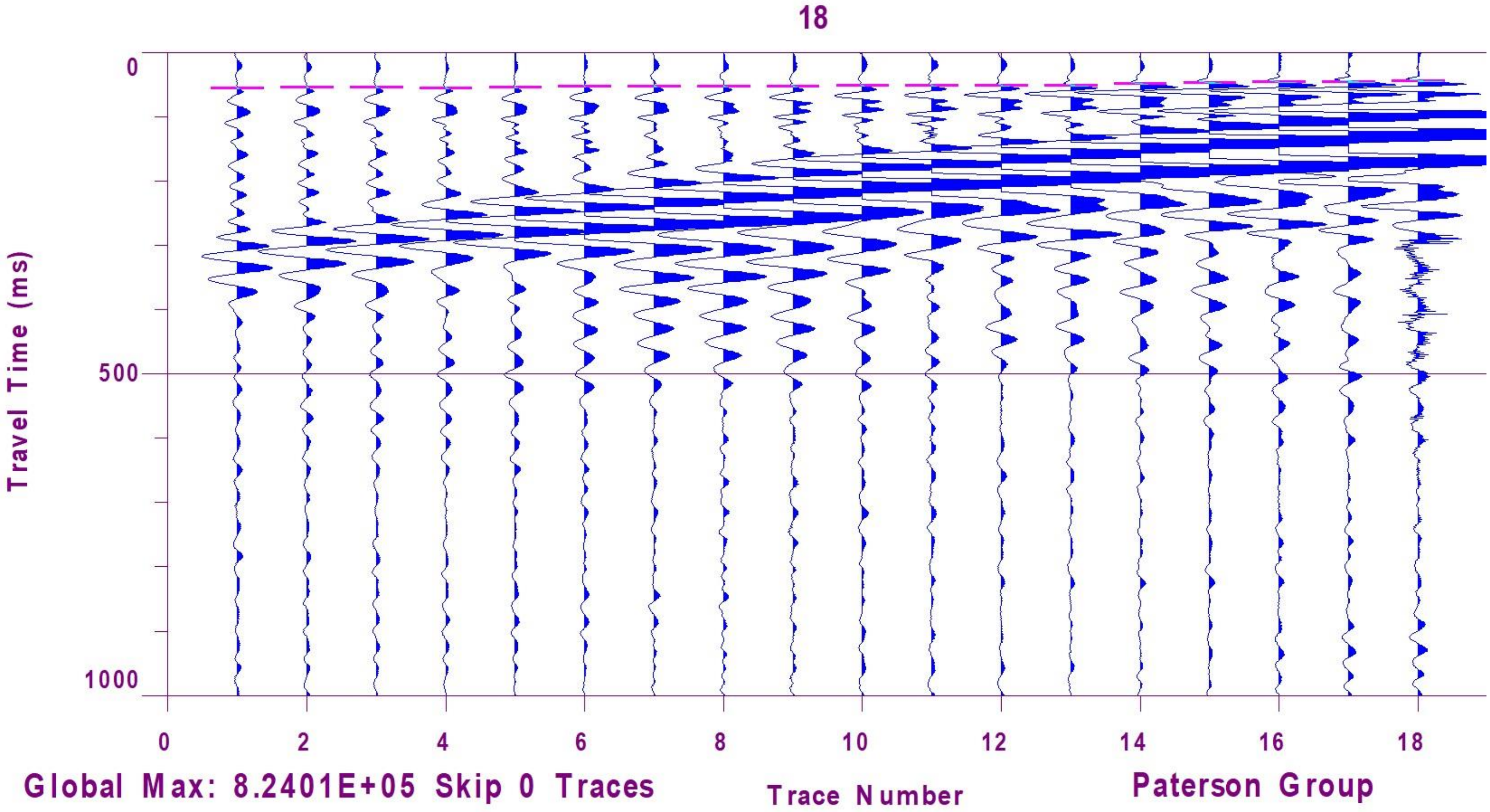
