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Geotechnical Investigation

Proposed Multi-Storey Building
333 Montreal Road
Ottawa, Ontario

Prepared For

The Salvation Army

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Report: PG3970-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by The Salvation Army to conduct a geotechnical investigation for the proposed multi-storey buildings to be located at 333 Montreal Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

☐ Obtain subsurface soil and groundwater information by means of boreholes completed within the subject site.

☐ Provide geotechnical recommendations for 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

Based on the conceptual drawings available at the time of issuance of this report, it is understood that two multi-storey buildings, up to 6 storeys high, and connected by a one storey link building are currently proposed. The proposed buildings include one basement level and access lanes, car parking areas and landscaped areas are also proposed.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program was completed on December 6 and 15, 2016. At that time, a total of 7 boreholes were selected in the field to provide general coverage of the proposed development taking into consideration of existing features, underground utilities and existing boreholes. The boreholes were advanced to a maximum of depth of 5.4 m below existing ground surface. The locations of the test holes are shown on Drawing PG3970-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using either a truck-mounted auger drill rig or portable drilling equipment operated by a two person crew. All fieldwork was conducted under the full-time supervision of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

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

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was carried out at BH 5 to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

Groundwater

A 32 mm diameter PVC groundwater monitoring well was installed at BH 5 and BH 7 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The remainder of the boreholes were outfitted with a piezometer.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ Slotted 32 mm diameter PVC screen at base of borehole, for 3 m length.
- ☐ 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 are stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.2 Field Survey

The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevation at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located on the corner of Ste. Anne Avenue and Montreal Road. A geodetic elevation of 60.42 m was provided for the TBM by Annis O'Sullivan Vollebekk.

The borehole locations and ground surface elevation at the borehole locations along with the TBM location are presented on Drawing PG3970-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the soil investigation results.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were analyzed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the sample. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

Currently, the perimeter of the subject site is occupied by a low rise motel building and the remainder of the subject site consists of an asphalt parking lot. The site is relatively flat and at grade with the neighbouring properties.

4.2 Subsurface Profile

Generally, the soil profile encountered at the borehole locations consists of a pavement structure and/or various fill material overlying a fill deposit. A narrow band of peat was noted in most boreholes. Below the peat, compact to very compact glacial till consisting of silty sand with gravel and shale was observed. Shallow bedrock consisting of shale was noted at depths ranging from 2 m to 5.4 m below the existing ground surface.

Based on available geological mapping, shale of the Billings Formation is present in this area with an overburden thickness ranging between 3 to 10 m.

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

4.3 Groundwater

A monitoring well was installed at BH 5 and BH 7. Piezometers were installed in all other boreholes. Groundwater level readings were taken by Paterson personnel on December 15, 2016. The groundwater level (GWL) readings are presented in Table 1. It should be noted that water can become perched within a backfilled borehole, which can lead to higher than normal groundwater readings within piezometer tubing. Also, groundwater levels are subject to seasonal fluctuations. Therefore, groundwater levels could vary at the time of construction.

Table 1 - Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	89.96	4.11	85.85	December 15, 2016
BH 2	59.99	5.39	54.60	December 15, 2016
BH 3	60.39	Damaged	n/a	December 15, 2016
BH 4	60.36	1.37	58.99	December 15, 2016
BH 5	60.15	2.68	57.47	December 15, 2016
BH 6	60.17	3.00*	57.17	December 15, 2016

* - indicates an open hole groundwater level noted at time of drilling

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey buildings. The proposed building is expected to be founded on conventional footings placed on compact glacial till or a clean, surface sounded bedrock.

Bedrock removal may be required to complete the basement level. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as organic materials, should be stripped from under the proposed buildings and other settlement sensitive structures.

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed buildings, all existing overburden material should be excavated from within the proposed building footprint.

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 buildings and paved areas 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.

Excavated shale deteriorates upon exposure to air and is not suitable for placement as an engineered fill. Paterson recommends pouring a mud slab over the exposed bearing surface within 48 hours of exposure.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the basement level. 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 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 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.

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.

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

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

5.3 Foundation Design

Bearing Resistance Values

Footings placed over an undisturbed, compact glacial till bearing surface can be designed using a bearing resistance value at SLS of **200 kPa** and a factored bearing resistance value at ULS to **350 kPa**. Footings placed on a clean, surface sounded bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa** incorporating a geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS. Where bedrock is not encountered at the footing level, consideration could be given to excavating a near vertical trench down to the bedrock surface and backfill to underside of footing level with a minimum 20 MPa lean concrete.

