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Proposed Multi-Storey Building
929 Richmond Road and 108 Woodroffe Ave.
Ottawa, Ontario

Prepared For

Westboro Point Developments Inc.

December 19, 2017

Report PG1609-1 Revision 2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Westboro Point Developments Inc. to conduct a geotechnical investigation for a proposed multi-storey building which will be located at 929 Richmond Road and 108 Woodroffe Avenue 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 and groundwater conditions at this site by means of boreholes.

❏ To provide geotechnical recommendations to 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.

Paterson has completed an environmental site assessment for the subject site and the findings and recommendations are presented under separate cover.

2.0 Proposed Project

It is understood that the proposed development will consist of a 19 storey mixed-use building with 4 levels of underground parking. Associated parking areas, access lanes, and landscaped areas will be located around the proposed building.

The subject property is located at the northwest corner of the intersection of Woodroffe Avenue and Richmond Road. The site is bordered to the north by a utility easement followed by residential housing, to the west by a multi-storey apartment building, to the east by Woodroffe Avenue, and to the south by Richmond Road.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Field programs for the investigation were carried out on January 31 and Feburary 1, 2008 and March 13 and 16, 2015. A total of seven (7) boreholes were advanced to a maximum depth of 13.4 m. 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 PG1609 -1 - Test Hole Location Plan Revision 4 included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. 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.

Rock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to our laboratory. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) 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 (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Monitoring Well Installation

Groundwater monitoring wells were installed in five (5) of the boreholes (BH 2 and BH 4 through BH 7) upon completion of the sampling program. Typical monitoring well construction details are described below:

- ☐ 3 m of slotted 32 mm diameter PVC screen at the base of each borehole.
- ☐ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

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

Sample Storage

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The location of the boreholes and the ground surface elevations at the boreholes are presented on Drawing PG1609-1 Revision 3 - Test Hole Location Plan in Appendix 2.

The ground surface elevation at each borehole location was referenced to the top spindle of the fire hydrant located at the intersection of Woodroffe Avenue and Deschenes Street, north of the subject property. A geodetic elevation of 67.33 m was assigned to this temporary benchmark (TBM). The location of the TBM is presented on Drawing PG1609 -1 Revision 4 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and rock 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 site is currently occupied by a 2-storey commercial building (929 Richmond Road) and residential dwelling (108 Woodroffe Avenue). The remainder of the site is landscaped grass area, with trees and brush located along the north, east and west property lines. The site is also bordered to the north by an easement which contains a large diameter watermain.

4.2 Subsurface Profile

Overburden

The soil profile of the site consists of topsoil overlying brown silty sand with gravel and/or glacial till, which in turn is underlain by bedrock. Specific details of the soil profile at each test hole location can be seen on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Bedrock, consisting of limestone and dolostone with increasing shale content with depth was cored at each borehole location to a maximum depth of 8.1 m. The recovery values and RQD values for the bedrock cores were calculated. The recovery values range from 38 to 100%, while the RQD values vary between 16 and 100%. Based on these results the quality of the bedrock ranges from poor to excellent.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of dark grey limestone and dolomite of the Gull River Formation.

4.3 Groundwater

A standpipe was installed in BHs 1 and 3, and monitoring wells were installed in BH 2 and BH 4 at the time of drilling. The groundwater (GWL) readings are presented in Table 1. It should be noted that the groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could be vary at the time of construction.

Table 1
Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	67.67	3.91	63.76	February 6, 2008
BH 2	67.25	3.89	63.36	February 6, 2008
BH 3	67.62	Dry	--	February 6, 2008
BH 4	67.66	4.20	96.13	February 6, 2008
BH 5	67.79	Dry	--	March 30, 2015
BH 6	67.91	3.37	64.54	March 30, 2015
BH 7	67.62	3.68	63.94	March 30, 2015

Note: The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located at the intersection of Woodroffe Avenue and Deschenes Street. A geodetic elevation of 67.33 m was provided for the TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated that the proposed multi-storey building will be founded on shallow footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the three (3) levels of underground parking. Hoe ramming could be used where only small quantities of bedrock need to be removed. Line drilling and controlled blasting could be used where large quantities of bedrock need to be removed. The blasting operations should be planned and carried out under the guidance of a professional engineer with experience in blasting operations.

The alignment of a large diameter watermain runs within an easement along the north property boundary of the subject site. It is expected that the adjacent watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent watermain, a vibration attenuation trench is recommended for the bedrock along the north excavation face, as well as a vibration monitoring and control program during the blasting and excavation work required for the proposed building excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow depth of the bedrock at the subject site and the anticipated founding level for the proposed building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed building. Bedrock excavation will be required for the construction of the parking garage levels.

Bedrock Removal

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

Prior to considering blasting operations, the blasting effects on the existing watermain, 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 per second during the blasting program to reduce the risks of damage to the existing structures.

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

A vibration attenuation trench is recommended to be completed within the bedrock along the north property boundary, which is located adjacent to the existing watermain. The construction of the vibration attenuation trench would require line drilling in a tight pattern on both sides of the proposed 1 m wide trench alignment and within the interior portion of the trench to the design underside of footing elevation. A hoe ram operation would be used to break up the bedrock and remove it from the trench. It is expected that the coreholes for the bedrock blasting program may not be possible within 1 to 2 m of the attenuation trench due to the presence of the drilled holes within the attenuation trench, which can cause an energy loss and blow-out during blasting if connected to the blast source by potential fractures within the bedrock. Therefore, a hoe ramming operation will most likely be required to complete the bedrock removal within the area adjacent to the attenuation trench.

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 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.

Vibration Monitoring and Control Plan

To ensure that no disturbance to the existing watermain occurs, a vibration monitoring and control plan (VMCP) is recommended during the excavation program. The purpose of the vibration monitoring and control plan is to provide measures to be implemented by the contractor to manage excavation operations and any other vibration sources during the construction for the proposed development. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

The monitoring program will incorporate real time results at the existing watermain segment adjacent to the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. At least two vibration monitoring devices should be placed adjacent to the existing watermain. It is recommended that the vibration monitoring devices be installed at invert level of the existing watermain and periodically inspected during the construction program.

A copy of the geotechnical report, which includes the VMCP, should be provided to all parties involved with the construction for review. A meeting between Paterson and the site contractor should be conducted prior to any excavation or construction of the subject site to review the following:

- ☐ Review the pre-condition/pre-construction survey;
- ☐ Control measures (i.e vibrations, noise);
- ☐ Monitoring locations;
- ☐ Tracking and reporting of excavation progress, and;

❏ Review procedure for exceedances (i.e vibrations, noise), complaints, evaluation and corrective measures.

