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Geotechnical Investigation
Proposed Multi-Storey Building
180 Metcalfe Street
Ottawa, Ontario

Prepared For

Jadco Group

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Report PG4491-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Jadco Group to conduct a geotechnical investigation for the proposed multi-storey building, which is to be located at 180 Metcalfe Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- Determine the subsoil, groundwater, and bedrock conditions at this site by means of boreholes.

- Based on the results of the boreholes, provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under a separate cover.

2.0 Proposed Development

It is our understanding that the proposed development will consist of a 27 storey building with up to 6 levels of underground parking. The footprint of the proposed parking garage will occupy the entire site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out between April 3 to 5, 2018. At that time, 5 boreholes were advanced to a maximum depth of 21 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The approximate locations of the boreholes are shown on Drawing PG4491-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

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

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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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

Diamond drilling was carried out at BH1-18, BH2-18 and BH3-18, to determine the nature of the bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. 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.

Groundwater

A 32 or 51 mm diameter PVC groundwater monitoring well was installed in BH1-18, BH2-18, and BH3-18 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located on the south side of Nepean Street across from 91 Nepean Street. A Geodetic elevation of 71.82 m was previously provided for this TBM by Annis O'Sullivan Vollebekk Ltd. during a previous investigation for an adjacent site.

The locations of the boreholes, TBM, and the ground surface elevation at the boreholes, are presented on Drawing PG4491-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were 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 a six-storey building with an asphalt finished car parking area. The site is bordered to the south by a multi-storey residential building, to the west by a low-rise residential building, to the east by Metcalfe street and to the north by Nepean street. It should be noted that the adjacent buildings are in close proximity to the site boundaries. The ground surface across the site is relatively flat and approximately at grade with Metcalfe street and Nepean street.

4.2 Subsurface Profile

Overburden

Generally, the soil conditions encountered at the boreholes consist of a pavement structure at ground surface overlying a fill layer consisting of brown silty sand with trace gravel and/or clay. A stiff silty clay deposit was encountered below the fill layer. Glacial till, consisting of silty clay with gravel, cobbles and boulders, was encountered below the silty clay deposit. Practical refusal to augering was encountered at depths between 12.7 and 15.3 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the boreholes.

Bedrock

A poor (RQD values ranging between 25 to 50%) to an excellent (RQD values ranging between 90 to 100%) quality black shale bedrock was encountered at BH1-18, BH 2-18 and BH 3-18 at depths ranging between 12.9 and 15.3 m.

Based on available geological mapping, the bedrock in this area consists of shale of the Billings formation and dark grey limestone.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the monitoring wells installed at the borehole locations and the results are summarized in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. It should be further noted that the long term groundwater level may be at a higher elevation than encountered at the time of the geotechnical investigation which is being influenced by nearby construction activities which have temporarily lowered the groundwater level. Therefore, the groundwater levels could vary at the time of construction and in the long term.

Table 1
Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-18	71.33	13.76	57.57	April 23, 2018
BH 2-18	71.32	14.64	56.68	April 23, 2018
BH 3-18	71.42	14.04	57.38	April 23, 2018

Note: The ground surface elevation at each borehole was referenced to a temporary benchmark (TBM), which consists of the top spindle of the fire hydrant located on the south side of Nepean Street across from 91 Nepean Street . A Geodetic elevation of 71.82 m was previously provided for this TBM by Annis O'Sullivan Vollebekk Ltd during a previous investigation.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed hi-rise development. It is expected that temporary shoring will be required for the excavation of the underground parking levels. It is further expected that the influence of the excavation and the selection of the shoring system should account for the effects to adjacent structures including dewatering control measures.

Bedrock excavation is expected for the construction of the lower underground levels of the proposed building. A combination of line drilling, blasting, mechanical bedrock removal and rock grinding operations are expected for the removal of the bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the expected founding level for the proposed building to accommodate up to 6 levels of underground parking, it is anticipated that all overburden material will be excavated from the subject site. Bedrock excavation will be required for the construction of the lower levels of the underground parking garage.

Bedrock Removal

Based on the volume of the bedrock encountered in the area, it is expected that line-drilling in conjunction with controlled blasting and mechanical bedrock removal (hoe-ramming and rock grinding) will be required to remove the bedrock.

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

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s during the blasting program to reduce the risks of damage to the existing structures. Typically, it's suggested that the peak particle velocity be maintained at 25 mm/s at the property line when possible.

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

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system. If the shoring system will include drilled piles into the bedrock extending to 1 to 1.5 m below the proposed excavation bottom, the 1 m horizontal ledge can be omitted.

Vibration Considerations

Construction operations are also the cause of 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, as much as possible, a cooperative environment with the residents.

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Stabilization of Bedrock Excavation Sidewalls

Horizontal rock anchors will likely be required around the perimeter of the excavation sidewalls to prevent pop-outs and slippage of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors or rock bolts will be evaluated during the excavation operations.

Depending on the underside of footing elevation with respect to the top of the bedrock surface, it is further expected that a chain link fence with a geotextile layer against the bedrock (or approved alternative method) may be required to be installed over the vertical bedrock surface to retain small bedrock pop-outs. This requirement can also be evaluated during the excavation operations.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, it is expected that shale bedrock and black limestone will be encountered at the founding level. If a partial tanking system is considered to reduce long term dewatering of adjacent sites, conventional spread footings founded on the sound bedrock can be used.

A factored bearing resistance value at ULS of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5 could be used if founded on the black limestone or very sound shale bedrock where the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within selected major footings and shear walls. At least one drill hole should be completed per major footing and shear wall. The drill hole inspection should be carried out by the geotechnical consultant. 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. As an alternative, if sufficient bedrock vertical surface is exposed, the bedrock quality can be evaluate and avoid having to drill probeholes.