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.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

5.4 Design for Earthquakes

A site specific shear wave velocity test was completed to accurately determine the applicable seismic site classification for foundation design of the proposed building as presented in Table 4.1.8.4.A of the Ontario Building Code 2012. A seismic shear wave velocity test was completed by Paterson at the subject site. Two shear wave velocity profiles are presented in Appendix 2.

Field Program

The shear wave test location is presented in Drawing PG3970-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel installed 24 horizontal geophones in a straight line oriented roughly in a north-south direction along the west site boundary. 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 connected to a computer and a trigger switch attached to a 12 pound dead blow hammer. The hammer trigger sends a signal to the seismograph to commence recording. The hammer strikes an I-Beam seated into the ground surface, which produces a polarized shear wave. The shots are repeated between four to eight times at each shot location to provide an accurate signal and reduce noise. 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 were distributed at the centre of the geophone array and 1, 3 and 16.5 m away from the first and last geophone.

The test method completed by Paterson are guided by the standard test procedures outlined by 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. The shear wave velocity measurement was calculated by the 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, V_{s30}, immediately below the proposed building foundation of the upper 30 m profile. To compute the bedrock depth at each location, the layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave graphs. 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 increases.

Based on our analysis, the bedrock seismic shear wave velocity was calculated to be between 2,500 m/s. The V_{s30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012, as presented below;

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

$$V_{s30} = \frac{30m}{\left(\frac{0m}{205m/s} + \frac{30m}{1{,}857m/s}\right)}$$

$$V_{s30} = 1{,}875m/s$$

Based on the seismic results, the average shear wave velocity, V_{s30}, for shallow foundations located at the subject site placed directly on the bedrock surface is 1,875 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building at the subject site, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil, within the building footprint, should be removed from the subject site and founded on a bedrock surface. An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of the SPMDD could be placed around the proposed footings. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone.

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

5.6 Basement Wall

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Rock Anchor Design

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

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

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

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

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

Grout to Rock Bond

Generally, the unconfined compressive strength of the shale bedrock ranges between 40 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1,000 kPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

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

For calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair Quality Shale Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

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

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	3	1.5	4.5	250
	4.2	2.2	6.4	500
	6.5	2.6	9.1	1000
	10	3.5	13.5	2000
125	2.8	1.5	4.3	250
	3.5	2.4	5.9	500
	5.5	2.8	8.3	1000
	8	3.8	11.8	2000

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and flushed clean with water prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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

5.8 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and access lanes, if required.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

Table 5 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

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 II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 6 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres below the basement level. 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

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, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Adverse Effects of Dewatering on Adjacent Properties

The proposed building will be founded above the long-term groundwater level. Therefore, no adverse effects to the surrounding buildings or properties are expected with the lowering of the groundwater in this 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.

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.

6.3 Excavation Side Slopes

The excavations for the proposed development will be through fill or a native glacial till material. The subsurface soil is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. For excavations to depths of approximately 3 m, above the groundwater level, the excavation side slopes should be stable in the short term at 1H:1V. Shallower slopes should be provided for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be installed.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be maintain safe working distance 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.

A trench box is recommended to be installed at all times to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the SPMDD. The bedding material should extend at least 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 95% of the 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 SPMDD.

6.5 Groundwater Control

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

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

For typical ground or surface water volumes being pumped during the construction phase, 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, and EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

Impacts on Neighbouring Properties

Based on the proximity of neighbouring buildings. the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions should be considered if construction occurs during the winter. The subsurface soil conditions 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. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until 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 difficult activities to complete during winter without introducing frost in the excavation subgrade base or walls. Precautions should be considered if such activities are to be completed during sub-zero temperatures.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive corrosive environment.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

☐ Review of the geotechnical aspects of the excavation contractor’s 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.

☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

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 that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than The Salvation Army or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Stephanie A. Boisvenue, P.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Jan. 9-2017
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

FILE NO. PG3970

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger

DATE December 6, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	59.21		
Asphaltic concrete 0.10									
FILL: Crushed stone 0.60		AU	1						
FILL: Brown silty sand 1.52		SS	2	62	10	1	58.21		
FILL: Crushed stone 1.83		SS	3	67	16				
FILL: Grey silty clay, trace gravel 2.34						2	57.21		
PEAT 2.49									
GLACIAL TILL: Dark brown sand and gravel with shale		SS	4	33	5				
						3	56.21		
		SS	5	100	18				
4.11		SS	6	100	50+	4	55.21		

End of Borehole

Inferred bedrock depth at 4.11m depth

(BH dry - Dec. 15, 2016)

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

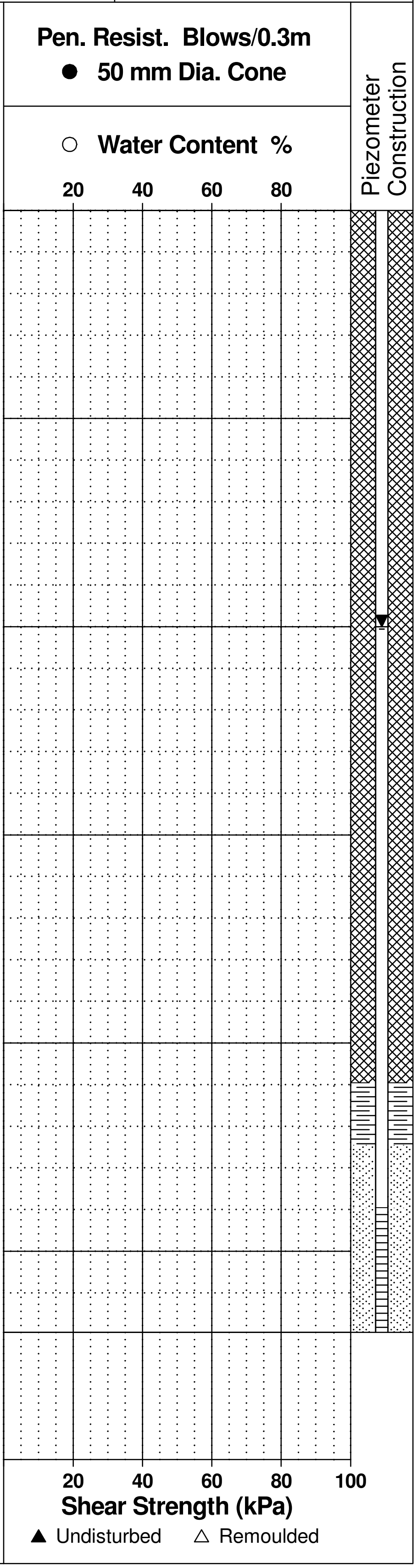
BORINGS BY CME 55 Power Auger

DATE December 6, 2016

FILE NO. PG3970

HOLE NO. BH 2

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	59.24
Asphaltic concrete	0.05						
FILL: Crushed stone	0.60	AU	1				
FILL: Dark brown sandy silt, some gravel, wood and concrete, trace brick	1.65	SS	2	54	5	1	58.24
PEAT	1.83	SS	3	75	11		
						2	57.24
		SS	4	50	31		
						3	56.24
GLACIAL TILL: Brown silty clay with sand, gravel and shale		SS	5	100	69		
		SS	6	100	26	4	55.24
		SS	7	100	63	5	54.24
(GWL @ 2.00m - Dec. 15, 2016)	5.39	SS	8	100	50+		
End of Borehole Inferred bedrock depth at 5.39m depth							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