When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in the weekly report. The following table outlines the vibration limits for the adjacent watermain segment.

Table 2 - Structure Vibration Limits for adjacent Watermain Segment

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)	Event	Description of Event
<10	all	none	no action required
<40	>10	trigger level	Warning e-mail sent to contractor.
<40	≥15	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.
>40	>15	trigger level	Warning e-mail sent to contractor.
>40	≥20	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.

The monitoring protocol should include the following information:

Trigger Level Event
❏ Paterson will review all vibrations over the established warning level, and;
❏ Paterson will notify the contractor if any vibration occur due to construction activities and are close to exceedance level.

Exceedance Level Event
❏ Paterson will notify all the relevant stakeholders via email;
❏ Ensure monitors are functioning, and;
❏ Issue the vibration exceedance result.

Rock Stabilization

Due to the proximity of existing multi-storey residential apartment building to the west of the proposed development, stabilization of bedrock will be required when the proposed foundation extends below the existing founding elevations.

Horizontal rock anchors may be required at specific locations to prevent pop-outs 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 will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, it is expected that limestone/dolostone bedrock will be encountered at the founding levels.

A factored bearing resistance value at ULS of **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **3,000 kPa** could be used if founded on limestone/dolostone bedrock and 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 the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

Foundations founded on a clean, surface sounded limestone/dolostone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

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.

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

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

Field Program

The seismic shear wave test was completed through the middle of the property, as presented in Drawing PG1609 -2 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an southwest-northeast 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 1 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was connected to a 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 strikes an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four to eight times at each shot location to improve signal to noise ratio. The shot locations are 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 center of the geophone array, as well as 3, 4.5 and 15 m away from the first and last geophone.

The test method completed by Paterson are guided by the standard test procedures outlined by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was completed by 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 below the structures 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 by 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. As bedrock quality increases, the bedrock shear wave velocity also increases. Based on testing results, bedrock is present at a depth of 1.25 m. Once the underground portions of the building are excavated, the structure's foundation will be sitting directly on the dolostone bedrock.

Based on the test results, the bedrock seismic shear wave velocity is 2,200 m/s. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the OBC 2012.

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

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}200m/s}\right)}$$

$$V_{s30} = 2{,}200m/s$$

Based on the seismic test results, the average shear wave velocity, Vs_{30}, for foundations at the subject site is **2,200 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not considered to be susceptible to liquefaction.

5.5 Basement Slab

For the subject site development, all overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the basement floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its standard Proctor maximum dry density.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor.

5.6 Basement Wall

Earth Pressure Distribution

The basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to K γ H where:

K = 0.33 - where slight movement is permissible
K = 0.5 - where no movement is permissible (normal basement wall)
γ = 22 kN/m^3, unit weight of the fill
H = height of the basement wall above sound bedrock if wall poured against bedrock wall excavation, otherwise use full height of basement wall (m)

An additional pressure having a magnitude equal to K q and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q, that may be placed at ground surface adjacent to the wall. The above-noted earth pressure assumes that the wall will be provided with a drainage system.

Horizontal Spring Constant Values

Based on our findings, bedrock is expected at a 1 to 2 m depth below existing ground surface. It is expected that the proposed building's foundation walls will be poured directly against a composite drainage blanket, which will be placed against the bedrock face. It should be noted that deflections in bedrock will not exceed 1 mm. Therefore, the following k values are applicable:

P1: k = 15 MPa/m
P2: k = 35 MPa/m
P3: k = 75 MPa/m

5.7 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or 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.

Anchors can be of the “passive” or the “prestressed” type, depending on whether the anchor tendon is provided with prestress load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the prestressed 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, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed tower, it is recommended that the rock anchors for this project be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength of the weaker of the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the unconfined compressive strength of limestone ranges between about 30 and 80 MPa, which is stronger than most routine grouts. It is recommended that an allowable grout to rock bond stress of 1,000 kPa be used, for Working Stress Design (WSD), for grouted anchor design at this site.

For Limit States Design (LSD), a factored tensile grout to rock bond resistance value at ULS of 1.2 MPa, incorporating a resistance factor of 0.4, can be used.

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. Typical anchor capacities for simple grouted deformed rebar anchors are provided in following table.

Grout Strength Required

The allowable grout to steel bond strength for deformed reinforcing steel bars, in grouted anchors is 10% of the cylinder compressive strength, f'c, of the grout, for WSD. This bond is developed over the perimeter of the bar for the length of the grouted anchor. A minimum grout strength of 30 MPa is recommended.

Table 2 - Typical Grouted Rock Anchor Capacities (WSD)

Anchor Bar Size	Drill Hole Diameter (mm)	Anchor Lengths (m)			Anchor Groups and Capacity (kN)		
		Free	Fixed	Total	Number and Spacing	Allowable Capacity	Factored Tensile Resistance
25M	50	4.15	0.8	4.95	4 at 1 m	120	145
		3.75	0.8	4.55	4 at 1.8 m	120	145
35M	75	3.65	1.05	4.7	2 at 1.8 m	240	290
		5.35	1.05	6.4	4 at 1 m	240	290
		4.95	1.05	6	4 at 1.8 m	240	290
		6.25	1.05	7.3	6 at 1 m	240	290
		5.85	1.05	6.9	6 at 1.8 m	240	290

Notes:

1. Anchor capacities (per anchor) are estimated to provide full allowable tensile resistance (see note 2) for the anchor groups (number and spacing) noted.
2. Allowable capacities use 60% Fy for Grade 400 steel (per anchor).
3. Factored tensile resistance values incorporate a resistance factor of 0.4.
4. Multiply the anchor capacity by the number of anchors in the group to get the group capacity.
5. A minimum grout strength of 30 MPa is recommended.

Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of 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 Drainage

It is understood that the portion of the proposed building foundation walls located below the long-term groundwater table will be placed against a groundwater infiltration control system. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which comes in contact with the proposed building's foundation walls. For the groundwater infiltration control system for the foundation walls, the following is suggested:

❑ Line drill the excavation perimeter.

❑ Hoe ram and grind any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface.

❑ Spray the bedrock vertical surfaces using an elastomeric coating (6 mm thick). The coating should be applied to areas 1 m above the long term groundwater level extending to the bottom of the excavation. The coating should also extend horizontally a minimum 600 mm below the perimeter footings to create a seal at the juncture of the horizontal and vertical bedrock surfaces.

❑ Place a composite drainage layer, such as Miradrain G100N or equivalent, over the coating. The composite drainage layer should extend from finished grade to underside of footing level. All joints should be taped with the appropriate adhesive tape based on a reversed shingle effect.

❑ Pour foundation wall against the composite drainage system.

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 any water that breaches the coating system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to a cistern or sump pit(s) within the lower basement area.