Settlement

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 the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. Two seismic shear wave velocity profiles from the testing are presented in Appendix 2.

Field Program

The shear wave testing location is presented in Drawing PG4491-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in an approximately east-west orientation. The 4.5 Hz horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was 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 5 to 10 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, 3, 4.5 and 10 m away from the first geophone and 3, 4.5 and 12 m from the last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Given the depth of the bedrock encountered in the test holes at the site, it is anticipated that the proposed building will be founded directly on the bedrock. Based on our testing results, the bedrock shear wave velocity is **2,103 m/s**.

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

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

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}103m/s}\right)}$$

$$V_{s30} = 2{,}103m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for the proposed building is 2,103 m/s provided the footings are placed directly on the bedrock surface. Therefore, a **Site Class A** 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

It is expected that most of the basement area will be used for parking and possible storage lockers. Since the foundation will consist of a conventional spread footing foundation, a minimum 150 mm layer of OPSS Granular A crushed stone material or 19 mm clear stone (if better drainage is required) will be used over the footings. The final floor slab will be poured over the compacted OPSS Granular A or 19 mm clear stone.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, 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 drained unit weight of 20 kN/m^3.

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

Static Earth Pressures

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

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 a_c \gamma H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.
The total earth pressure (P_{AE}) is considered to act at a height, h, (m) from the base of the wall. Where:

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

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

5.7 Rock Anchor Design

If required in the structural design, rock anchors can be designed using the following geotechnical parameters:

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. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

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) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 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, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 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	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

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm diameter hole are provided in Table 3. A detailed analysis for the anchorage system could be provided once the details of the loading for the proposed tower are known. It should be noted that the factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects.

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	3.2	1.1	4.3	250
	3.8	2.2	5.8	450
	4.1	2.6	6.7	600
	5	2.5	7.5	750

Other considerations

The anchor drill holes should 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 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.

6.0 Design and Construction precautions

6.1 Foundation Drainage and Backfill

Foundation Waterproofing

It is understood that the building footprint will occupy the entire boundary of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a drainage system placed on the bedrock excavation face and against the temporary shoring system.

Since the lower basement levels will be located below the expected groundwater level, consideration should be given to installing a waterproofing membrane or coating for the vertical surfaces from the bottom of the excavation (bedrock vertical face) up to 1 m above the long term groundwater level (approximately 10 m below the existing finished grade). The bedrock vertical surface will require bedrock grinding to create a smoother bedrock surface and lessen the potential of bedrock over breakage. By waterproofing the vertical excavation sides, it will be possible to lessen the groundwater volumes entering the excavation. A composite drainage system should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water breaching the waterproofing membrane system.

For preliminary design purposes, the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) should extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration that breaches the concrete mud slab. For design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at 6 to 9 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation program for a better assessment.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard. Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate 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 Temporary Shoring

Temporary shoring may be required for the overburden soil 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.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system will consist of a combination of soldier pile and lagging system for open areas such as roadways and parking lots and interlocking steel sheet piling for areas adjacent or in close proximity to existing structures. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 5 - Soil Parameters

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

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

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

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

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential 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

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

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) Category 3 may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater breaching the waterproofing system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (less than 50,000 L/day) with higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, the long term groundwater level is expected at a depth ranging from 11 to 12 m below the existing grade and within the glacial till deposit. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site as a result of the proposed shoring system and waterproofing required adjacent to any neighbouring structures. The installation of a temporary shoring system will disturb the soil immediately behind the shoring system which may cause some movement of adjacent structures. To lessen these effects, consideration should be given to adding brackets to the shoring system that could support the adjacent structures. Furthermore, until the waterproofing is completed, temporary dewatering will also cause typical minor differential settlements to unsupported structures.

6.6 Winter Construction

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

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

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

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

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

6.7 Protection of Exposed Bedrock Surfaces

Shale bedrock of the Billings Formation is susceptible to deterioration upon exposure to oxygen. As such, it recommended that consideration be given to protecting exposed bedrock surfaces with a thin concrete coating to retard the deterioration process. A concrete mud slabs can be used for horizontal surfaces and waterproofing coatings can be used for vertical surfaces.

Where bearing surfaces and the bases of excavations are wet and/or will be quickly covered with concrete, the coating is not required.

Although the bedrock does contain pyrite, this development is not considered to be susceptible to the shale expansion process, as there will not be significant amounts of heat applied to the bedrock. If sub-basement floor heating systems or other sources of heat, such as transformers are planned to be located close to the bedrock, special measures will be required to ensure that pyritic expansion will be prevented.

7.0 Recommendations

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

- ❏ Review the bedrock stabilization and excavation requirements.
- ❏ Review waterproofing system for foundation walls.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations made in this report are in nature and in accordance with our present understanding of the project. A detailed investigation should be carried out to validate the recommendations presented in this report. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

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

The 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 Jadco Group or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Colin Belcourt

Colin Belcourt, P.Eng.

