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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise & Vibration Studies

Geotechnical Investigation

Proposed Residential Development
1125 to 1149 Cyrville Road
Ottawa, Ontario

Prepared For

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Report: PG6072-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Westrich Pacific Corp. to conduct a geotechnical investigation for the proposed commercial development to be located at 1125 to 1149 Cyrville Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objectives of the geotechnical investigation were to:

❏ Determine the subsoil and groundwater conditions at this site by means of boreholes,

❏ Provide geotechnical recommendations pertaining to 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 carried out as a separate. A report addressing environmental concerns for the subject site was issued under separate cover.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of two multi-storey residential buildings with at least one underground parking level. Associated at grade access lanes, pedestrian pathways and landscaped areas are also anticipated. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The fieldwork program for the investigation was carried out on August 16 and17, 2011. At that time, 12 boreholes were advanced to depths ranging from 0.5 to 5.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration site features. The locations of the boreholes are shown on Drawing PG6072-1 - Test Hole Location Plan in Appendix 2.

The boreholes were advanced 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 from our geotechnical department. The drilling procedure consisted of augering to the required depths at the selected locations, 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 soil samples were transported to our laboratory. Bedrock was cored at three (3) borehole locations using diamond drilling techniques. The bedrock core was recovered from each core run, placed in cardboard boxes, and sent to our laboratory for further review. The depths at which the split spoon, auger and bedrock samples were recovered from the boreholes are shown as SS, AU, and RC, 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.

Diamond drilling using BX size coring equipment was carried out in three (3) boreholes to determine the nature of the bedrock and to assess its quality. Recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run).

The RQD value is the ratio, in percentage, of the total length of 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.

Subsurface conditions observed in the boreholes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

. Groundwater

Flexible polyethylene standpipes and three (3) monitoring wells were installed at selected borehole locations (BH 1, BH 2 and BH 12) to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

Typical monitoring well construction details are described below:

- ❏ Slotted 50 mm diameter PVC screen.
- ❏ 50 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.
- ❏ bentonite hole plug placed from 300 mm above PVC slotted screen to near ground surface.
- ❏ 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 monitoring well construction details.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant, located near the intersection of Michael Street and Cyrville Road. A geodetic elevation of 70.62 m was provided for the TBM. The location and ground surface elevations at borehole locations are presented on Drawing PG6072-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject site is presently vacant, relatively flat and covered by gravel and asphalt parking areas and access lanes with some treed and landscaped areas.

The site is bordered to the north by commercial buildings and vacant land, to the east by institutional and commercial buildings, to the south by Cyrville Road and to the west by commercial buildings.

4.2 Subsurface Profile

The soil profile encountered at the test hole locations consists of pavement structure, topsoil or crushed stone fill at ground surface underlain by brown silty clay with gravel and/or silty sand with gravel. A weathered shale bedrock was encountered below the abovenoted layers at all borehole locations. Shale bedrock was cored at BH 1, BH 2 and BH 12 to a maximum depth of 5.7 m below existing ground surface. Specific details of the soil profile at the test hole locations can be seen on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the bedrock in the immediate area of the subject site consists of potentially expansive shale from the Billings Formation at a 2 to 5 m depth.

4.3 Groundwater

Groundwater levels were noted to be dry upon completion of the field program. Groundwater levels at the borehole locations were measured on August 22, 2011 and the results are presented in Table 1 below. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Table 1 - Groundwater Levels

Test Hole Number	Ground Elevation (m)	Groundwater Level	
		Depth (m)	Elevation (m)
BH 1*	68.66	2.29	66.37
BH 2*	69.49	2.94	66.55
BH 3	69.54	Dry	N/A
BH 4	68.66	1.98	66.68
BH 7	69.16	Dry	N/A
BH 9	68.65	Dry	N/A
BH 11	68.89	Dry	N/A
BH 12*	70.24	3.29	66.95

Notes:
* - Monitoring well installed
Borehole locations not indicated were unaccessible or could not be located

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed building will be founded by conventional footings placed on the shale bedrock surface.

Bedrock removal will be required for the proposed building excavations. Bedrock removal may also be required fro installation of site services, dependent on the depths of the proposed utilities.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Bedrock Removal

Where the bedrock is weathered and/or only small quantities of bedrock need to be removed, hoe ramming is an option of bedrock removal. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and 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 25 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.

Vibration Considerations

Construction operations could be 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 a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

Bedrock stabilization may be required where the proposed foundation extends into the sound bedrock.

Rock anchors and rock face protection may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for rock face protection and rock anchors within the sound bedrock should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site.

The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface-sounded bedrock surface can be designed using a bearing resistance value at SLS of **500 kPa** and a factored bearing resistance value at ULS of **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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

The potential long term post-construction total and differential settlements for footings placed on surface-sounded bedrock are estimated to be negligible.

Footings placed on engineered fill approved by the geotechnical consultant can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **300 kPa**. Engineered fill should consist of an OPSS Granular A or Granular B Type II material placed in maximum 300 mm loose lifts and compacted to a minimum 98% of its SPMDD.

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

Footings placed on an engineered fill bearing medium and designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizantally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity, such as concrete.

Adequate lateral support is provided to an engineered fill bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

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 (OBC) 2012. The shear wave velocity testing was completed by Paterson personnel. The shear wave velocity profiles from our testing are presented in Figures 2 and 3 in Appendix 2.

Field Program

The shear wave testing location is presented in Drawing PG6072-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line oriented roughly along north-south within the east portion of the site. 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 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot location's are located at the centre of the geophone array and 1, 2 and 10.5 m away from the first and last geophones.

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 bedrockquality increases, the bedrock shear wave velocity also increases.

It is expected that the footings of the proposed building will be founded on the bedrock surface. Based on our analysis, the bedrock shear wave velocity was calculated to be 1,951 m/s.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2006, as presented below.

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

$$V_{s30} = \frac{30m}{\left(\frac{30m}{1{,}951m/s}\right)}$$

$$V_{s30} = 1{,}951m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for shallow foundations located at the subject site is 1,951 m/s. 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 site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the native soil or engineered fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed is considered, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft or poor performing areas within the subgrade should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone compacted to 98% of the material's SPMDD.

In consideration of the groundwater conditions encountered at the time of the construction, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet such as the building's sump pit, should be provided in the clear stone under the basement floor.

5.6 Basement Wall

Below the bedrock surface, it is expected that the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.25 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_O) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of the pavement structure for the parking garage and flexible pavement structure for the at-grade parking areas and access lanes.

Table 2 - Recommended Rigid Pavement Structure - Parking Garage

Thickness (mm)	Material Description
125	Reinforced Concrete Slab
200	**BEDDING** - OPSS Granular A Crushed Stone
SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 3 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Based on the current building plans to provide groundwater control and to protect the expansive shale, the following programs are recommended:

Option A - Double Sided Pour

- A minimum 100 mm thick layer of lean concrete (minimum 15 MPa - 28 day compressive strength) should be placed across the base of the excavation.

- The excavation sidewalls below the bedrock surface, where groundwater is encountered at the time of construction, should be sparyed with an elastomeric coating waterproofing layer. The elastomeric coating should extend horizontally at least 300 mm below the underside of footing. The extent of the elastomeric membrane layer should be determined by the geotechnical consultant at the time of construction.

- The base and sidewalls of the elevator pit and sump pits should be lined with a waterproofing membrane to prevent groundwater infiltration.

- A perimeter foundation drainage system should be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The perimeter drainage system should be drained by gravity to the storm sewer.

- The exterior foundation wall should be dampproofed.