Underfloor Drainage

An underfloor drainage system will be required to control water infiltration. For design purposes, it is recommended that minimum 150 mm diameter perforated pipes be placed at 6 to 8 m centres. The 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

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, as recommended above, 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.

It is anticipated the building foundation walls will occupy the entire property boundary, and, therefore, insufficient room will be available for exterior backfill. The foundation wall should be poured against a drainage system placed against the excavation wall face.

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) 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 the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

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.

For a building provided with two (2) underground parking levels, the requirement for frost protection can be reduced to 1 m in the lowest parking level in areas away from sources of below freezing air temperature such as but not limited to air intakes.

6.3 Excavation Side Slopes

Temporary Shoring System

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 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.

At this site, temporary shoring will be required to complete the required excavations. 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 preliminary 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.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

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. Because the depth at which the apex 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 capacity, than one where the bonded length was just the bottom part of the overall anchor.

The design of the rock anchors can be based on an allowable grout to rock bond stress of 700 kPa at this site. It is recommended that the upper 1.5 m of the bedrock be disregarded. A minimum grout strength of 30 MPa is recommended. Permanent rock anchors should be provided with a double corrosion protection system.

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

Table 3 - 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
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

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 $0.65\ K\ \gamma\ H$ for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of $K\ \gamma\ H$ 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.

If a soldier pile and lagging system is used, the piles could be socketed in the bedrock in pre-augered holes. The augered holes should be advanced at least 2 m into the bedrock and at least 2 m below the bottom of the excavation. A minimum factor of safety of 1.5 should be used.

Underpinning of Adjacent Structures

Based on the relatively shallow depth of the bedrock at the subject site, it is expected that the multi-storey building along the west boundary of the site is most likely founded on the bedrock surface. Therefore, underpinning is not expected to be required for this project.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa (5th edition, March 31, 2006). Trench details should be as per Drawing Nos. W17, S6 and S7.

At least 150 mm of OPSS Granular A should be used for bedding for sewer pipes when placed on soil subgrade. If the bedding is placed on grey silty clay, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (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.

At least 150 mm of OPSS Granular A should be used for bedding for water pipes. The bedding material, which should extend to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular M. 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

Groundwater Control for Building Construction

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. Based on the groundwater level being located within the bedrock, infiltration levels will be low to moderate through the excavation face.

A temporary MOE permit to take water (PTTW) will be required for this project since more than 50,000 L/day will be pumped during the construction phase. A minimum of four to five months should be allocated for completion of the application and issuance of the permit by the MOE.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. 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 (i.e. less than 50,000 L/day) with peak periods noted after precipitation events. It is anticipated that the groundwater flow will be controlled using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that up to 3.5 levels of underground parking are planned for the proposed building with the lower portion of the foundation having a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place against the bedrock face, long-term groundwater lowering is anticipated to be negligible for the area.

Based on our observations, the long term groundwater level is anticipated at a depth of 4 m below the existing grade within the bedrock. Therefore, no adverse effects to neighbouring properties is expected.

6.6 Winter Construction

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.

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 of the geotechnical aspects of the excavating contractor's final shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

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

A 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, we request immediate notification to permit reassessment of our recommendations.

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

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

Scott S. Dennis, P.E.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Dec. 19-2017
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 97.33m.

REMARKS

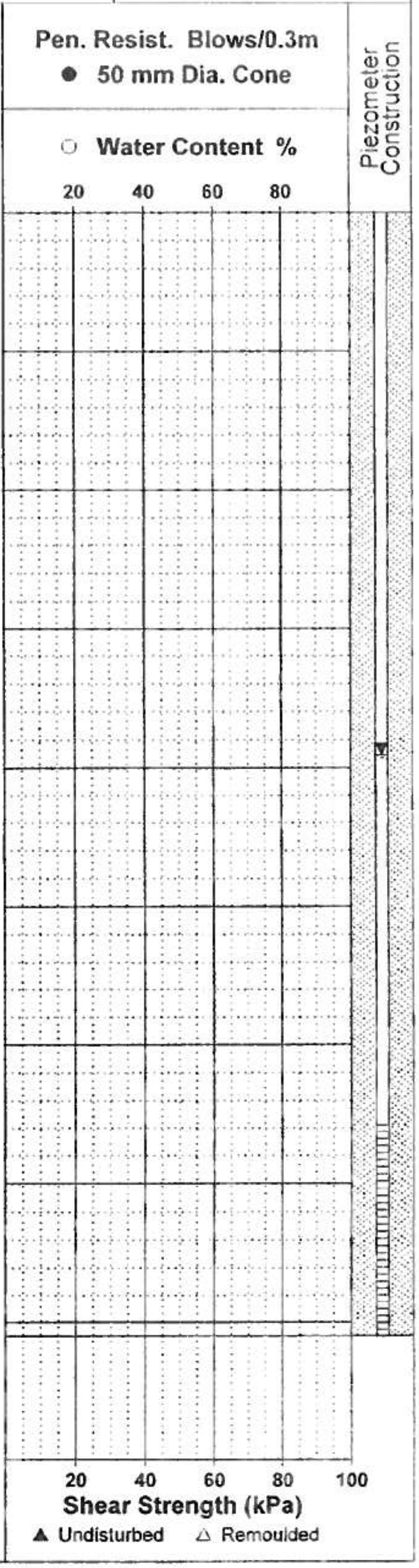
BORINGS BY CME 55 Power Auger

DATE 31 Jan 08

FILE NO. **PG1609**

HOLE NO. **BH 1**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.67
TOPSOIL 0.13	AU	1				
Very dense, brown **SILTY SAND** with gravel	SS	2	77	50+	1	66.67
	SS	3	80	50+		
2.28					2	65.67
BEDROCK Weathered limestone with mud seams	SS	4	71	50+		
2.62	RC	1	100	45	3	64.67
	RC	2	97	97	4	63.67
BEDROCK Grey dolostone	RC	3	100	100	5	62.67
					6	61.67
	RC	4	100	100	7	60.67
- increasing shale content below 7.5m depth	RC	5	100	100		
8.10					8	59.67
End of Borehole						
(GWL @ 3.91m-Feb. 6/08)						



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 97.33m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 31 Jan 08

FILE NO. PG1609

HOLE NO. BH 2

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL	0.15	AU	1			0	67.25
GLACIAL TILL: Very dense, brown silty sand and gravel with cobbles and boulders		SS	2	100	50+		
						1	66.25
	1.42						
BEDROCK: Weathered limestone with mud seams							
						2	65.25
		RC	1	55	16		
	2.80						
						3	64.25
		RC	2	100	78		
						4	63.25
BEDROCK: Grey dolostone							
						5	62.25
		RC	3	100	65		
						6	61.25
		RC	4	100	93		
						7	60.25
	7.40						
BEDROCK: Dark grey shaley dolostone		RC	5	100	100		
	7.95						
End of Borehole							
(GWL @ 3.89m-Feb. 6/08)							

STRATA PLOT

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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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 97.33m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 31 Jan 08

FILE NO. **PG1609**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	67.62		
TOPSOIL 0.13		AU	1						
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	2	40	60+	1	66.62		
1.58		SS	3	100	50+				
End of Borehole Practical refusal to augering @ 1.58m depth (BH dry-Feb. 6/08)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 97.33m.