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

Report Distribution

- ❑ Jadco Group (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
180 Metcalfe Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

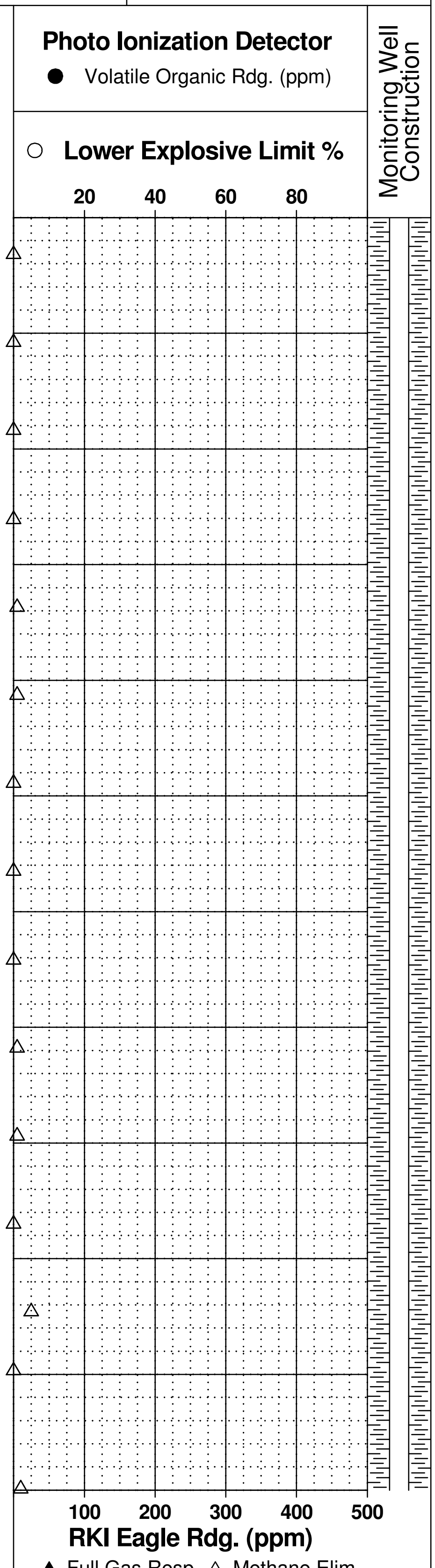
REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** April 3, 2018

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with silt and sand	0.30	AU	1		
FILL: Brown silty sand - trace gravel and clay by 1.45m depth		SS	2	42	9
		SS	3	58	4
	3.30	SS	4	8	2
Very stiff to soft, grey **SILTY CLAY,** trace sand		SS	5	58	27
		SS	6	100	W
		SS	7	100	W
		SS	8	100	W
		SS	9	100	W
	7.54	SS	10	100	W
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders		SS	11	100	W
		SS	12	75	9
		SS	13	58	1
		SS	14	33	50+

DEPTH (m)	ELEV. (m)
0	99.51
1	98.51
2	97.51
3	96.51
4	95.51
5	94.51
6	93.51
7	92.51
8	91.51
9	90.51
10	89.51
11	88.51



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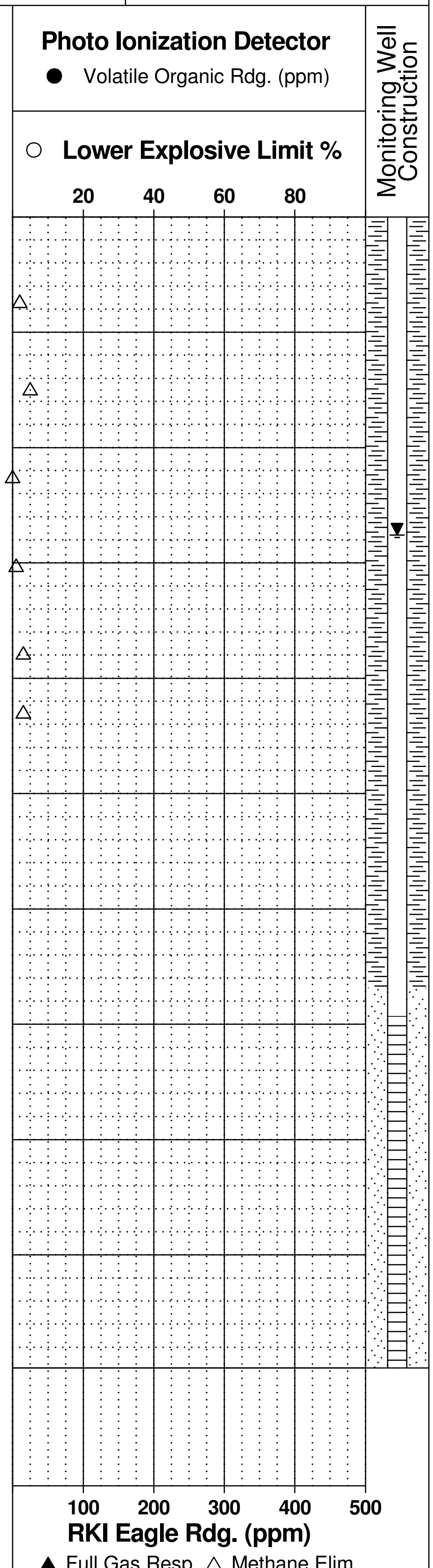
FILE NO. PE4280

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** April 3, 2018

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders	SS	15	42	3	11	88.51
	SS	16	25	17	12	87.51
	SS	17	62	21	13	86.51
	SS	18	75	13		
	SS	19	42	16	14	85.51
14.40						
Inferred weathered **BEDROCK**	SS	20	83	22	15	84.51
15.34	SS	21	93	50+		
BEDROCK: Black shale	RC	1	100	38	16	83.51
	RC	2	97	88	17	82.51
	RC	3	100	100	18	81.51
	RC	4	100	94	19	80.51
	RC	5	95	95	20	79.51
20.98						
End of Borehole						
(GWL @ 13.76m - April 23, 2018)						