- A composite drainage blanket, such as Miradrain G100N or Delta Drain 6000, should be placed across the foundation wall and connected to the perimeter foundation drainage system.

- A sub-floor drainage system, consisting of a 150 mm diameter perforated corrugated PVC pipe, placed at 6 m centres and connected to the building's sump pit units is recommended.

Option B - Blind-Sided Pour

- ❑ The basement wall will be blind poured against the bedrock excavated face.
- ❑ The excavation sidewalls below the bedrock surface, where groundwater is encountered at the time of construction, should be sprayed with an elastomeric coating waterproofing layer. The elastomeric coating should extend horizontally at least 300 mm below the underside of footing. The extent of the elastomeric membrane layer should be determined by the geotechnical consultant at the time of construction.
- ❑ The base and sidewalls of the elevator pit and sump pits should be lined with a waterproofing membrane to prevent groundwater infiltration.
- ❑ A foundation drainage system should be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing to allow the infiltration of water to flow to the interior sub-floor drainage system.
- ❑ A composite drainage blanket, such as Miradrain G100N or Delta Drain 6000, should be placed across the foundation wall and connected to the perimeter foundation drainage system.
- ❑ Footings should be surrounded on all exposed sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure.
- ❑ A sub-floor drainage system, consisting of a 150 mm diameter perforated corrugated PVC pipe, placed at 6 m centres and connected to the building's sump pit units is recommended.

Paterson should be provided with the finalized design drawings and founding depths to provide a full waterproofing and drainage design based on the finalized layout and depth of construction.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

Exterior unheated footings, such as 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.

It has been our experience that insufficient frost protection is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

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 should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided below is to assist the contractor in completing a safe shoring system.

The shoring designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes during construction to the approved shoring design should be reported immediately to the owner's consultants prior to implementation.

The temporary system could 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 included to the earth pressures described below. The shoring system could be cantilevered, anchored or braced. Generally, the shoring systems is provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. If consideration is given to utilizing a raker style support for the shoring system, the structural engineer should ensure that the design selected minimizes lateral movements to tolerable levels.

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

Table 4 - 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 movement is not permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

A 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

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of its 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 minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

A temporary Category 3 Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded directly over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Protection of Potential Expansive Bedrock

It is expected that potentially expansive shale will be encountered at the subject site. A potential for heaving and rapid deterioration of the shale bedrock exists at this site. Our recommendations provided in Subsection 6.1 take into consideration the potential for heaving and rapid deterioration of the shale bedrock which could occur at the subject site.

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.

- ☐ 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.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.
- ☐ Review bedrock excavation activities and exposed vertical bedrock faces.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

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

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

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

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

O.Canton

Owen Canton, E.I.T.

LICENSED PROFESSIONAL ENGINEER
Nov. 16, 2021
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 16, 2011

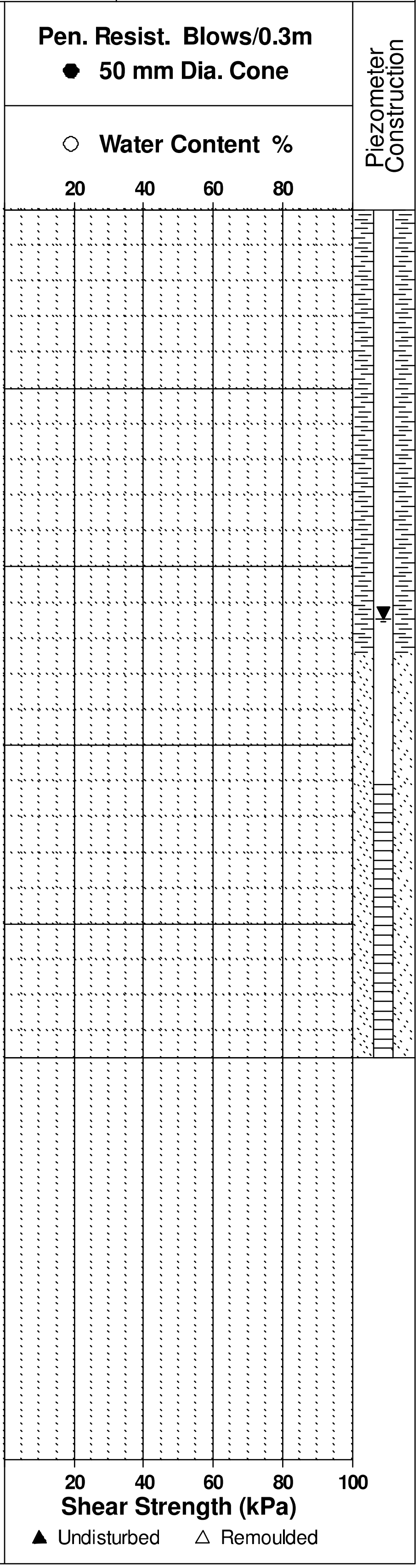
FILE NO. PG2471

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
25mm Asphaltic concrete **FILL:** Crushed stone	0.51
Brown **SILTY CLAY,** trace shale and organics	1.35
BEDROCK: Heavily fractured and weathered, black shale - moderately fractured by 3.0m depth	4.75
End of Borehole (GWL @ 2.29m-Aug. 22/11)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	10
SS	3	0	50+
AU	4		
RC	1	100	93
RC	2	100	96

DEPTH (m)	ELEV. (m)
0	68.66
1	67.66
2	66.66
3	65.66
4	64.66



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REMARKS

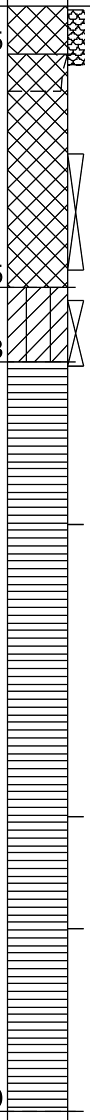
BORINGS BY CME 55 Power Auger

DATE August 16, 2011

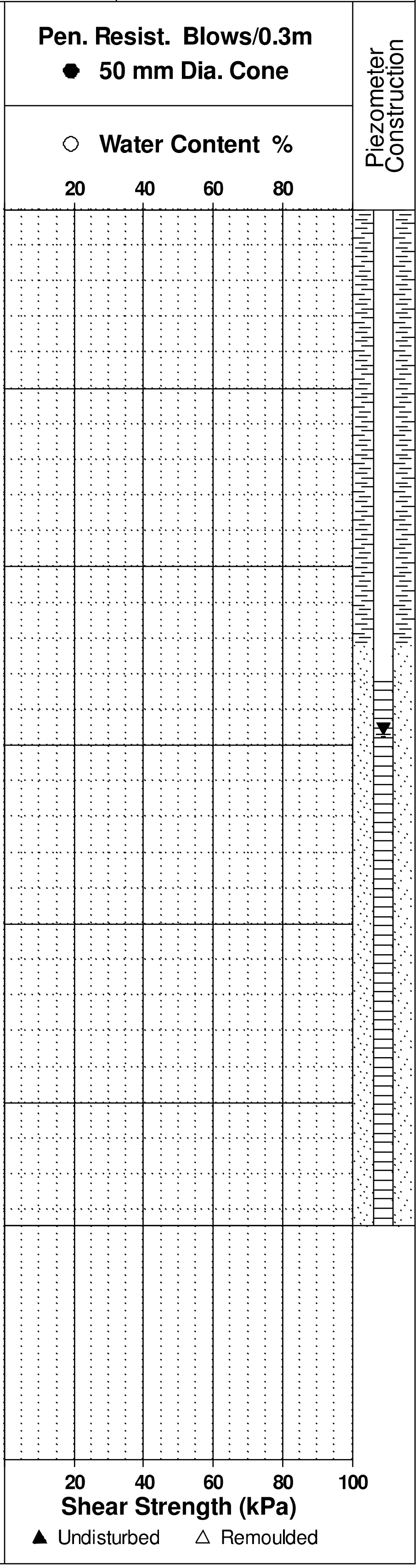
FILE NO. PG2471

HOLE NO. BH 2

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
13mm Asphaltic concrete		AU	1		
FILL: Crushed stone	0.25				
FILL: Brown silty clay with sand, trace shale and gravel	1.45	SS	2	50	12
Black **SILTY CLAY** with sand seams and topsoil	1.83	SS	3		50+
BEDROCK: Heavily fractured and weathered, black shale	5.69	RC RC RC RC	1 2 3 4	62 95 87 97	0 0 0 14
End of Borehole (GWL @ 2.94m-Aug. 22/11)					