REMARKS

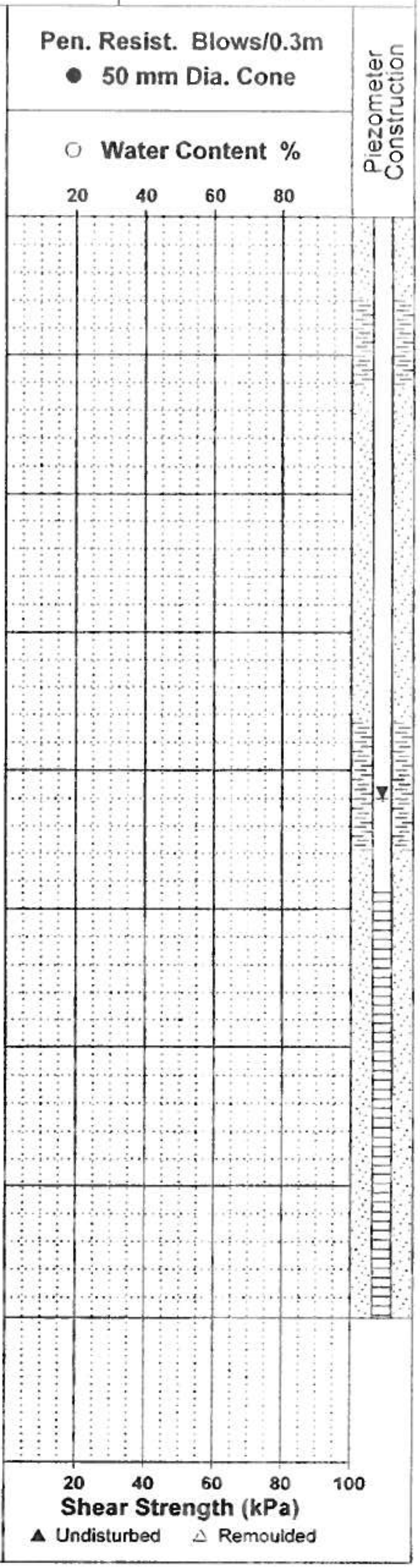
BORINGS BY CME 55 Power Auger

DATE 1 Feb 08

FILE NO. PG1609

HOLE NO. BH 4

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	67.66
TOPSOIL	0.15						
Very dense, brown **SILTY SAND**, trace clay	0.46	AU	1				
GLACIAL TILL Very dense, brown silty sand, gravel, cobbles and boulders	0.79 1.20	SS	2	100	50+		
BEDROCK Weathered limestone with mud seams		RC	1	38	16	1	66.66
		RC	2	87	22	2	65.66
						3	64.66
BEDROCK Grey dolostone		RC	3	100	93	4	63.66
		RC	4	93	93	5	62.66
	6.28					6	61.66
BEDROCK Grey shaley dolostone		RC	5	100	97	7	60.66
	7.95	RC	6	100	100		
End of Borehole							
(GWL @ 4.20m-Feb. 6/08)							



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 67.33m.

REMARKS

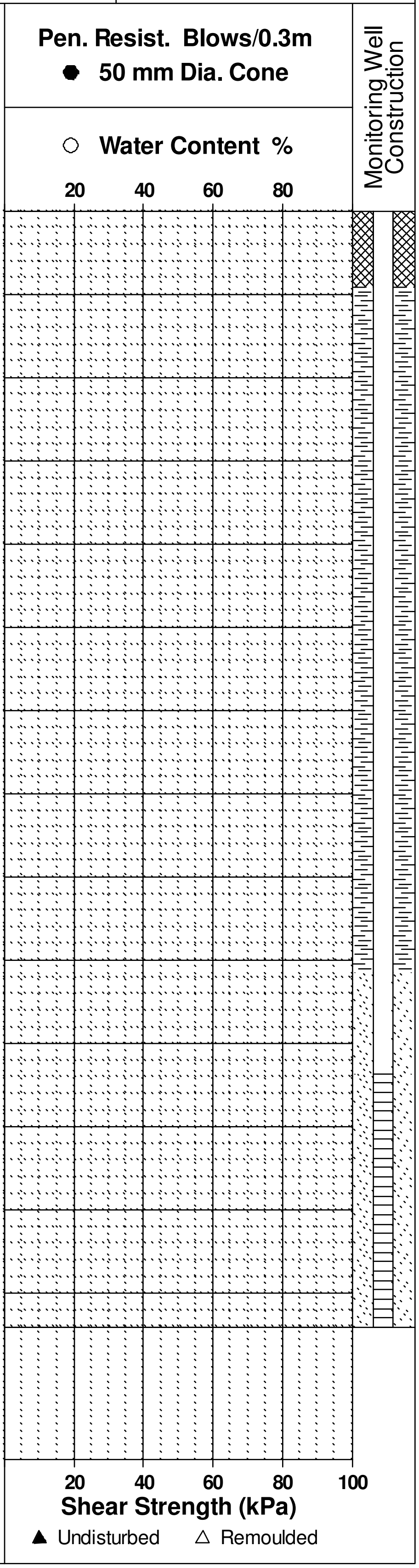
BORINGS BY CME 75 Power Auger

DATE Mar 13, 15

FILE NO. PG1609

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.79
FILL: Black silty sand with gravel and metal nails	AU	1				
0.91	RC	1	92	0	1	66.79
	RC	2	72	49	2	65.79
					3	64.79
	RC	3	93	79		
BEDROCK: Poor to good quality, grey dolomite					4	63.79
	RC	4	98	92	5	62.79
					6	61.79
- calcite vanes noted from 6.2 to 7.7m depth	RC	5	100	100	7	60.79
	RC	6	100	100	8	59.79
					9	58.79
	RC	7	93	94	10	57.79
	RC	8	98	92	11	56.79
					12	55.79
	RC	9	100	98	13	54.79
13.41						
End of Borehole (BH dry - March 30, 2015)						



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 67.33m.