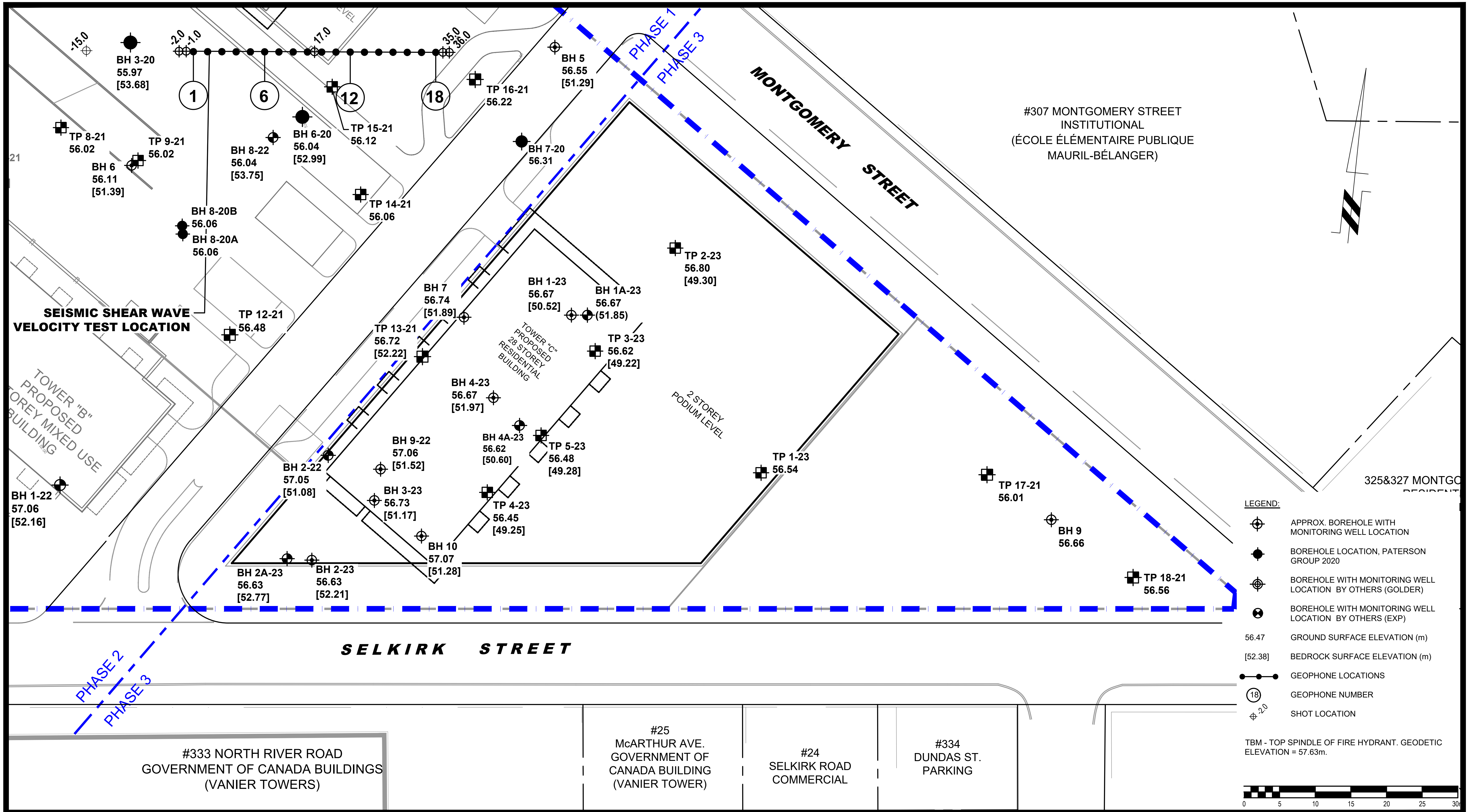