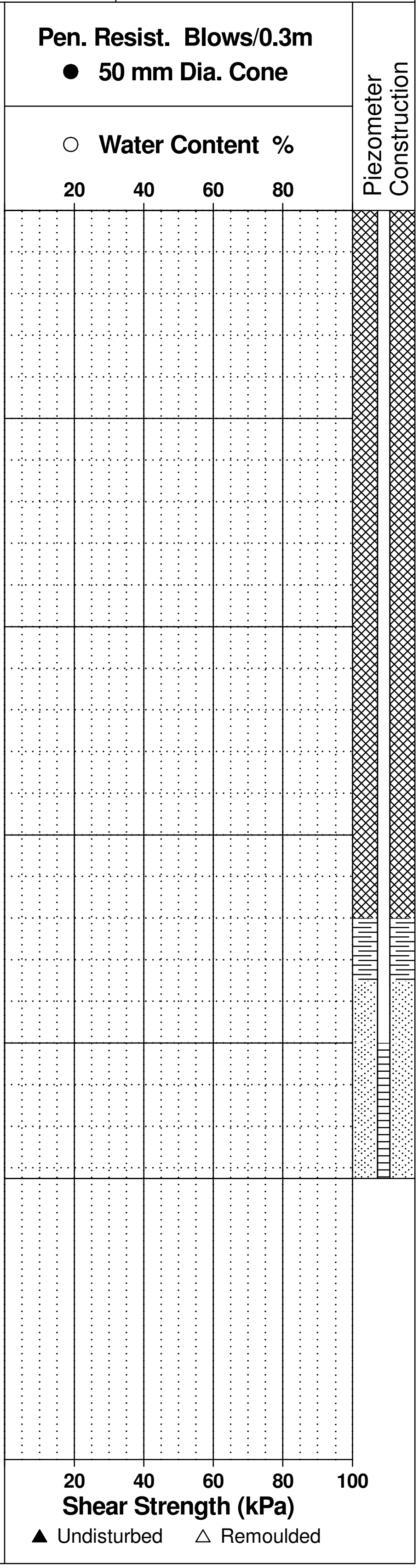
FILE NO. PG3970

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger

DATE December 6, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	59.64
Asphaltic concrete 0.05						
FILL: Crushed stone 0.20						
FILL: Brown sand, some concrete and brick	AU	1				
	SS	2	42	8	1	58.64
1.17						
PEAT 1.37						
GLACIAL TILL: Dark brown silty clay with shale, trace sand	SS	3	50	11		
					2	57.64
	SS	4	83	35		
					3	56.64
	SS	5	58	41		
3.81						
BEDROCK: Weathered black shale	SS	6	80	50+		
					4	55.64
4.65	SS	7	50	50+		
End of Borehole (Piezometer damaged - Dec. 15, 2016)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 6, 2016

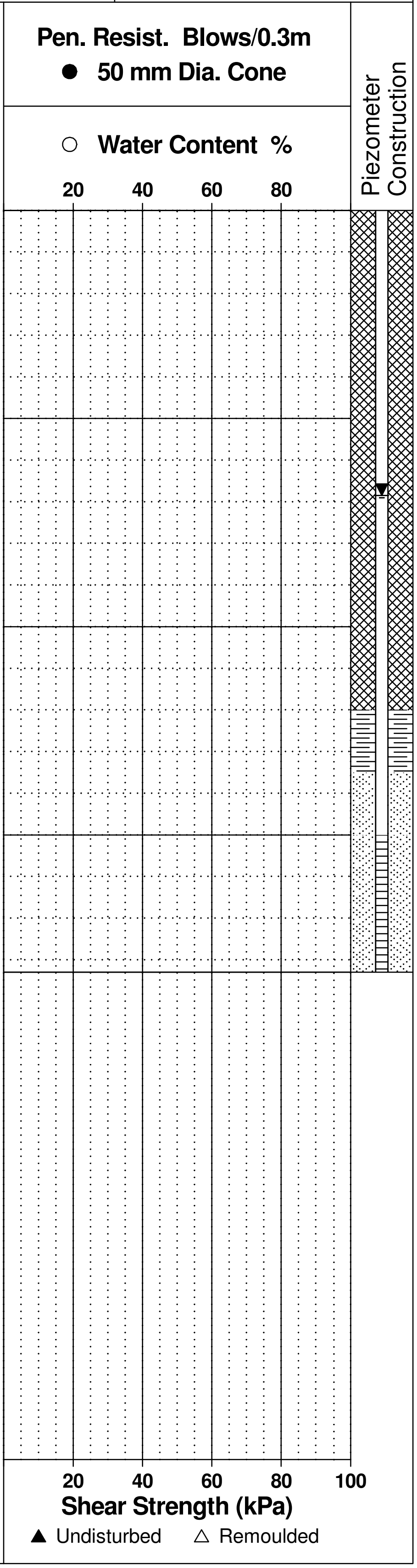
FILE NO. PG3970

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone	0.15
FILL: Dark brown sand, some gravel, trace wood and brick	1.42
GLACIAL TILL: Dark brown clayey silt, some shale and gravel, occasional boulders	3.56
Inferred shale **BEDROCK**	3.66
End of Borehole	
(GWL @ 1.37m - Dec. 15, 2016)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	25	8
SS	3	92	16
SS	4	83	31
SS	5	90	27