REMARKS

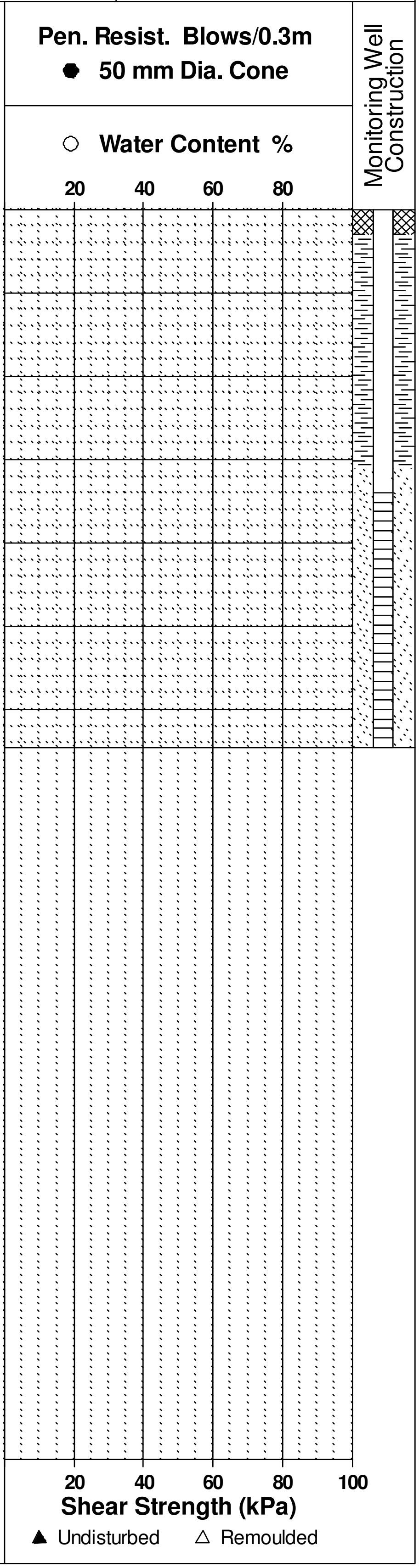
BORINGS BY CME 75 Power Auger

DATE Mar 13, 15

FILE NO. PG1609

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.91
FILL: Brown sandy silty with gravel and cobbles		AU	1				
		SS	2	83	50+	1	66.91
1.67		SS	3	83	50+		
BEDROCK: Poor to good quality, grey dolomite		RC	1	83	40	2	65.91
		RC	2	97	74	3	64.91
						4	63.91
		RC	3	100	75	5	62.91
6.45		RC	4	100	88	6	61.91
End of Borehole (GWL @ 3.68m-March 30, 2015)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building, Woodroffe Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, corner of Woodroffe Avenue and Deschenes Street. Geodetic elevation = 67.33m.

REMARKS

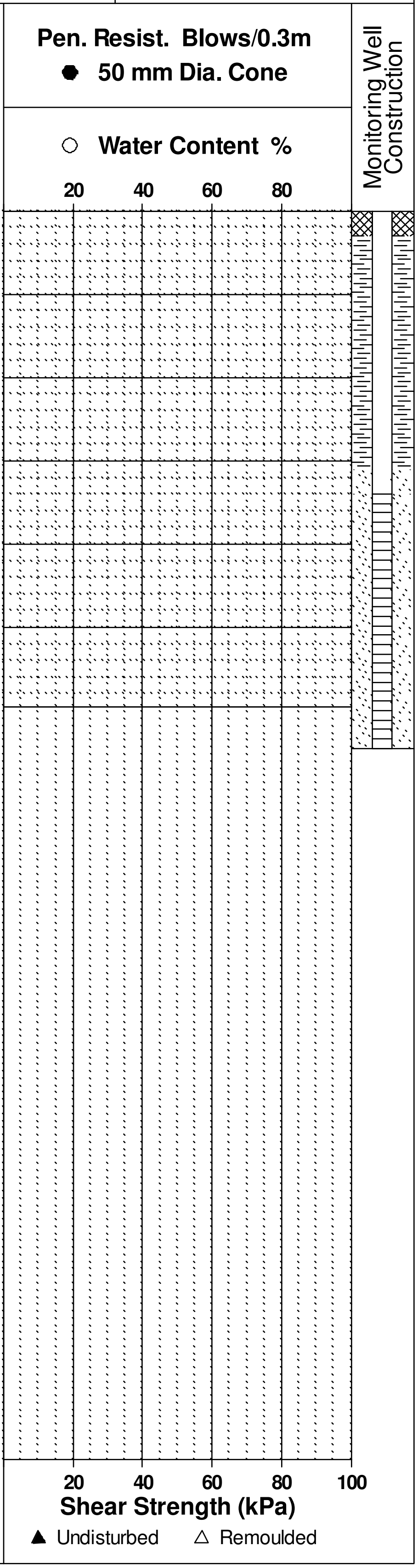
BORINGS BY CME 75 Power Auger

DATE Mar 16, 15

FILE NO. PG1609

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	67.62
FILL: Topsoil with sand, some gravel, cobbles and boulders		AU	1	0			
		SS	2	100	50+	1	66.62
1.57		SS	3	100	50+		
		RC	1	100	51	2	65.62
						3	64.62
BEDROCK: Good quality, dolostone bedrock		RC	2	100	87		
						4	63.62
						5	62.62
		RC	3	100	78		
5.96							
End of Borehole (GWL @ 3.68m-March 30, 2015)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

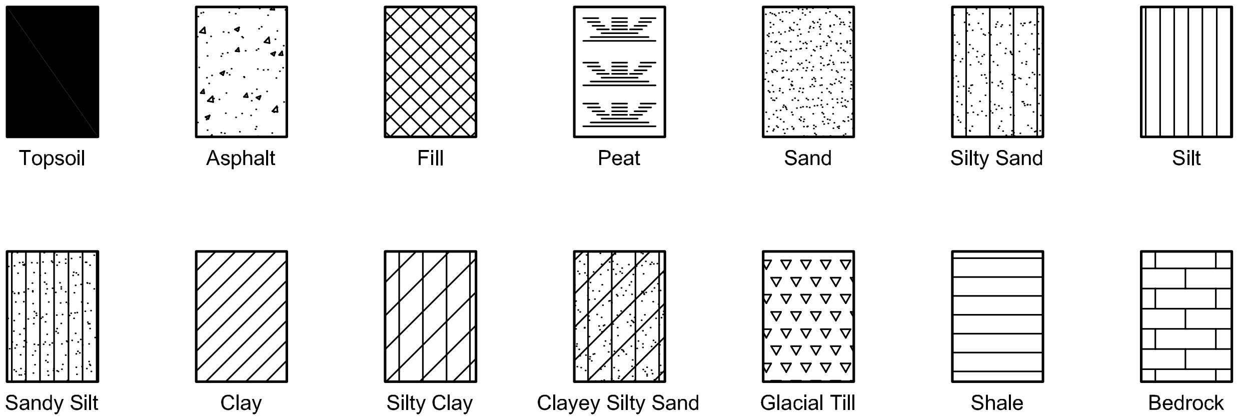
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