Figure 4 – Shear Wave Velocity Profile at Shot Location +49 m



PATERSON GROUP

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OTTAWA, ON
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NO.	REVISIONS	DATE	INITIAL

MAIN AND MAIN DEVELOPMENTS

GEOTECHNICAL INVESTIGATION - PROPOSED HIGH-RISE BUILDING - PHASE 3
3-33 SELKIRK STREET AND 2 MONTREAL ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500

Date: 07/2023

Drawn by: RCG

Report No.: PG4915-1

Checked by: DP

Approved by: DJG

PG4915-1

Revision No.: 0

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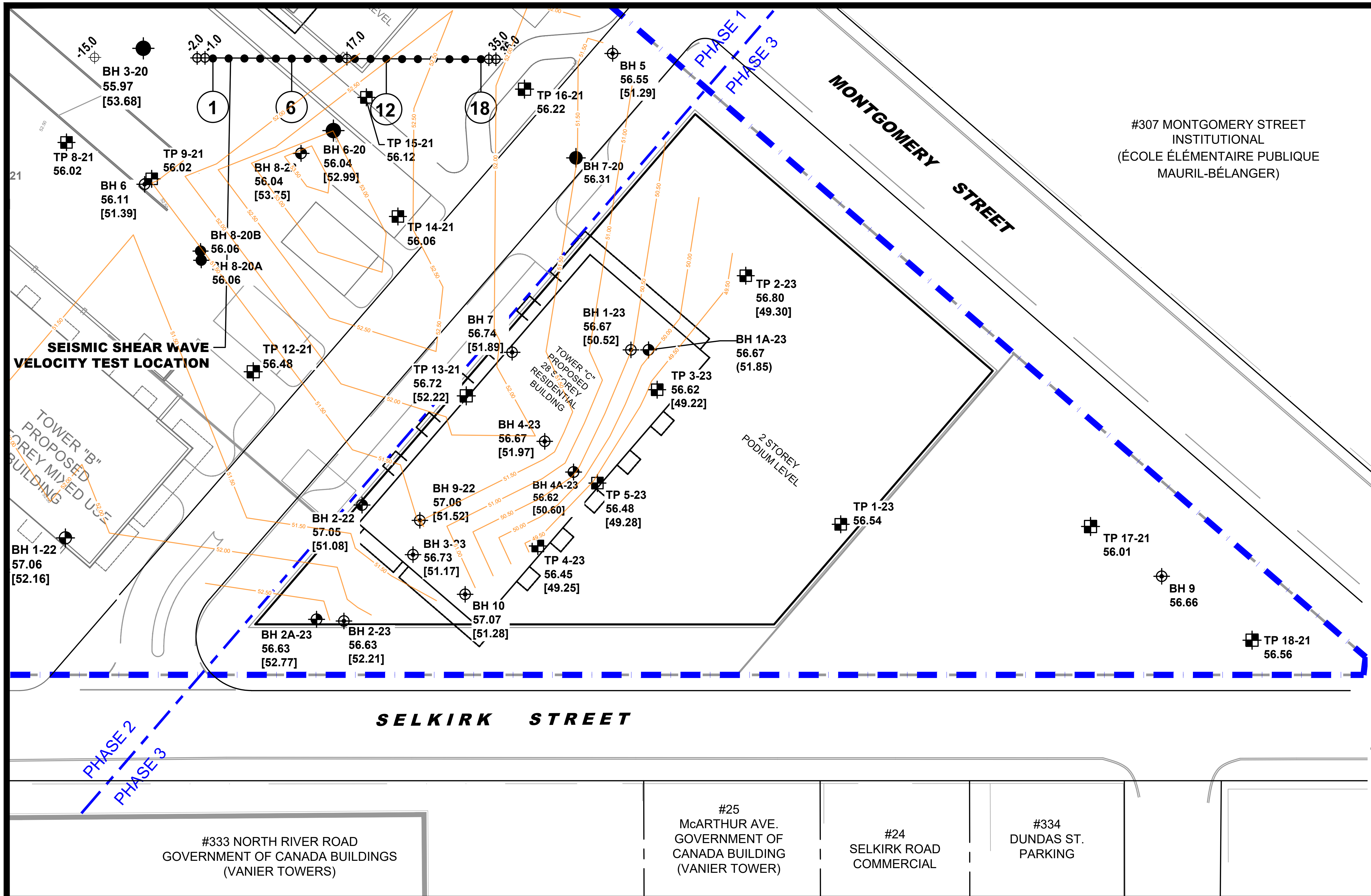

LEGEND:

⌖	APPROX. BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION, PATERSON GROUP 2020
⊕	BOREHOLE WITH MONITORING WELL LOCATION BY OTHERS (GOLDER)
⊗	BOREHOLE WITH MONITORING WELL LOCATION BY OTHERS (EXP)
56.47	GROUND SURFACE ELEVATION (m)
[52.38]	BEDROCK SURFACE ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
(18)	GEOPHONE NUMBER
⌖ 2.0	SHOT LOCATION
— 54.00 —	BEDROCK CONTOUR(m)

TBM - TOP SPINDLE OF FIRE HYDRANT. GEODETIC ELEVATION = 57.63m.