DEPTH (m)	ELEV. (m)
0	59.61
1	58.61
2	57.61
3	56.61



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY Portable Drill

DATE December 15, 2016

FILE NO. PG3970

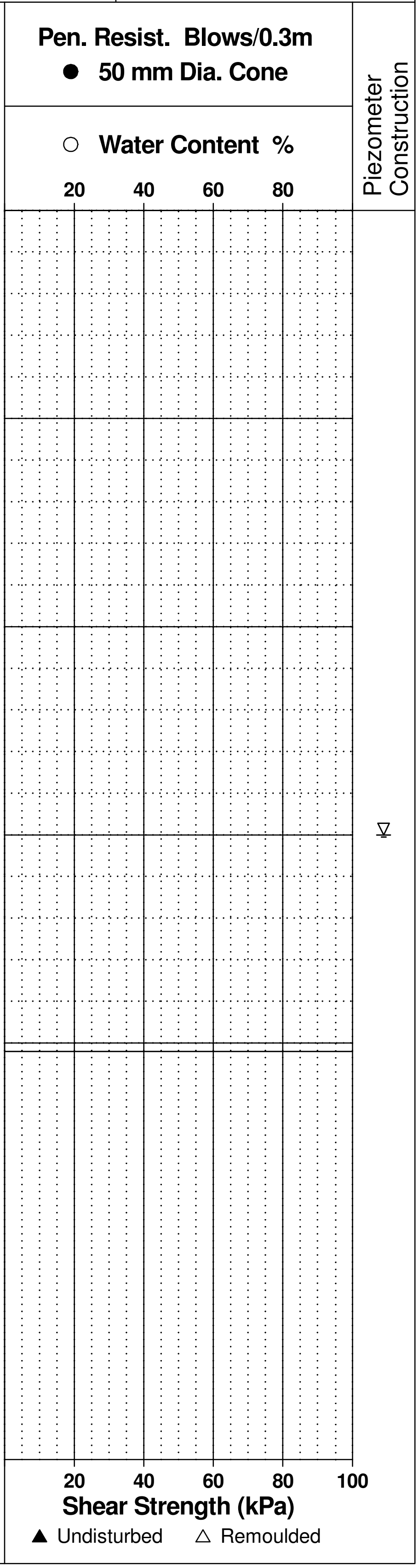
HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	59.42
TOPSOIL 0.25						
	SS	1	67			
	SS	2	42			
					1	58.42
	SS	3	50			
FILL: Brown fine to medium sand						
					2	57.42
	SS	4	42			
	SS	5	58			
					3	56.42
	SS	6	67			
3.83						
GLACIAL TILL: Brown silty clay with shale, trace sand and gravel 4.04	SS	7	100			
					4	55.42

End of Borehole

Inferred bedrock depth at 4.04m depth

(GWL @ 3.0m depth based on field observations)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 6, 2016