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
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Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** April 4, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.50
Asphaltic concrete 0.05							
FILL: Crushed stone with silt and sand 0.23		AU	1				
FILL: Brown silty sand, trace gravel 1.07		SS	2	100	12	1	98.50
Compact, brown **SILTY SAND** 1.45							
Stiff, brown to grey **SILTY CLAY,** trace sand		SS	3	100	10	2	97.50
		SS	4	100	5		
		SS	5	100	P	3	96.50
		SS	6	100	P	4	95.50
		SS	7	100	P	5	94.50
						6	93.50
		SS	8	100	P		
						7	92.50
8.30		SS	9	54	P	8	91.50
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders		SS	10	58	P		
						9	90.50
		SS	11	42	50		
		SS	12	67	12	10	89.50
						11	88.50

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
180 Metcalfe Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE April 4, 2018

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
	SS	13	67	14	11	88.50
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders	SS	14	46	4		
					12	87.50
	SS	15	42	8		
12.90						
	RC	1	100	50	13	86.50
	RC	2	93	32	14	85.50
BEDROCK: Black shale					15	84.50
	RC	3	100	76		
					16	83.50
	RC	4	98	76	17	82.50
17.68						
End of Borehole						
(GWL @ 14.64m - April 23, 2018)						

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**
20 40 60 80

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

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DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

REMARKS

HOLE NO. BH 3

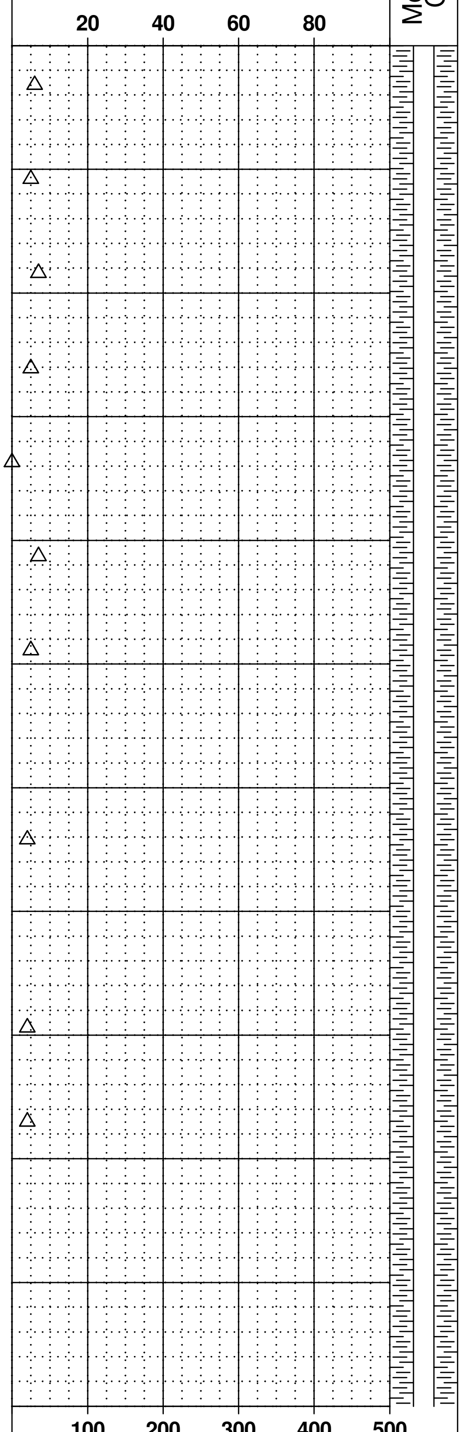
BORINGS BY CME 55 Power Auger **DATE** April 4, 2018

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
25mm Asphaltic concrete over crushed stone with silt and sand	0.25	AU	1		
FILL: Brown silty sand, trace gravel and organics	0.79				
Compact, brown **SILTY SAND**	1.65	SS	2	100	10
Stiff, brown to grey **SILTY CLAY,** trace sand	7.39	SS	3	75	10
		SS	4	100	5
		SS	5	0	P
		SS	6	100	P
		SS	7	100	P
		SS	8	100	P
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders		SS	9	62	6
		SS	10	58	2
		SS	11	50	7
		SS	12	0	18

DEPTH (m)	ELEV. (m)
0	99.60
1	98.60
2	97.60
3	96.60
4	95.60
5	94.60
6	93.60
7	92.60
8	91.60
9	90.60
10	89.60
11	88.60

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**



RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
180 Metcalfe Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2018

FILE NO. PE4280

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders		SS	13	67	8	11	88.60
						12	87.60
						13	86.60
13.89							
BEDROCK: Black shale		RC	1	100	84	14	85.60
		RC	2	100	81	15	84.60
						16	83.60
		RC	3	100	92	17	82.60
17.68							
End of Borehole							
(GWL @ 14.04m - April 23, 2018)							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

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DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger **DATE** April 5, 2018

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with silt and sand	0.20	AU	1		
	0.69				
FILL: Brown silty sand, trace gravel and organics		SS	2	83	27
Compact, brown **SILTY SAND**	1.62				
Stiff, brown to grey **SILTY CLAY,** trace sand	6.63	SS	3	100	12
		SS	4	100	5
		SS	5	50	P
		SS	6	100	P
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders		SS	7	42	P
		SS	8	83	2
		SS	9	67	8
		SS	10	58	13
		SS	11	71	19