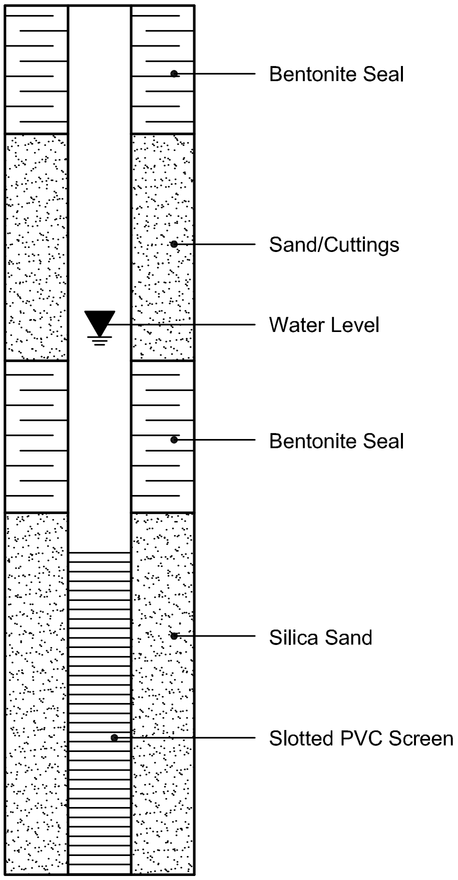
SYMBOLS AND TERMS (continued)