DEPTH (m)	ELEV. (m)
0	69.49
1	68.49
2	67.49
3	66.49
4	65.49
5	64.49



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REMARKS

BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

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	69.54		
FILL: Crushed stone 0.30	▨								
FILL: Brown silty clay with sand, gravel, trace shale 0.76	▨								
End of Borehole Practical refusal to augering @ 0.76m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

HOLE NO. BH 3A

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.54
FILL: Crushed stone 0.25	AU	1				
FILL: Brown silty sand with organics 1.07	SS	2	50	17	1	68.54
Brown **SILTY CLAY** with sand 1.45	SS	3	43	50+		
BEDROCK: Heavily fractured and weathered, black shale with mud seams 2.03					2	67.54

End of Borehole

Practical refusal to augering @ 2.03m depth

(BH dry - Aug. 22/11)

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

○ **Water Content %**
20 40 60 80

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

Piezometer Construction

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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

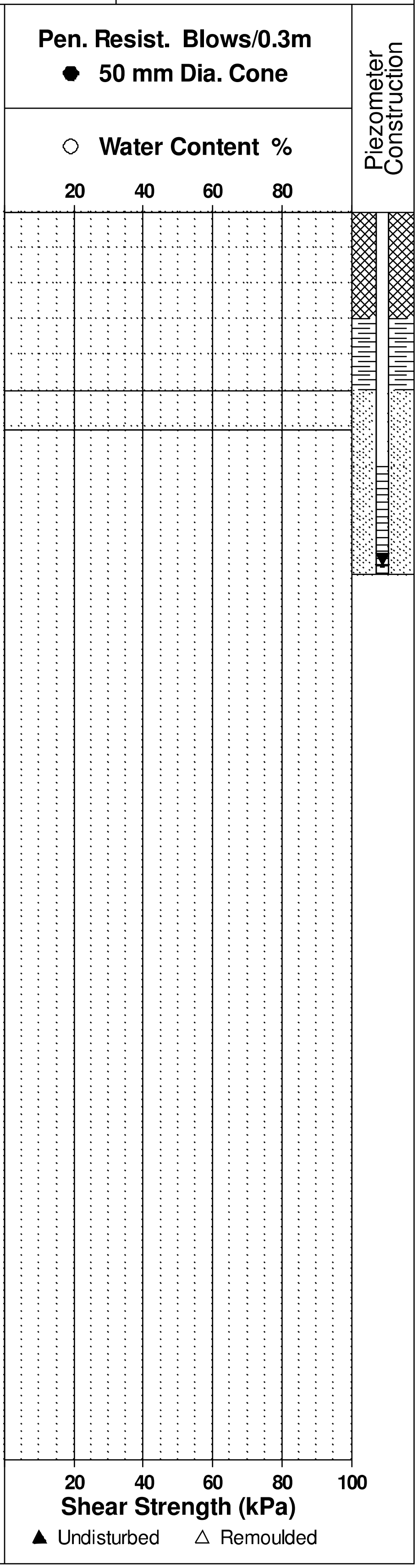
HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.66
FILL: Crushed stone 0.25							
FILL: Brown silty sand, trace gravel 0.89							
TOPSOIL, trace clay and sand 1.09		SS	1	88	50+	1	67.66
BEDROCK: Heavily fractured and weathered, black shale 1.22							

End of Borehole

Practical refusal to augering @ 1.22m depth

(GWL @ 1.98m-Aug. 22/11)



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REMARKS

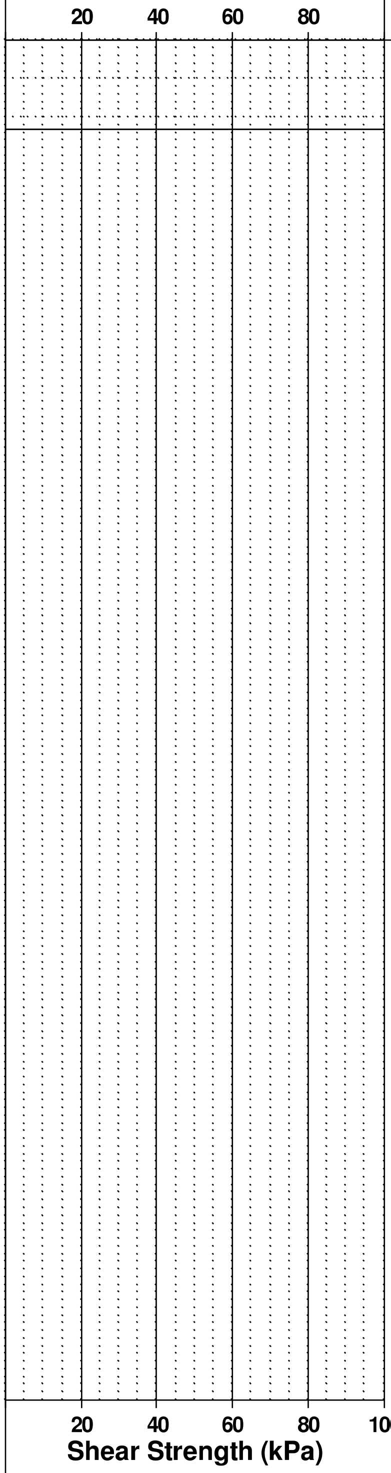
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	68.06		
FILL: Brown silty sand with gravel and shale 0.46	▨	SS	1	83	50+				
End of Borehole Practical refusal to augering @ 0.46m depth									



▲ Undisturbed △ Remoulded

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REMARKS

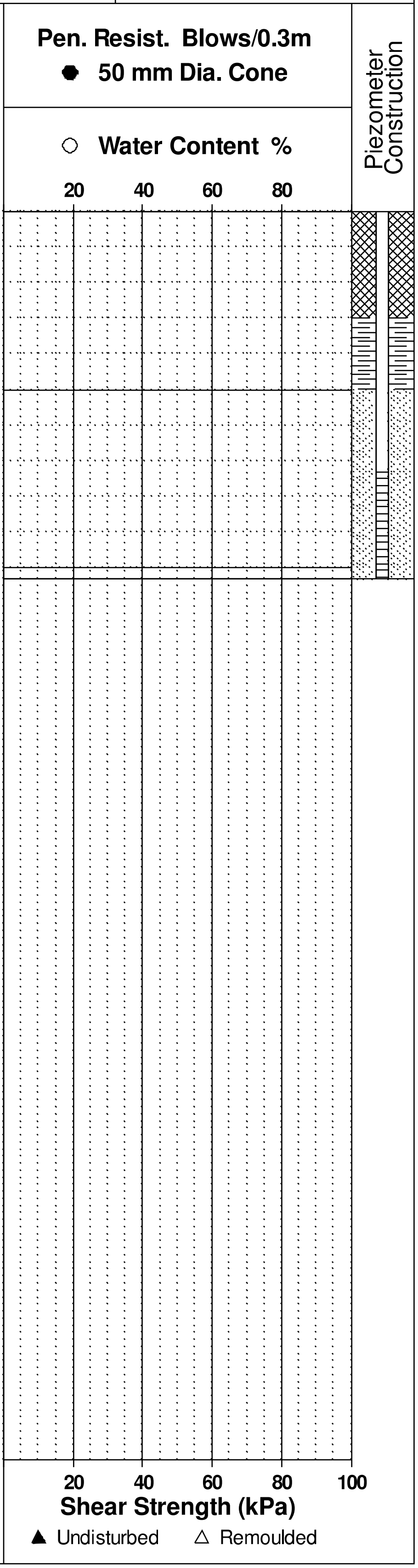
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	68.50
FILL: Brown silty sand with gravel and fractured shale 0.60		SS	1	42	8		
Brown **SILTY CLAY** with sand and fractured shale 1.22		SS	2	82	17	1	67.50
BEDROCK: Heavily fractured and weathered, black shale 2.06		SS	3	20	50+	2	66.50
End of Borehole Practical refusal to augering @ 2.06m depth							