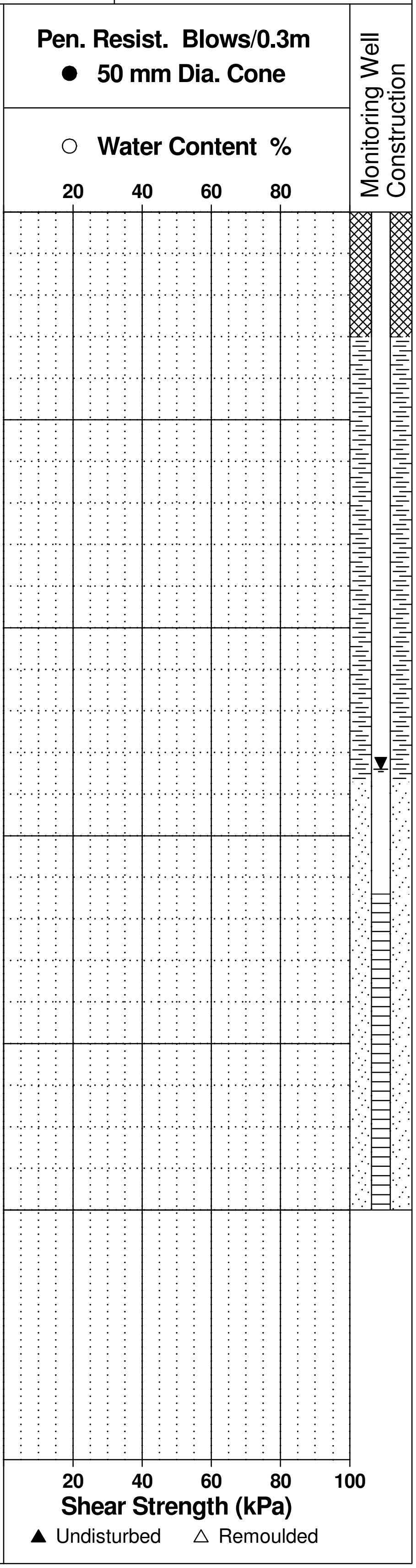
FILE NO. PG3970

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Crushed stone	0.41	AU	1		
FILL: Brown silty sand	0.76				
FILL: Dark brown silty clay, some shale, gravel, wood, glass and concrete	1.83	SS	2	79	13
		SS	3	100	9
PEAT	1.96				
BEDROCK: Black shale	4.80	SS	4	0	50+
		RC	1	33	0
		RC	2	100	40
End of Borehole					

(GWL @ 2.68m - Dec. 15, 2016)

DEPTH (m)	ELEV. (m)
0	59.40
1	58.40
2	57.40
3	56.40
4	55.40



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 333 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Ste. Anne Ave. and Montreal Road. Geodetic elevation = 60.42m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

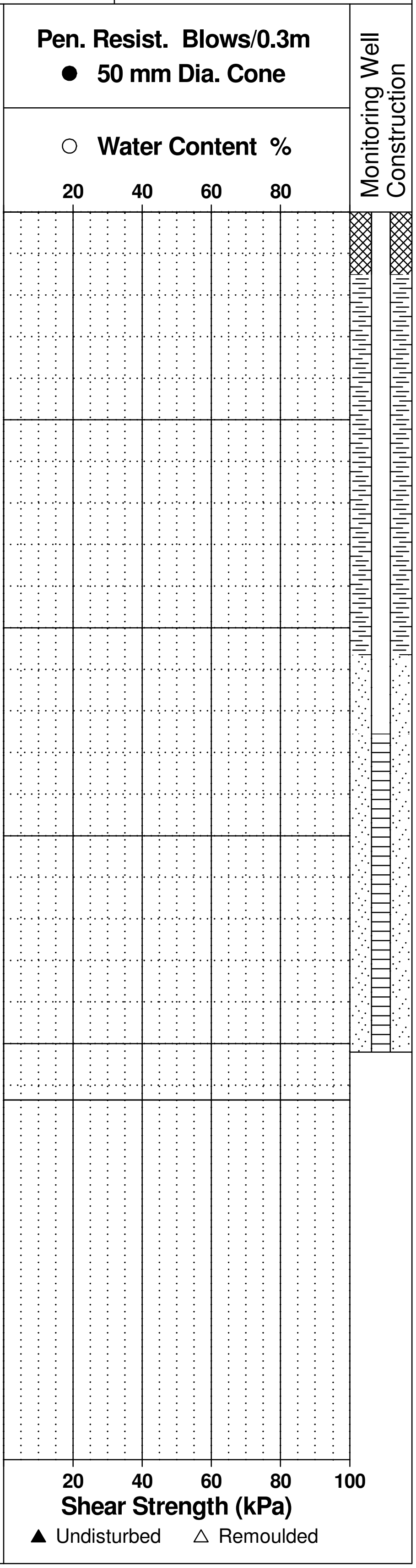
BORINGS BY Portable Drill

DATE December 15, 2016

FILE NO. PG3970

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	58.85
Asphaltic concrete 0.15							
FILL: Brown silty sand, some crushed stone and shale 0.71		SS	1	67			
FILL: Brown silty sand, some clay, trace gravel and crushed stone 1.98		SS SS	2 3	75 71		1	57.85
TOPSOIL and PEAT 3.05		SS SS	4 5	83 100		2	56.85
						3	55.85
GLACIAL TILL: Dark grey silty sand with gravel and shale, trace cobbles 4.27		SS	6	75		4	54.85
End of Borehole							



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)