STRATA PLOT

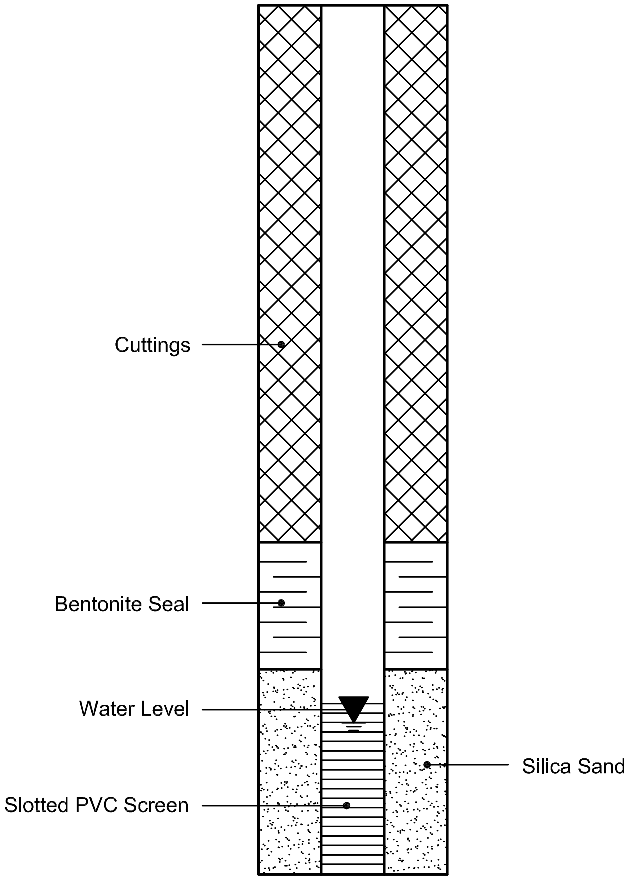


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION -15m

FIGURE 3 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION -4.5m

DRAWING PG1609-1 - TEST HOLE LOCATION PLAN

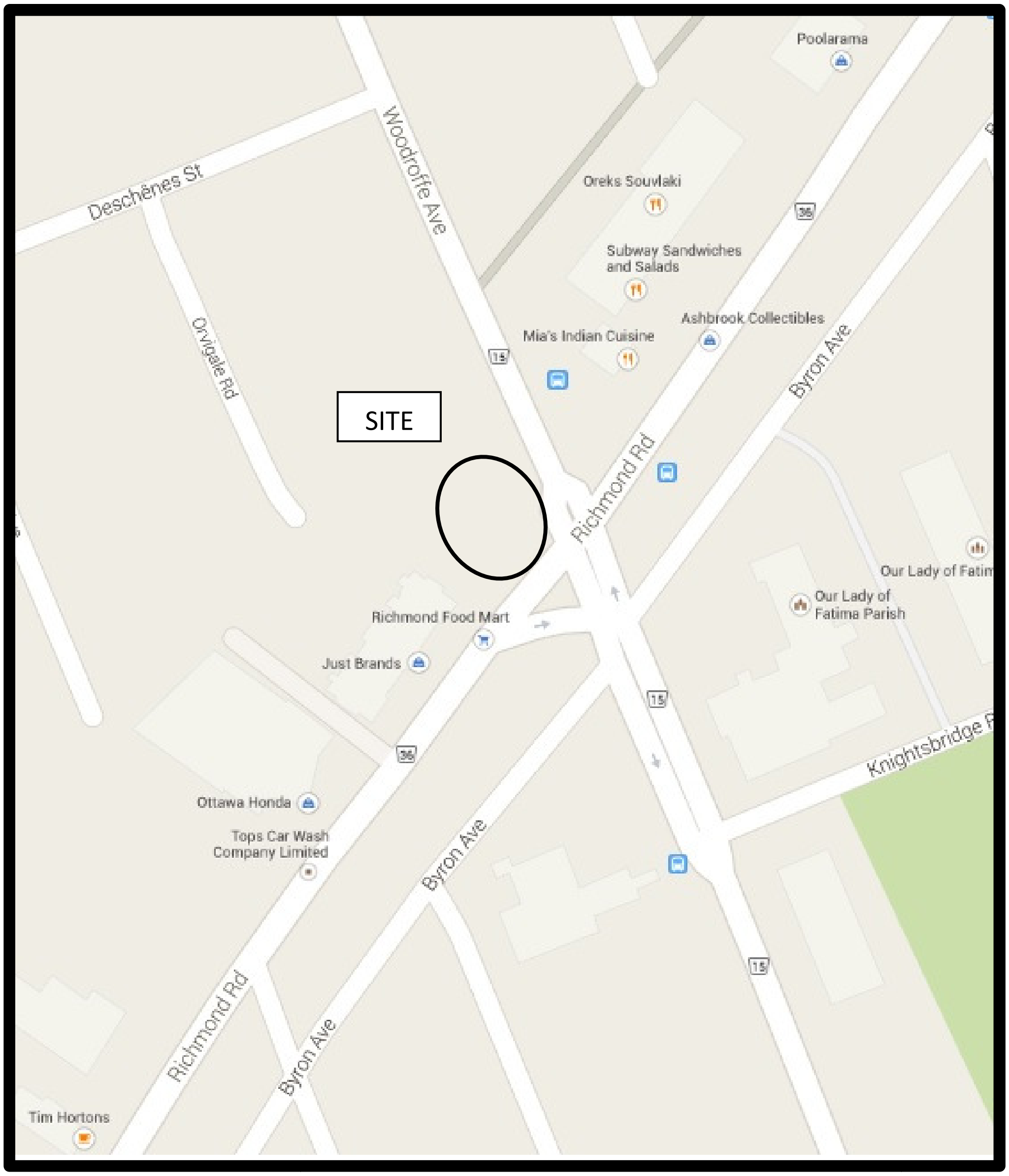


FIGURE 1

KEY PLAN

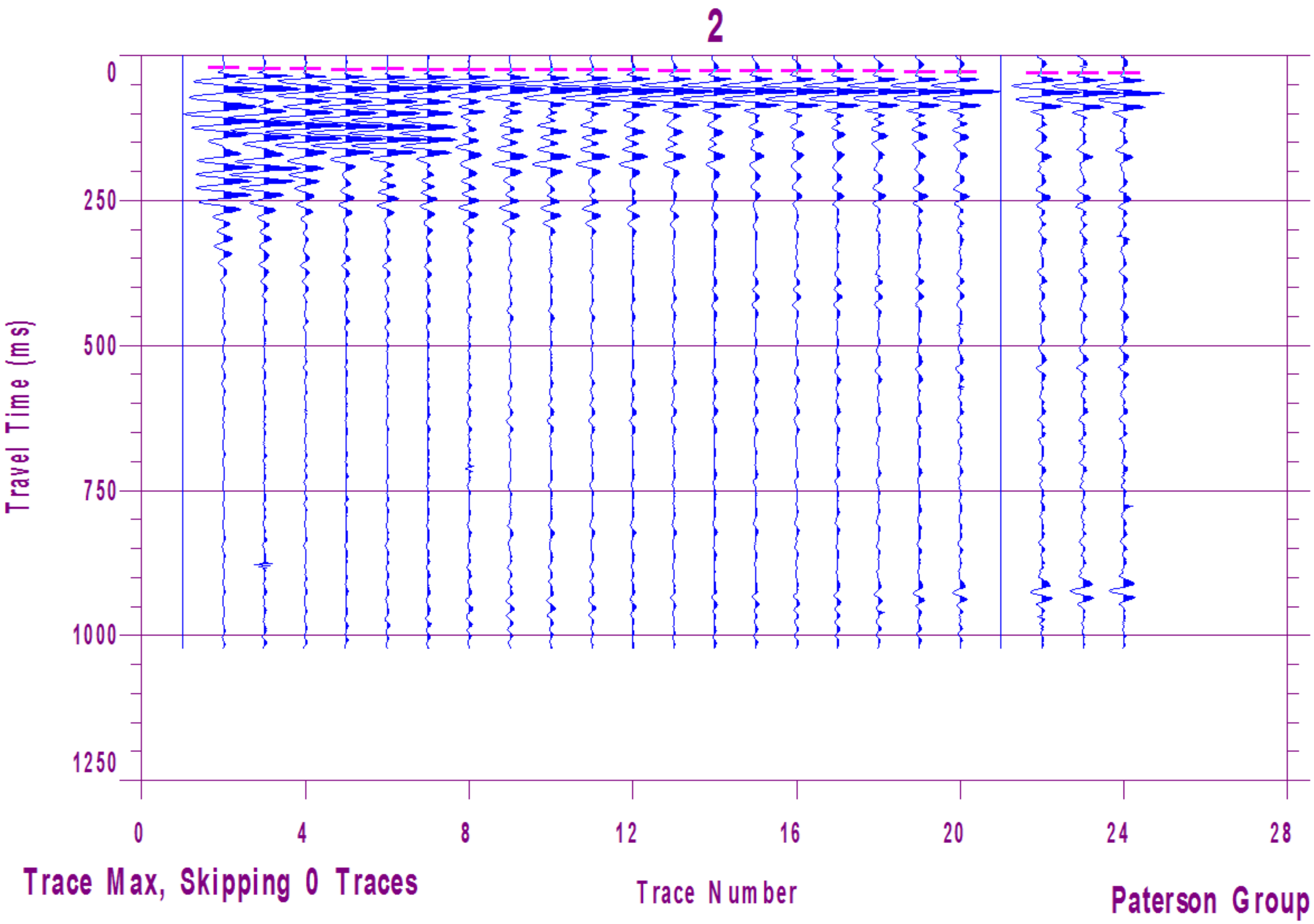


Figure 2 – Shear Wave Velocity Profile at Shot Location -15.0 m

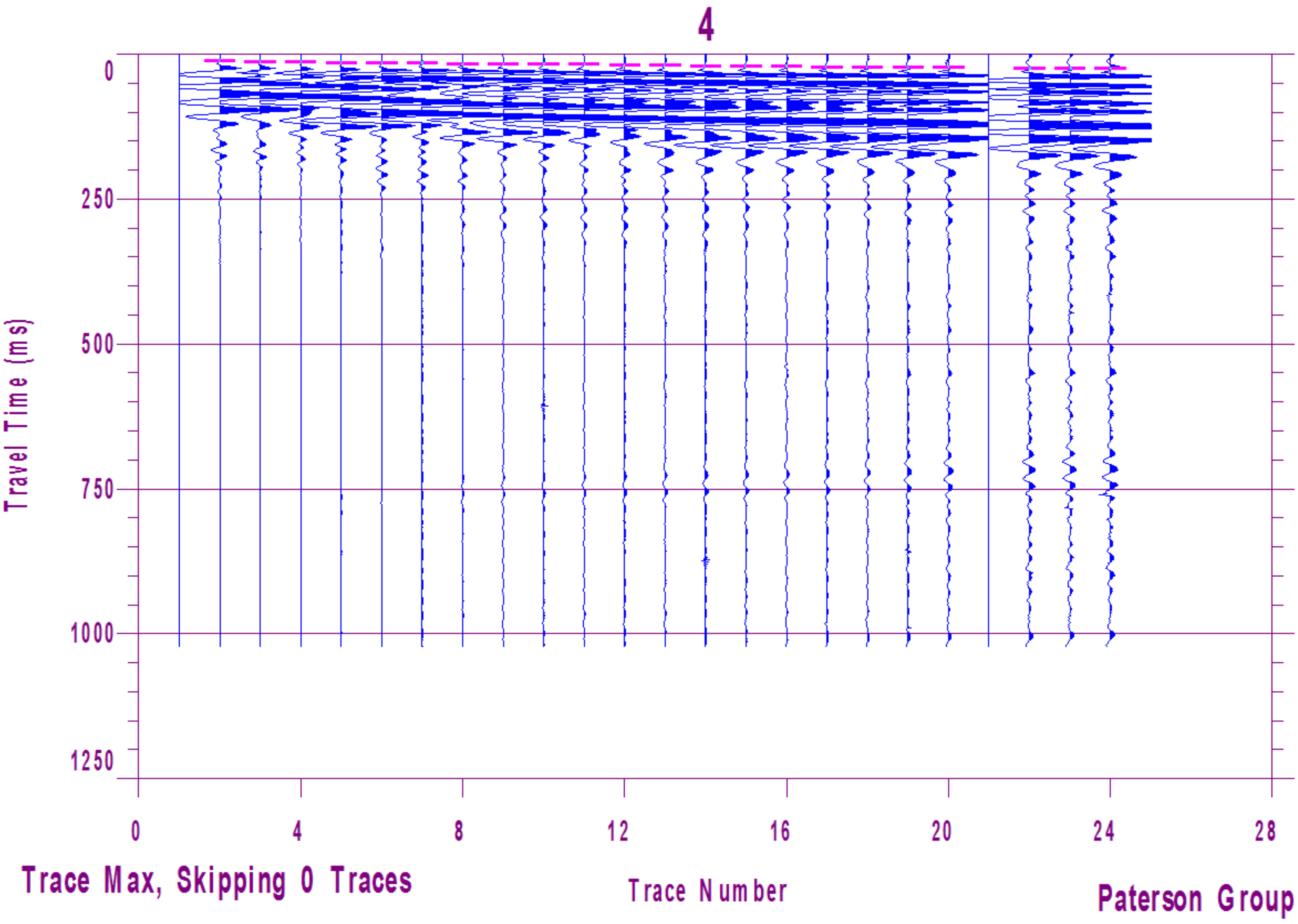
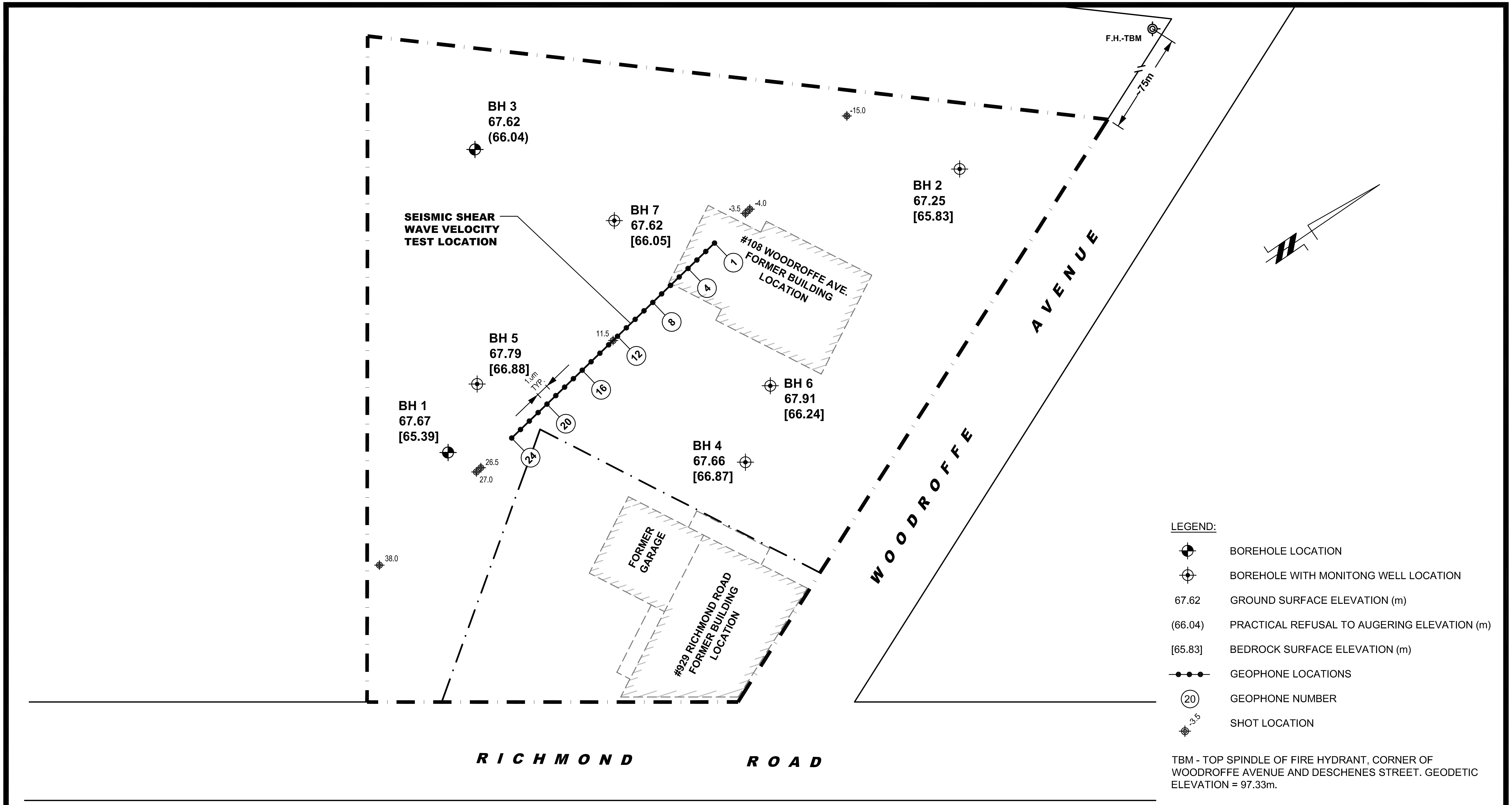


Figure 3 – Shear Wave Velocity Profile at Shot Location -4.5 m



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NO.	REVISIONS	DATE	INITIAL
4	UPDATED BASE PLAN REMOVED	10/11/2017	SD
3	BASE PLAN UPDATED AND NEW BOREHOLES ADDED	09/04/2015	DJG
2	SEISMIC ARRAY ADDED	15/12/2014	DJG
1	GRADES REFERED TO GEODETIC DATUM	16/09/2008	DJG

WESTBORO POINT DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
929 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 12/2017
Drawn by: MPG
Report No.: PG1609-1
Checked by: SD
Drawing No.: PG1609-1
Approved by: DJG

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