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Commercial Development - 1125-1149 Cyrville Road
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REMARKS

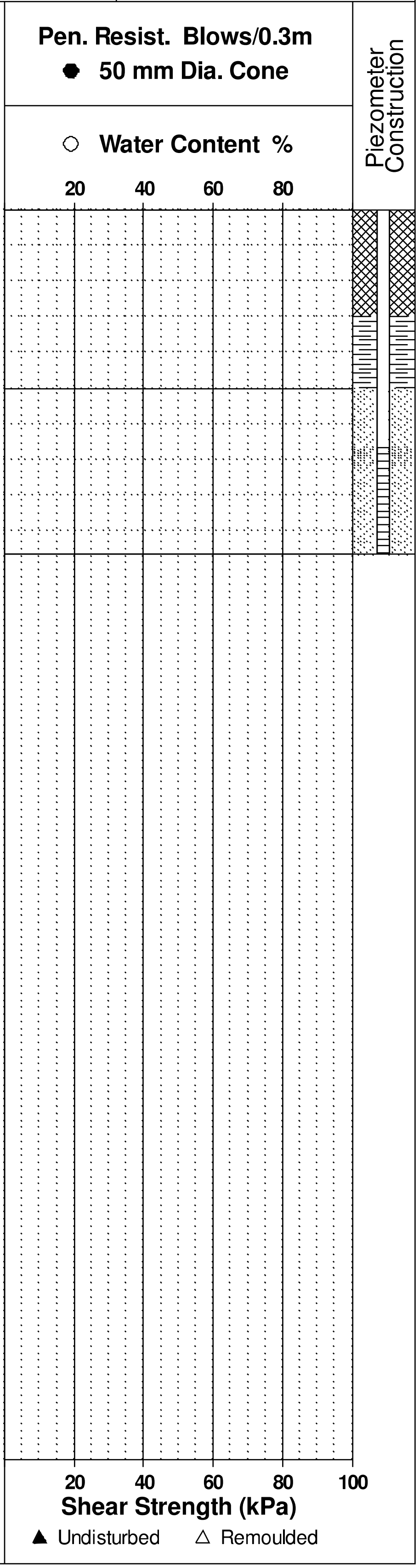
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.16
FILL: Crushed stone — 0.25							
FILL: Brown silty sand with crushed stone, trace concrete — 0.76		SS	1	54	38		
TOPSOIL with silty clay, trace gravel — 1.68		SS	2	79	5	1	68.16
BEDROCK: Heavily fractured and weathered, black shale — 1.93		SS	3	100	50+		
End of Borehole Practical refusal to augering @ 1.93m depth (BH dry - Aug. 22/11)							



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REMARKS

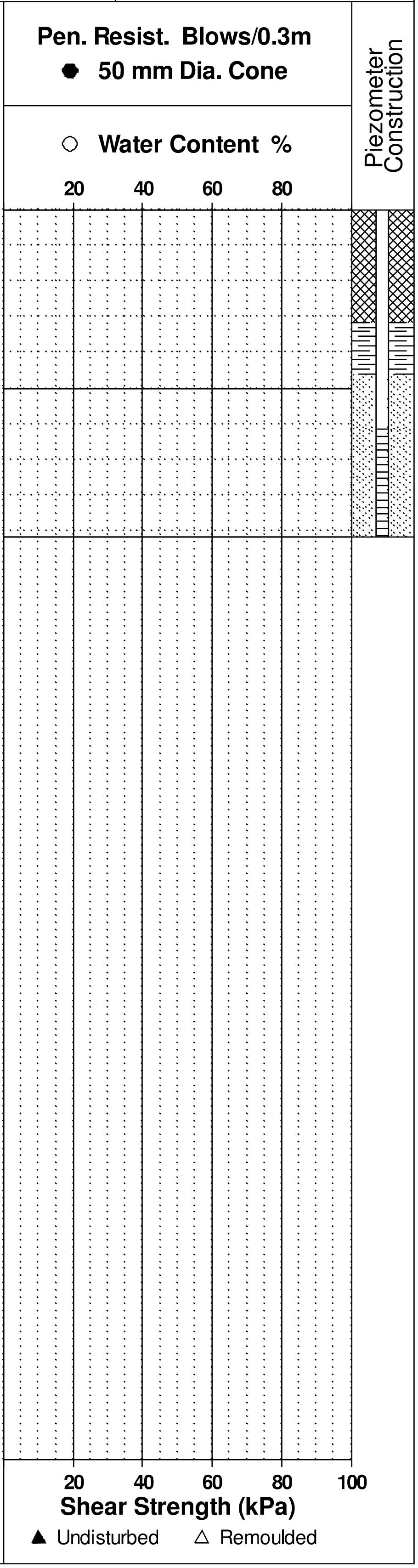
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 8

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	69.07
FILL: Crushed stone 0.33						
Brown **SILTY CLAY** with sand, gravel, trace organics 1.37	SS	1	15	6	1	68.07
BEDROCK: Heavily fractured, weathered, black shale 1.83	SS	2	27	50+		
End of Borehole						
Practical refusal to augering @ 1.83m depth						



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REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

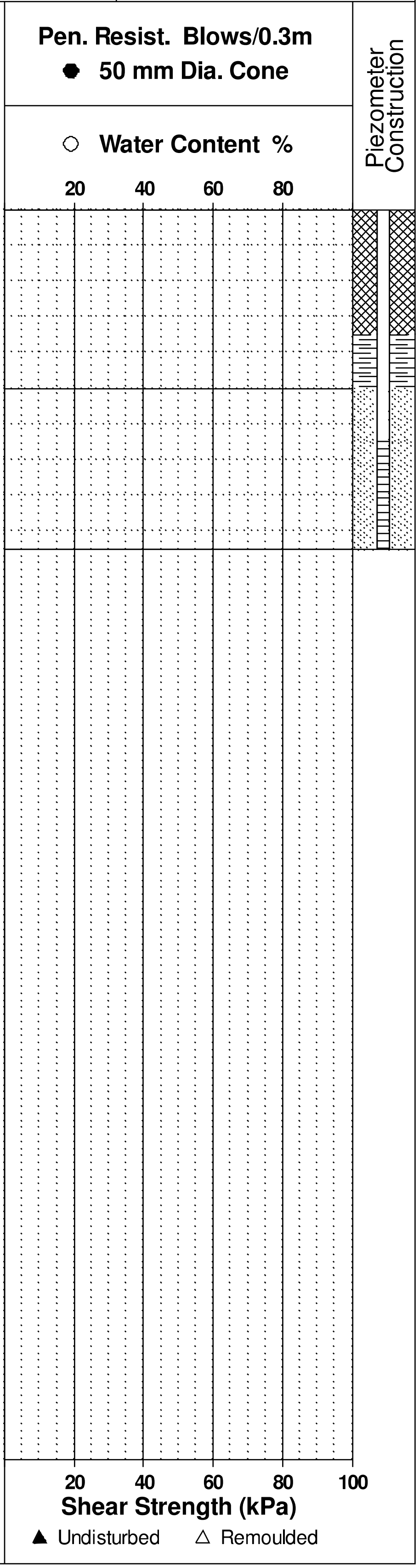
HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	68.65
FILL: Crushed stone 0.25	▩						
FILL: Brown silty sand, some crushed stone 0.60	▩	SS	1	42	23		
FILL: Brown silty sand 0.91	▩						
Dark brown **SILTY CLAY** with sand and topsoil 1.37	▨	SS	2	19	7	1	67.65
BEDROCK: Heavily fractured and weathered, black shale 1.90	▤	SS	3	67	50+		