DEPTH (m)	ELEV. (m)
0	99.39
1	98.39
2	97.39
3	96.39
4	95.39
5	94.39
6	93.39
7	92.39
8	91.39
9	90.39
10	89.39
11	88.39

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

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FILE NO. PE4280

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger **DATE** April 5, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE									
		SS	12	83	40	11	88.39		
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders		SS	13	79	34			△	
						12	87.39		
12.70		SS	14	84	34			△	
End of Borehole Practical refusal to augering at 12.70m depth									

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
180 Metcalfe Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

FILE NO. PE4280

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger **DATE** April 5, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	99.64		
						1	98.64		
						2	97.64		
						3	96.64		
						4	95.64		
						5	94.64		
OVERBURDEN									
						6	93.64		
						7	92.64		
						8	91.64		
						9	90.64		
						10	89.64		
						11	88.64		

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
180 Metcalfe Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant . Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 5, 2018

FILE NO. PE4280

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) ○ Lower Explosive Limit % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE									
						11	88.64		
						12	87.64		
OVERBURDEN									
						13	86.64		
14.02									
Inferred weathered **BEDROCK**						14	85.64		
14.43		SS	1	100	50+				
End of Borehole									

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

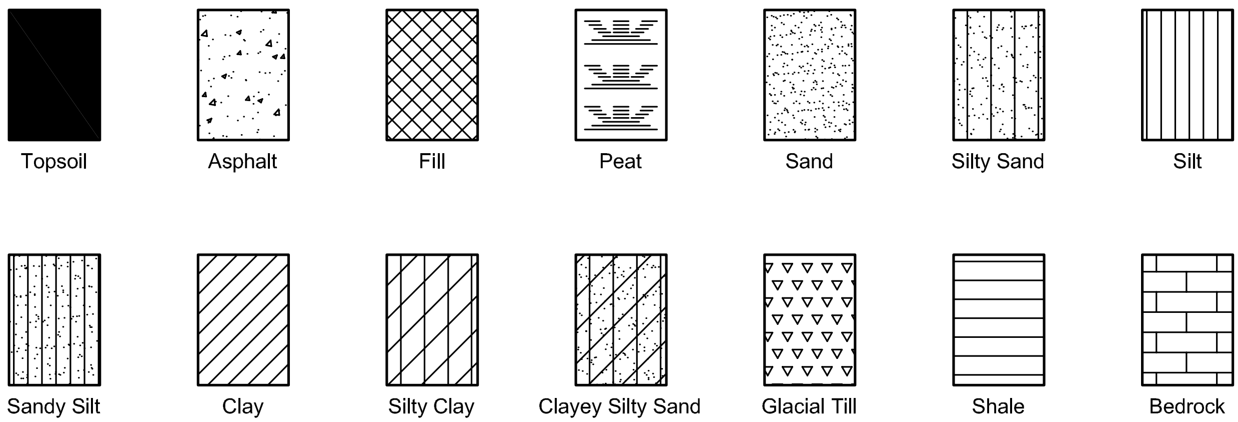
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

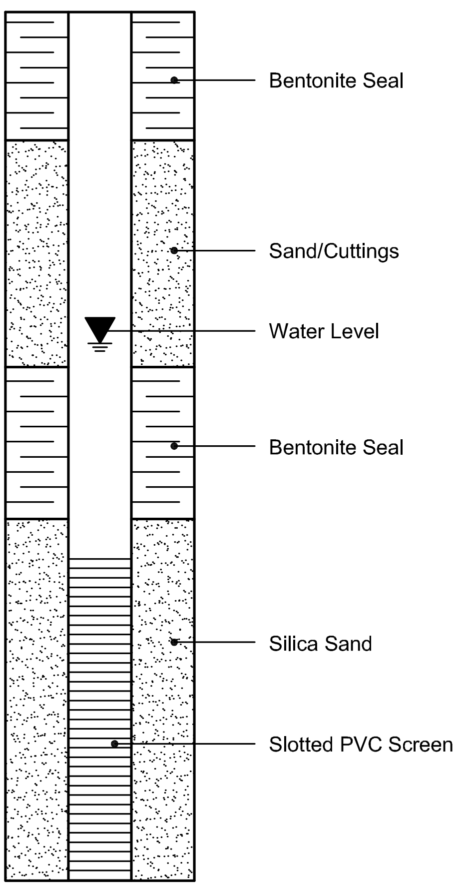
SYMBOLS AND TERMS (continued)