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3-33 SELKIRK STREET AND 2 MONTREAL ROAD

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:500
Date: 07/2023
Drawn by: RCG
Report No.: PG4915-1
Checked by: DP
PG4915-2
Approved by: DJG
Revision No.: 0

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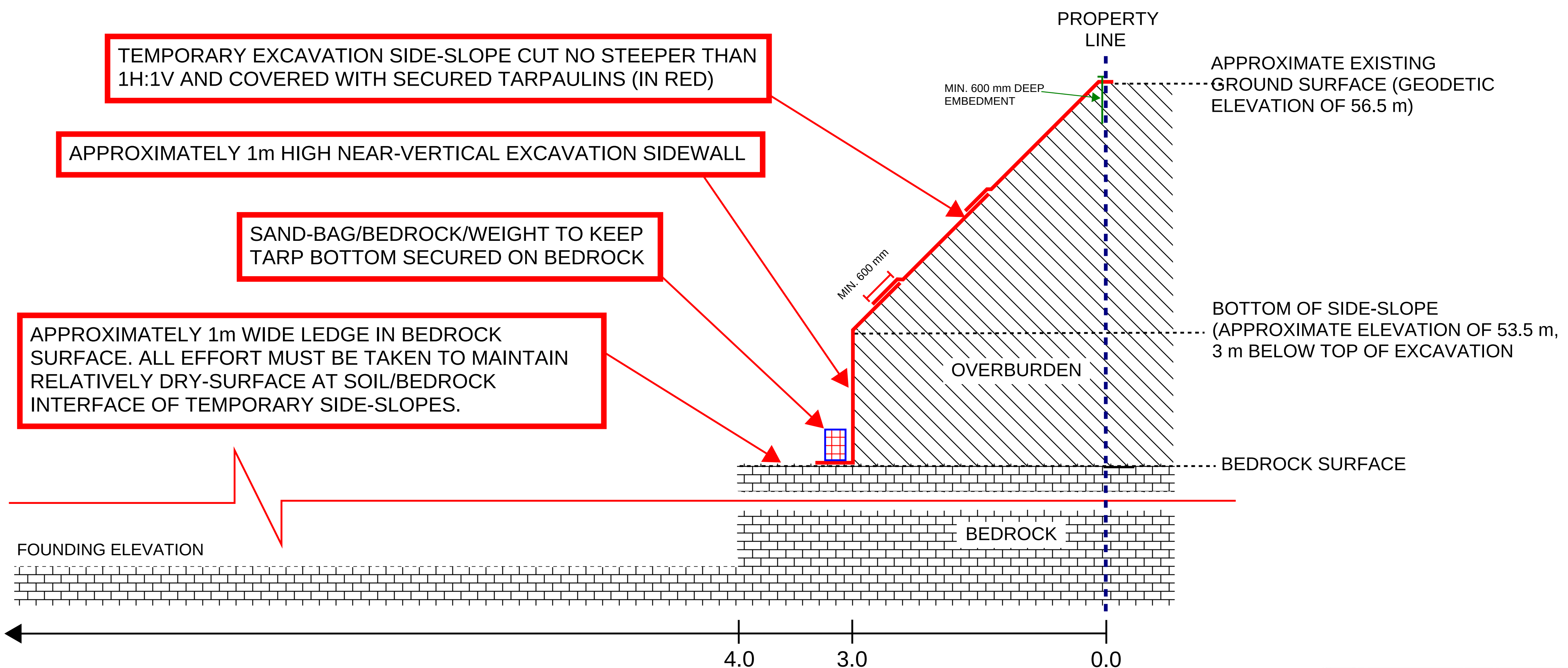
DETAIL A-A'



TEMPORARY EXCAVATION SIDE-SLOPE CUT NO STEEPER THAN 1H:1V AND COVERED WITH SECURED TARPAULINS (IN RED)

APPROXIMATELY 1m HIGH NEAR-VERTICAL EXCAVATION SIDEWALL

SAND-BAG/BEDROCK/WEIGHT TO KEEP TARP BOTTOM SECURED ON BEDROCK

APPROXIMATELY 1m WIDE LEDGE IN BEDROCK SURFACE. ALL EFFORT MUST BE TAKEN TO MAINTAIN RELATIVELY DRY-SURFACE AT SOIL/BEDROCK INTERFACE OF TEMPORARY SIDE-SLOPES.