End of Borehole

Practical refusal to augering @ 1.90m depth

(BH dry - Aug. 22/11)



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REMARKS

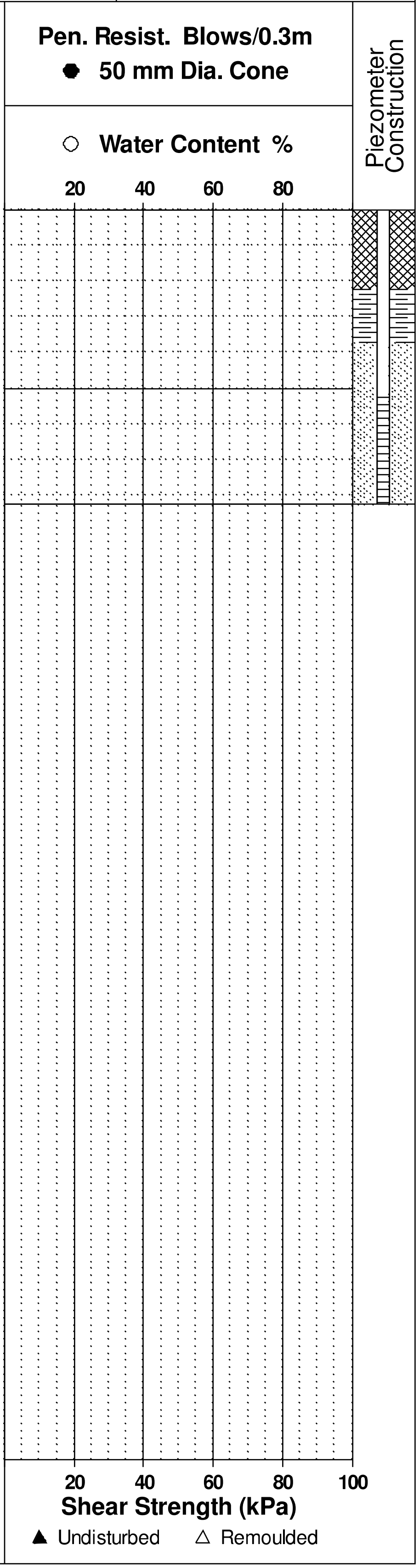
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.52
FILL: Crushed stone 0.25							
FILL: Brown silty sand with gravel 0.76		SS	1	46	41		
Brown **SILTY CLAY,** trace gravel and organics 1.50		SS	2	58	12	1	67.52
BEDROCK: Heavily fractured and weathered, black shale 1.65		SS	3	60	50+		
End of Borehole Practical refusal to augering @ 1.65m depth							



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REMARKS

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DATE August 17, 2011

FILE NO. PG2471

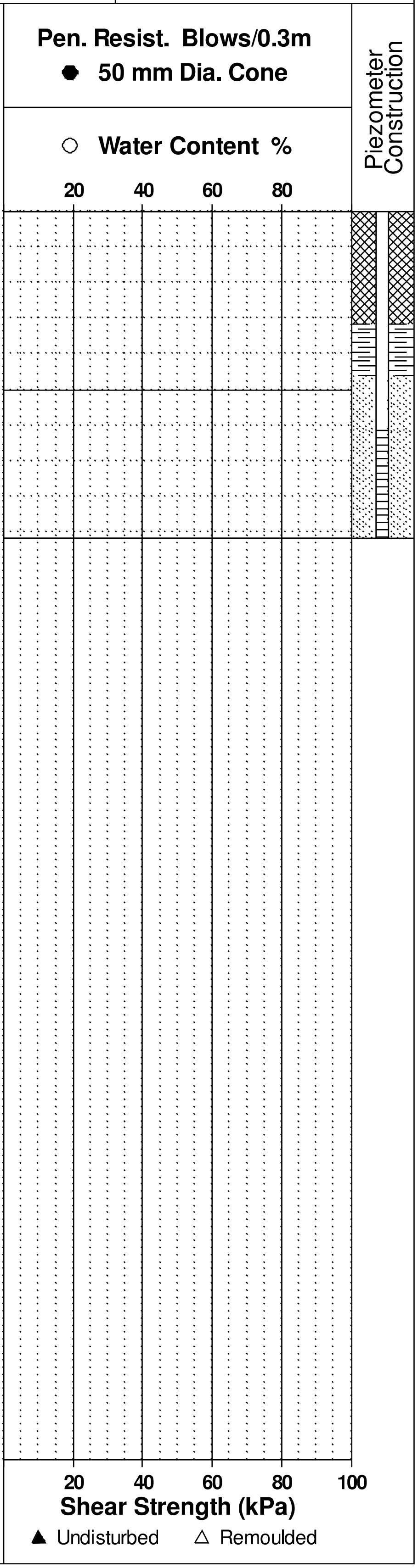
HOLE NO. BH11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.89
FILL: Crushed stone 0.15							
FILL: Brown silty sand with gravel and shale, trace clay 0.60		SS	1	50	18		
FILL: Brown silty sand with gravel 0.91							
TOPSOIL 1.22		SS	2	46	8	1	67.89
Brown **SILTY CLAY** with sand and roots 1.37							
BEDROCK: Heavily fractured and weathered, black shale 1.83		SS	3	50	50+		

End of Borehole

Practical refusal to augering @ 1.83m depth

(BH dry - Aug. 22/11)



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 55 Power Auger

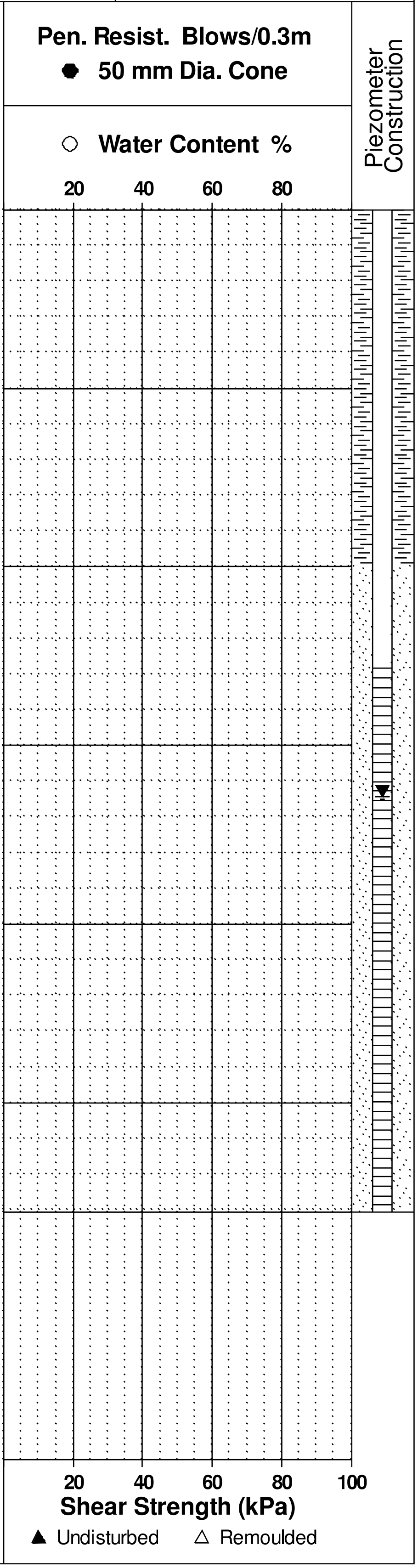
DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH12