STRATA PLOT

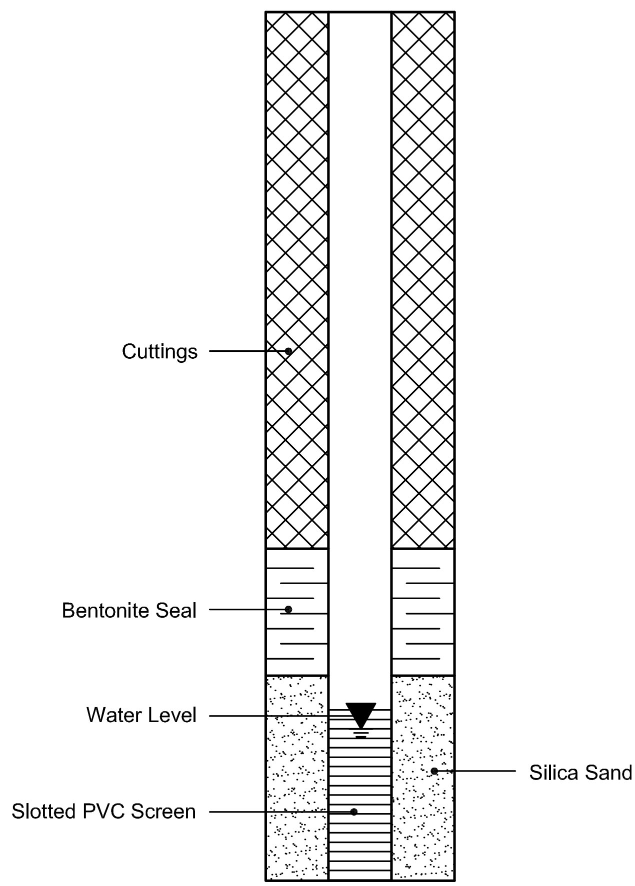


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

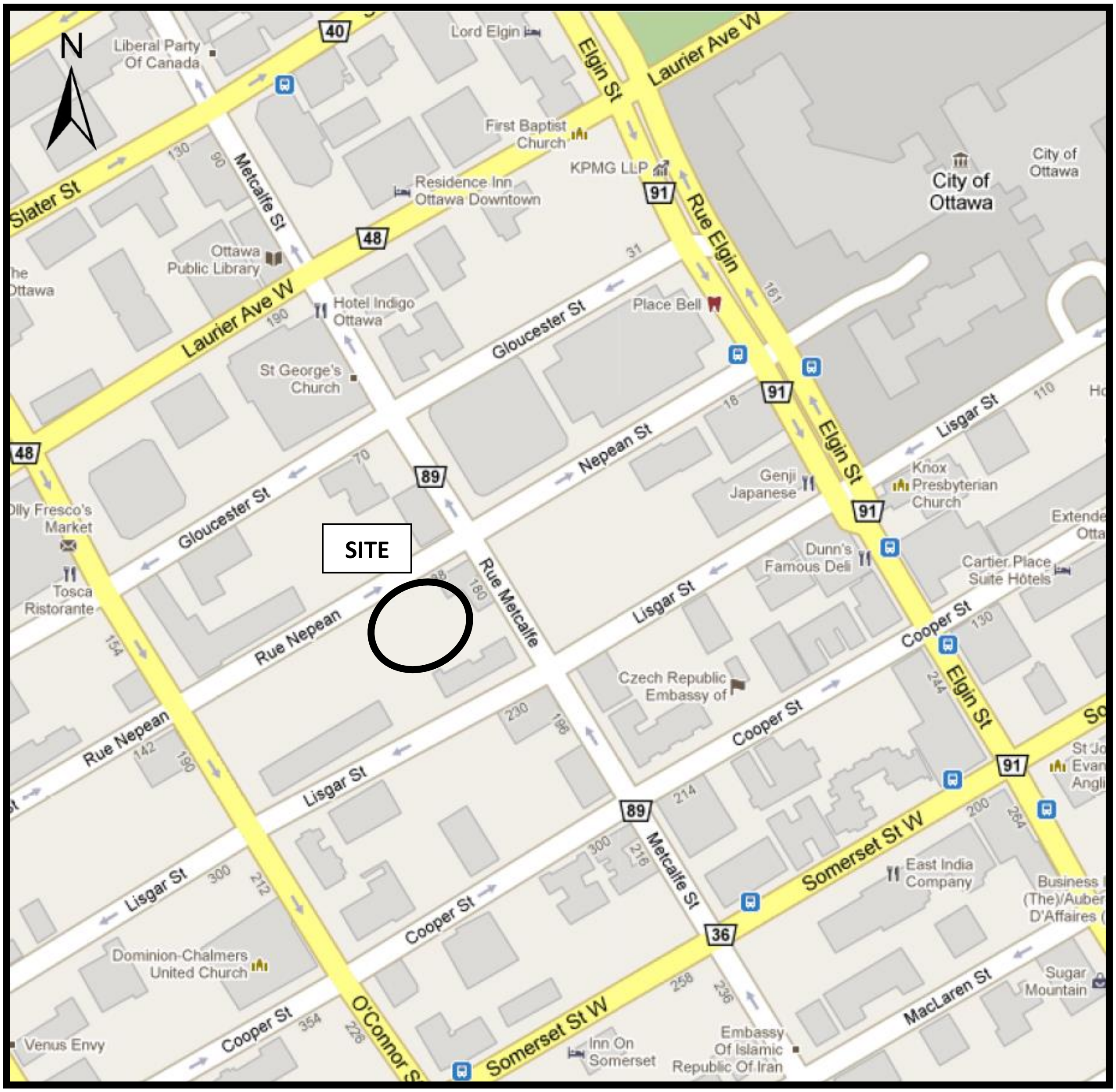


APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 AND FIGURE 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4491-1 - TEST HOLE LOCATION PLAN