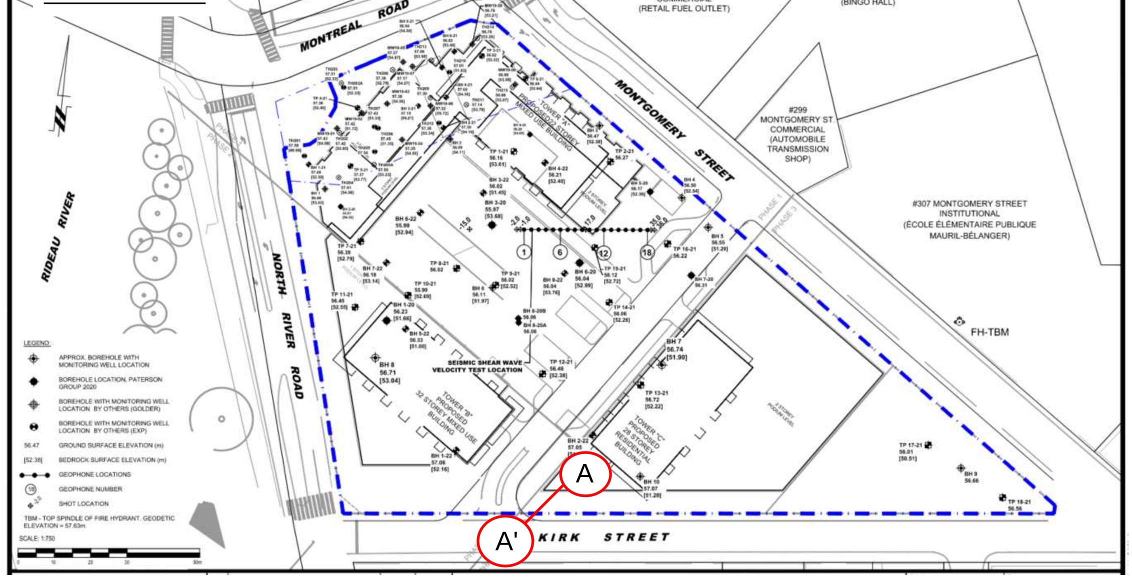
APPROXIMATE EXISTING GROUND SURFACE (GEODETIC ELEVATION OF 56.5 m)

BOTTOM OF SIDE-SLOPE (APPROXIMATE ELEVATION OF 53.5 m, 3 m BELOW TOP OF EXCAVATION

NOTES:

- EXCAVATION SIDE-SLOPES IN OVERBURDEN EXTENDING TO A MAXIMUM HEIGHT OF 3 m MAY BE CUT BACK TO A MAXIMUM STEEPNESS OF 1H:1V OR FLATTER.
- UP TO 1 m OF TEMPORARY SIDE-SLOPE MAY BE CUT AT NEAR VERTICAL FACE AT THE BOTTOM OF THE SLOPE.
- ENTIRE FACE OF TEMPORARY EXCAVATION SIDE-SLOPES SHOULD BE PROTECTED FROM EROSION BY THE USE OF SECURED TARPAULINS. TARPAULINS TO BE SECURED INTO PLACE BY EMBEDDING STEEL DOWELS OR WOODEN STAKES A MINIMUM OF 600mm BELOW THE GROUND SURFACE ON 2 m CENTRE TO CENTRE SPACING ALONG THE LENGTH OF THE TARPED SIDE-SLOPE. SECURING THE TOP OF TARPS IN PLACE BY THE USE OF WEIGHT BY ITEMS SUCH AS SAND BAGS, FRACTURED ROCK, ETC., WILL NOT BE CONSIDERED ACCEPTABLE AT THE TIME OF CONSTRUCTION. WEIGHT MAY BE USED TO SECURE BOTOM OF TARPS. EROSION CONTROL BLANKET PRODUCTS ARE NOT CONSIDERED SUITABLE FOR USE AS TARPAULINS.
- TARPAULINS TO BE OVERLAPPED BY A MINIMUM OF 600mm BETWEEN ABUTTING SHEETS,TIED TOGETHER AT OVERLAPS, SHINGLED TO PROMOTE DOWNWARD SHEET-DRAINAGE OF WATER AND AS DEPICTED HEREIN.
- EXCAVATED SOIL SHOULD NOT BE STOCKPILED DIRECTLY AT THE TOP OF EXCAVATIONS AND HEAVY EQUIPMENT SHOULD BE KEPT AWAY FROM THE TOP OF EXCAVATION SIDE-SLOPES.
- IMPLEMENTATION OF THE ABOVE-NOTED DETAIL SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION AND PERIODICALLY THEREAFTER FOR ON-GOING STABILITY AND EROSION PROTECTION PURPOSES BY PATERSON PERSONNEL.

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL
0			

MAIN AND MAIN DEVELOPMENTS INC.
RECOMMENDATIONS FOR SLOPE OF TEMPORARY EXCAVATIONS
3-33 SELKIRK STREET AND 2 MONTREAL ROAD

OTTAWA, ONTARIO

Title:
CROSS SECTION

Scale: AS SHOWN
Date: 07/2023
Drawn by: FC
Report No.: PG4915-2
Checked by: DP
Dwg. No.: PG4915-3
Approved by: DG
Revision No.: 0