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.15				
FILL: Brown silty sand with clay and shale 0.60	AU	1		
FILL: Brown silty sand with clay, gravel and shale, some organics 1.37	SS	2	42	7
BEDROCK: Heavily fractured and weathered, black shale 2.40	SS SS	3 4	96 100	38 50+
BEDROCK: Fractured, black shale 5.61	RC RC	1 2	70 87	21 42
End of Borehole (GWL @ 3.29m-Aug. 22/11)				

DEPTH (m)	ELEV. (m)
0	70.24
1	69.24
2	68.24
3	67.24
4	66.24
5	65.24



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

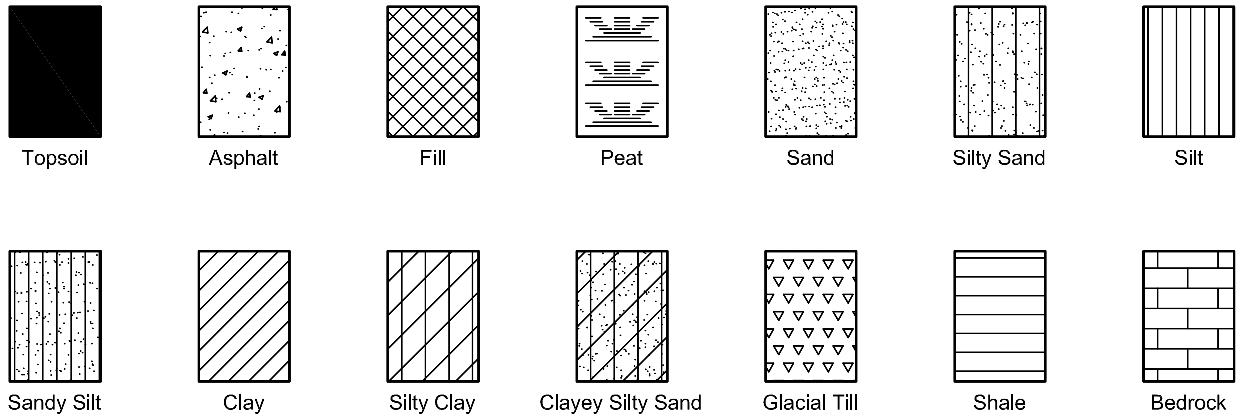
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

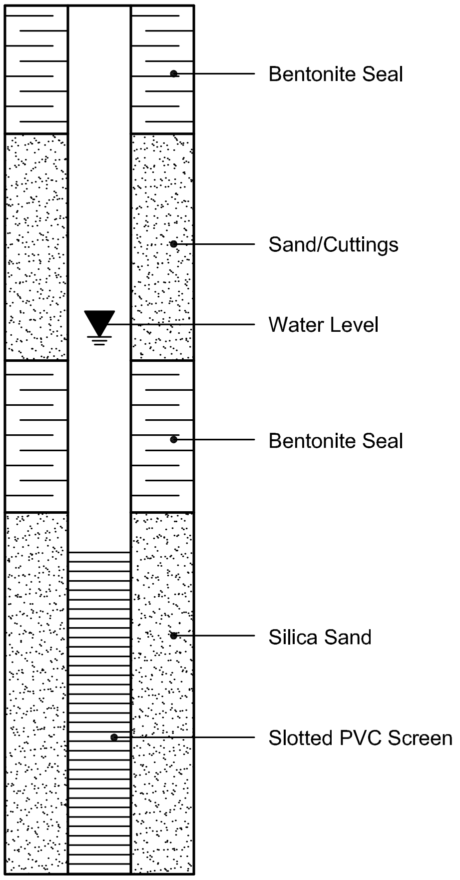
SYMBOLS AND TERMS (continued)

STRATA PLOT

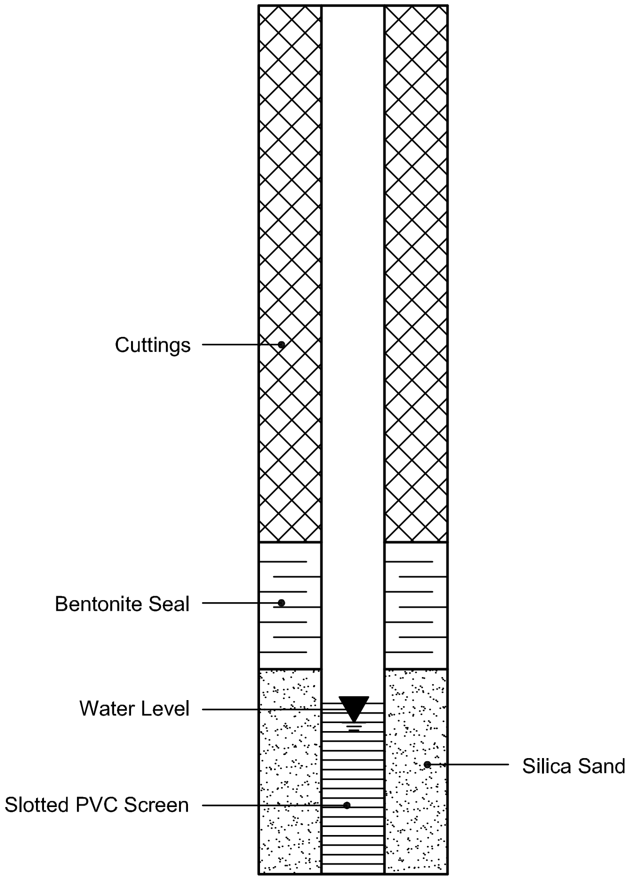


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6072-1 - TEST HOLE LOCATION PLAN