<u>FIGURE 1</u>

KEY PLAN

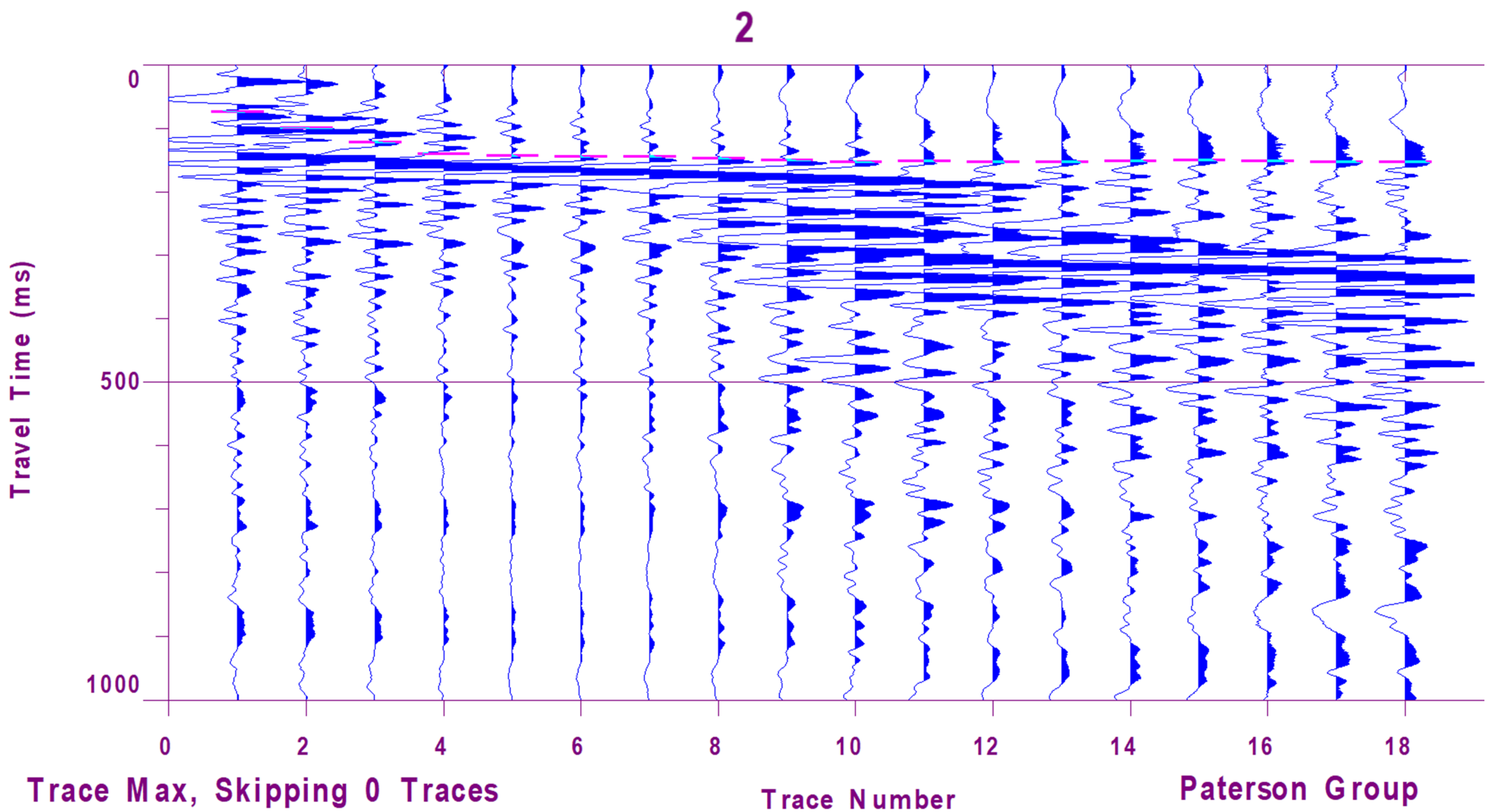


Figure 2 – Shear Wave Velocity Profile at Shot Location -10 m

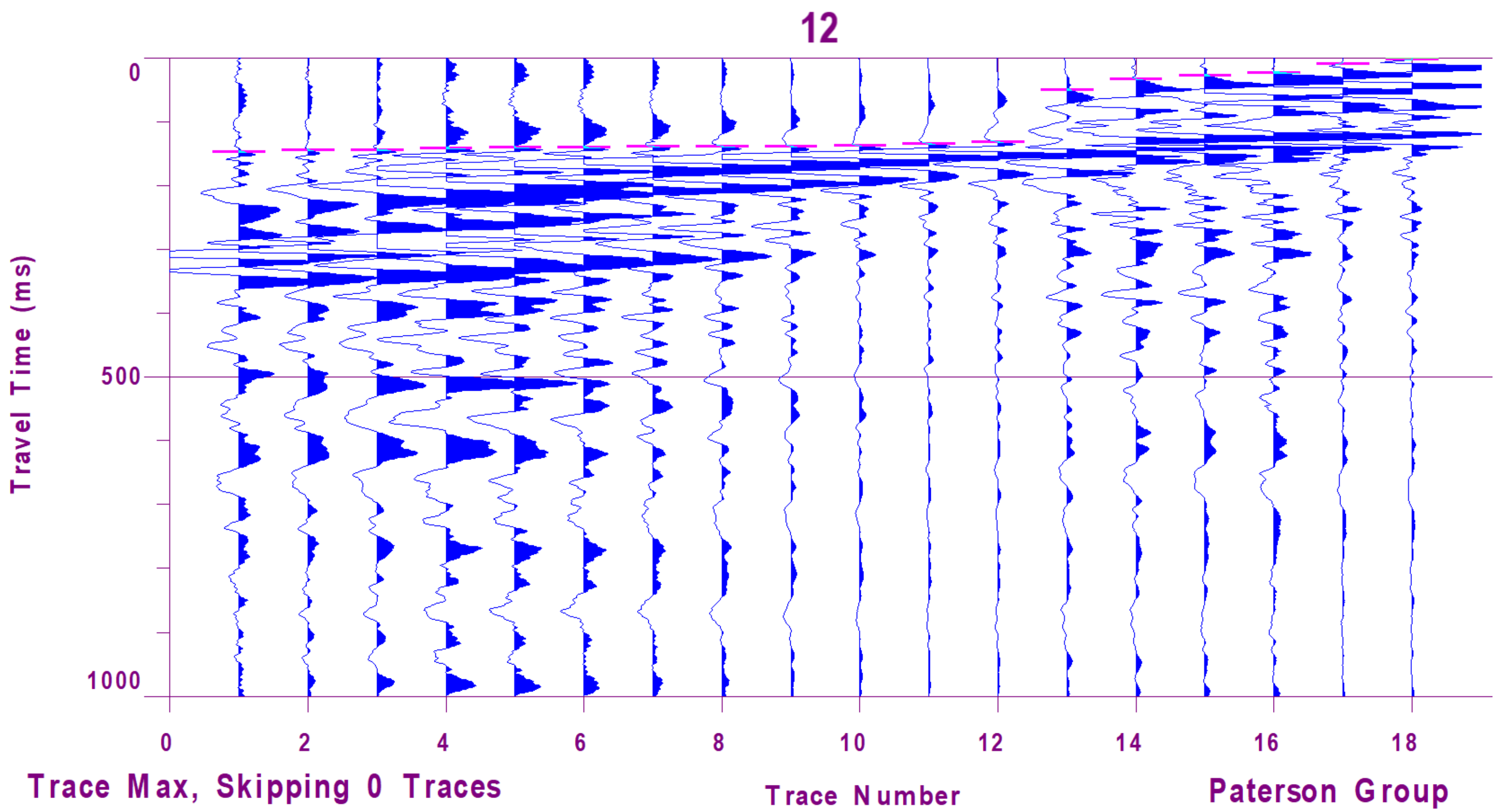
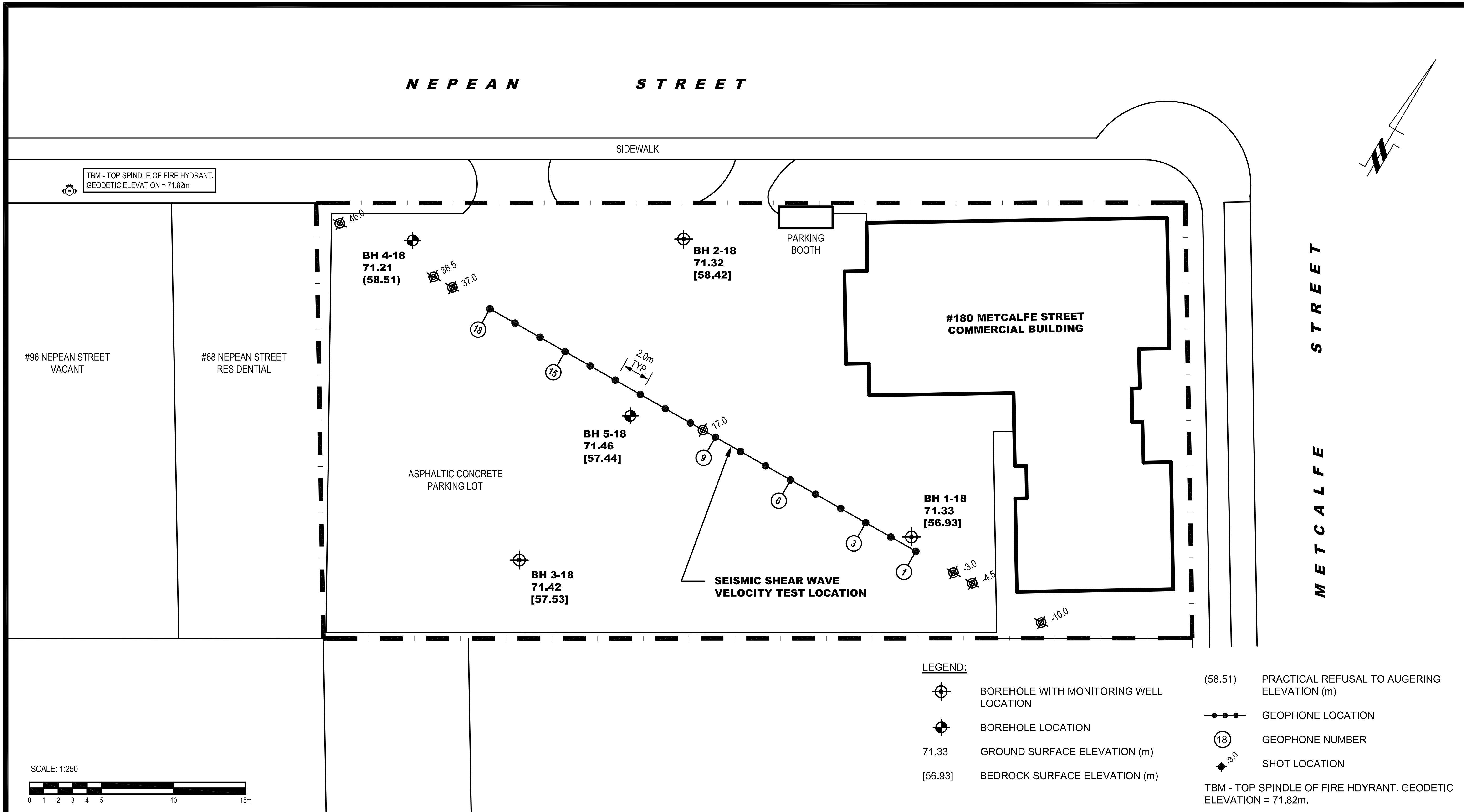


Figure 3 – Shear Wave Velocity Profile at Shot Location 37 m



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JADCO GROUP

GEOTECHNICAL INVESTIGATION
PROP. HIGH RISE DEVELOPMENT - 180 METCALFE STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 06/2018
Drawn by: MPG	Report No.: PG4491-1
Checked by: CB	Dwg. No.: **PG4491-1**
Approved by: DJG	Revision No.: 0

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