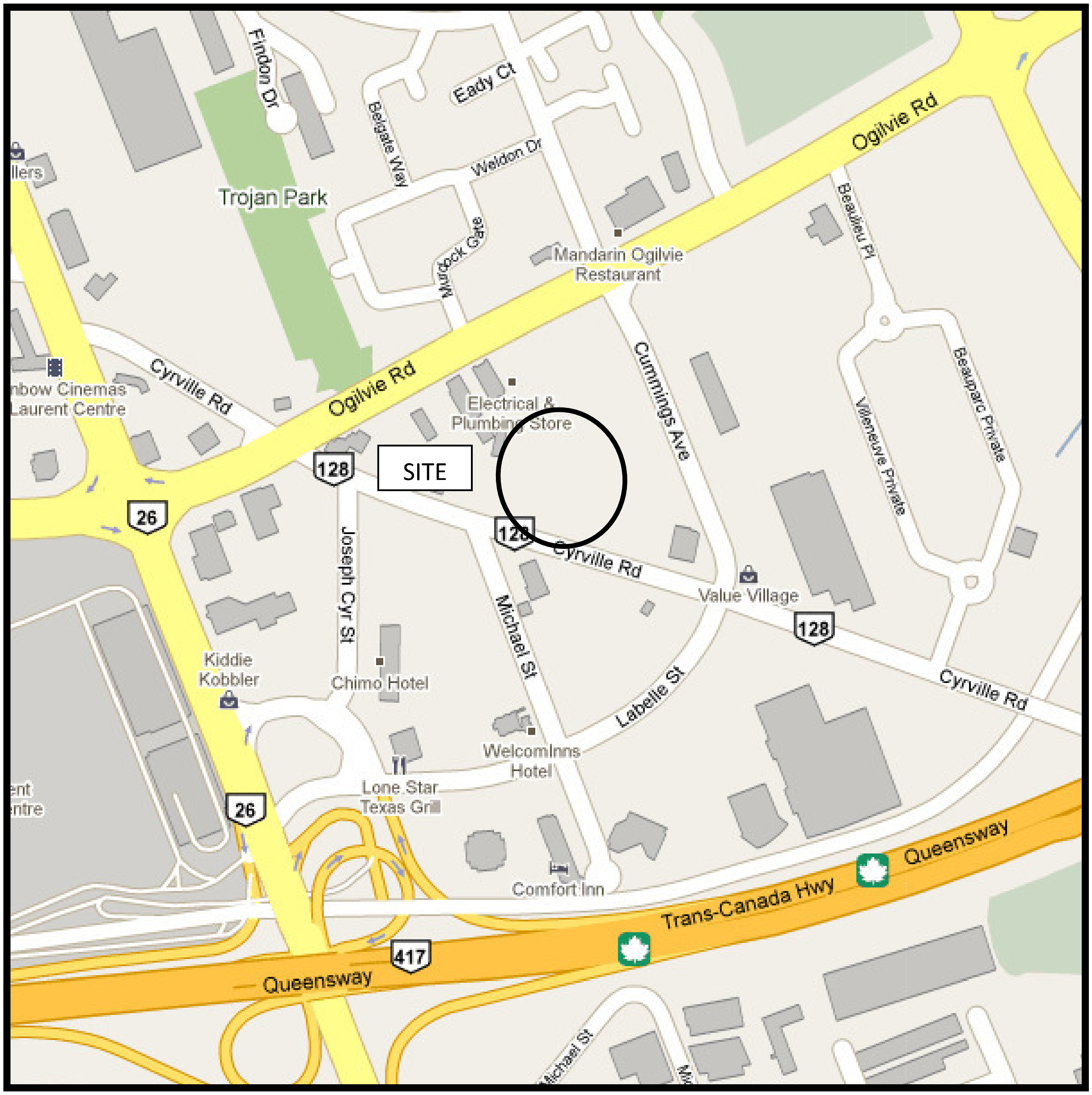


FIGURE 1

KEY PLAN

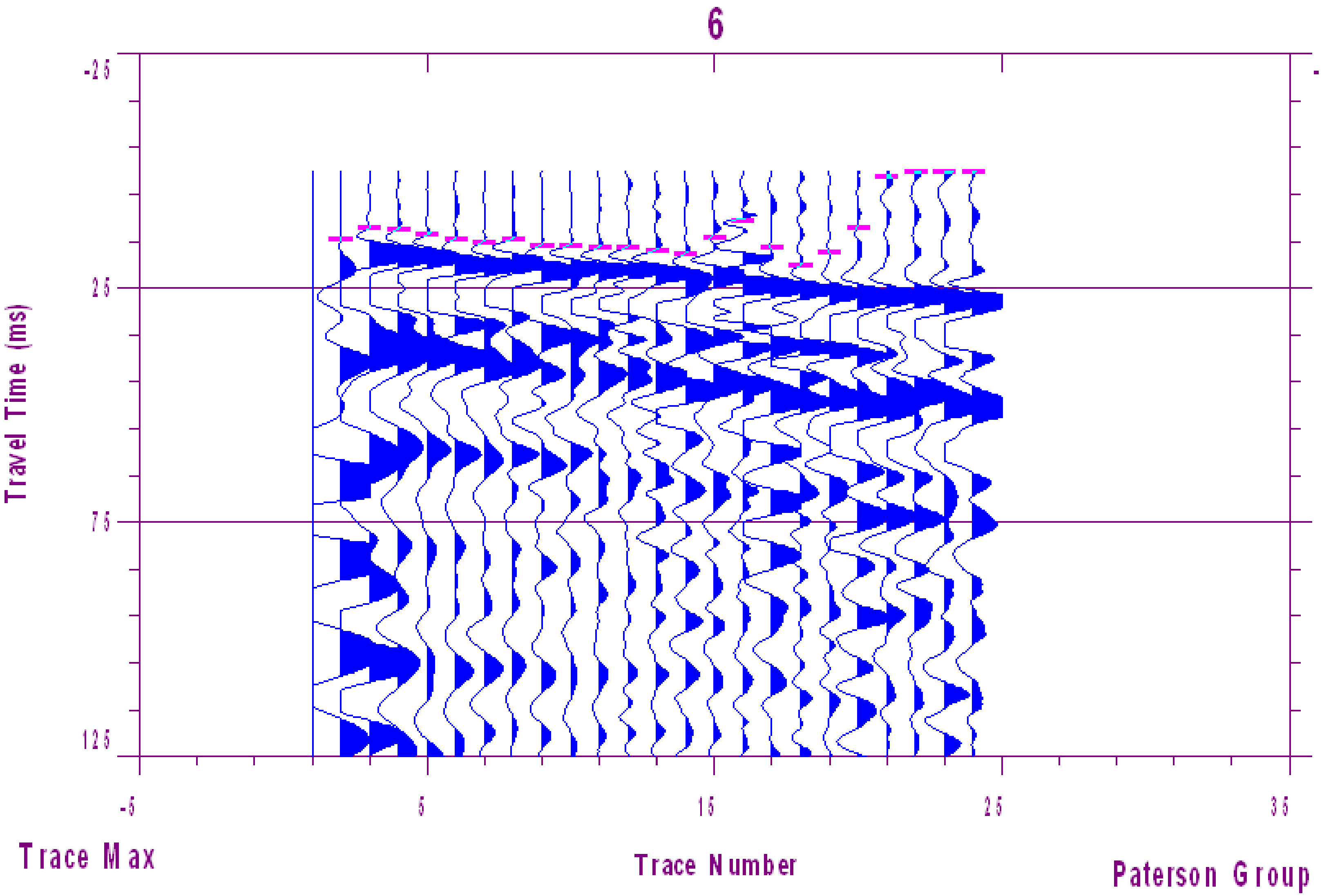


Figure 2 – Shear Wave Velocity Profile at Shot Location -1 m

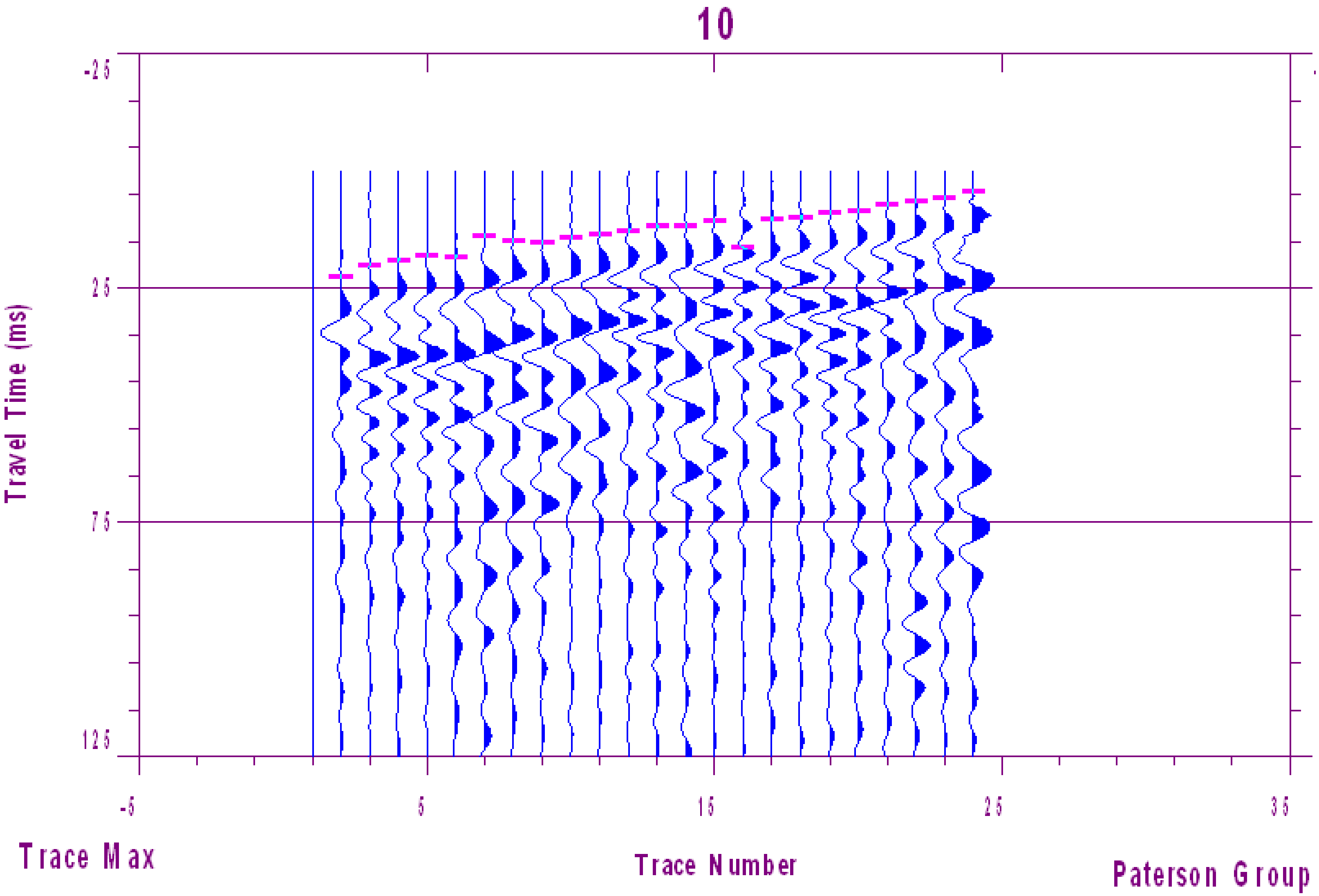
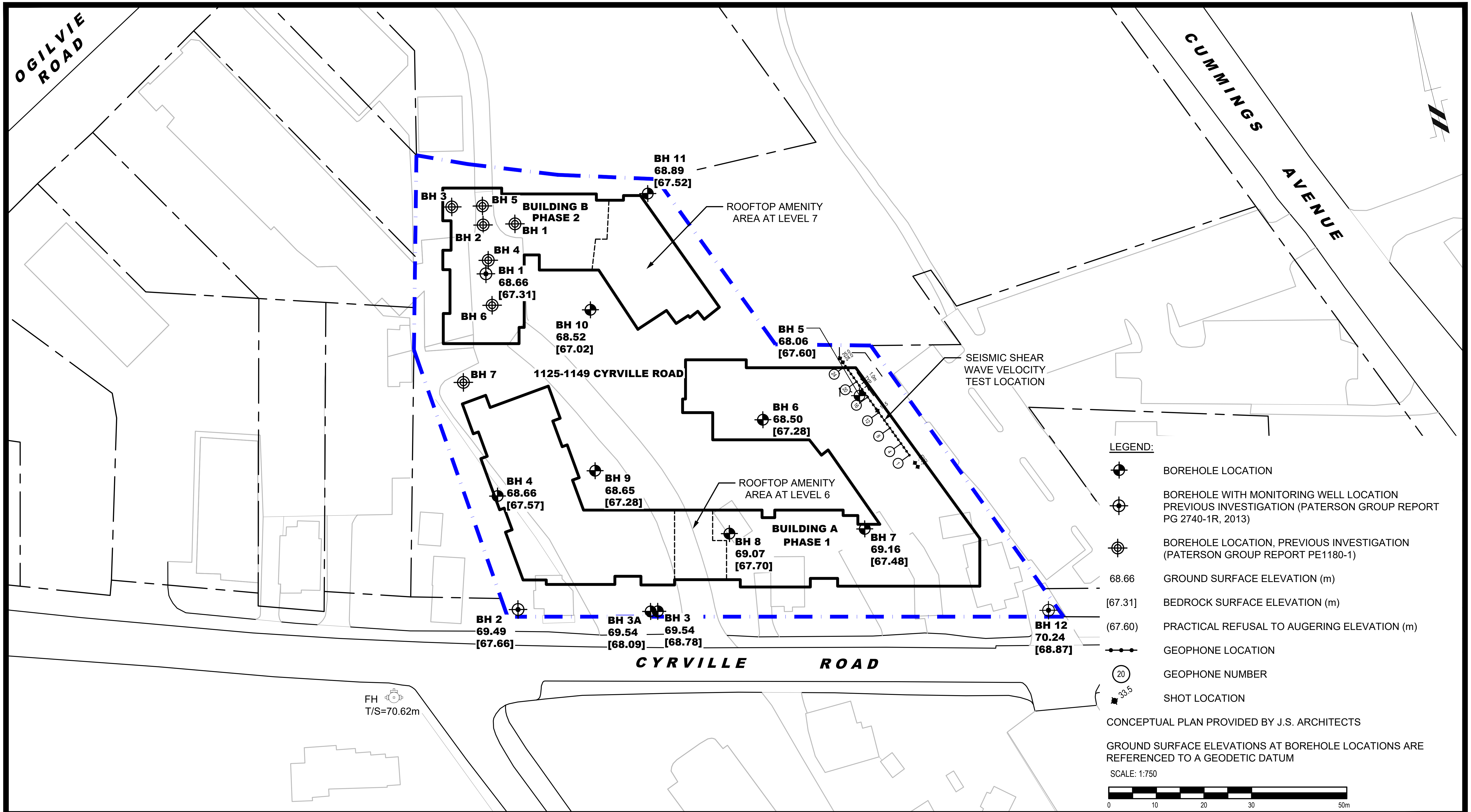


Figure 3 – Shear Wave Velocity Profile at Shot Location 24 m



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION PREVIOUS INVESTIGATION (PATERSON GROUP REPORT PG 2740-1R, 2013)
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PATERSON GROUP REPORT PE1180-1)
68.66	GROUND SURFACE ELEVATION (m)
[67.31]	BEDROCK SURFACE ELEVATION (m)
(67.60)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
•—•—•	GEOPHONE LOCATION
⑳	GEOPHONE NUMBER
✱ 33.5	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY J.S. ARCHITECTS

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:750

0 10 20 30 50m

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NO.	REVISIONS	DATE	INITIAL

WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125-1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 11/2021
Drawn by: YA	Report No.: PG6072-1
Checked by: OC	Dwg. No.: PG6072-1
Approved by: DJG	Revision No.:

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