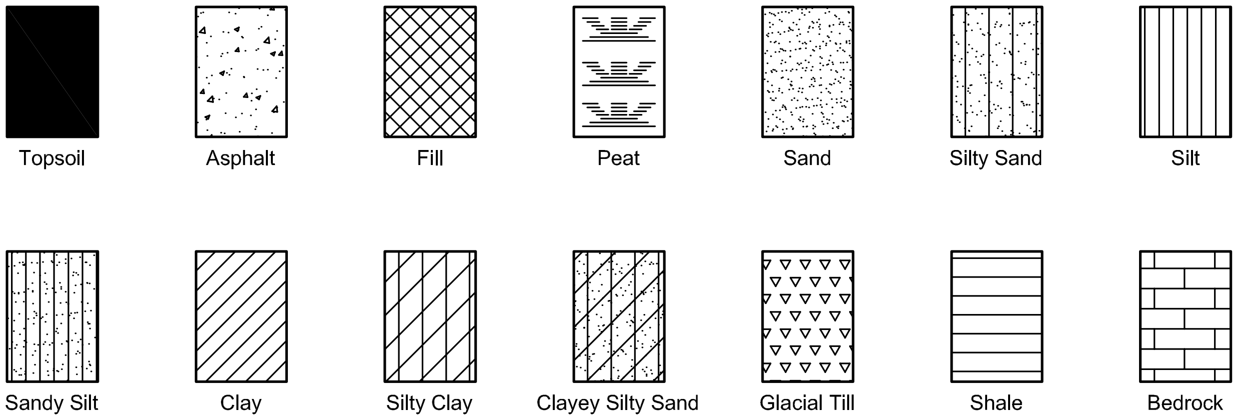
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

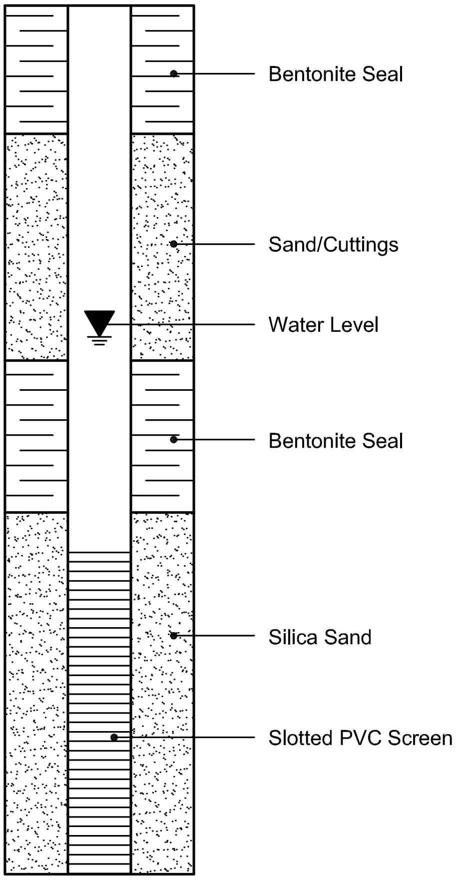
SYMBOLS AND TERMS (continued)

STRATA PLOT

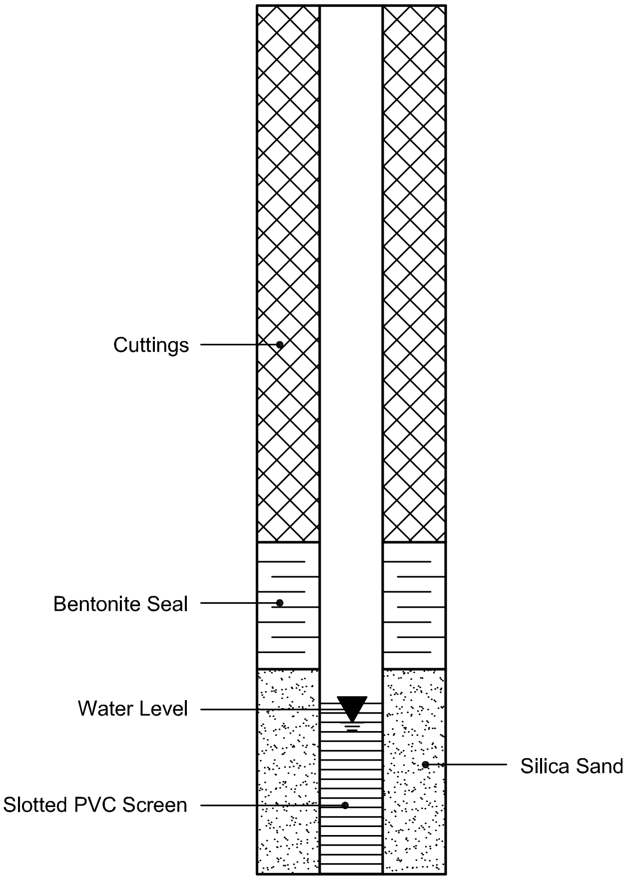


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



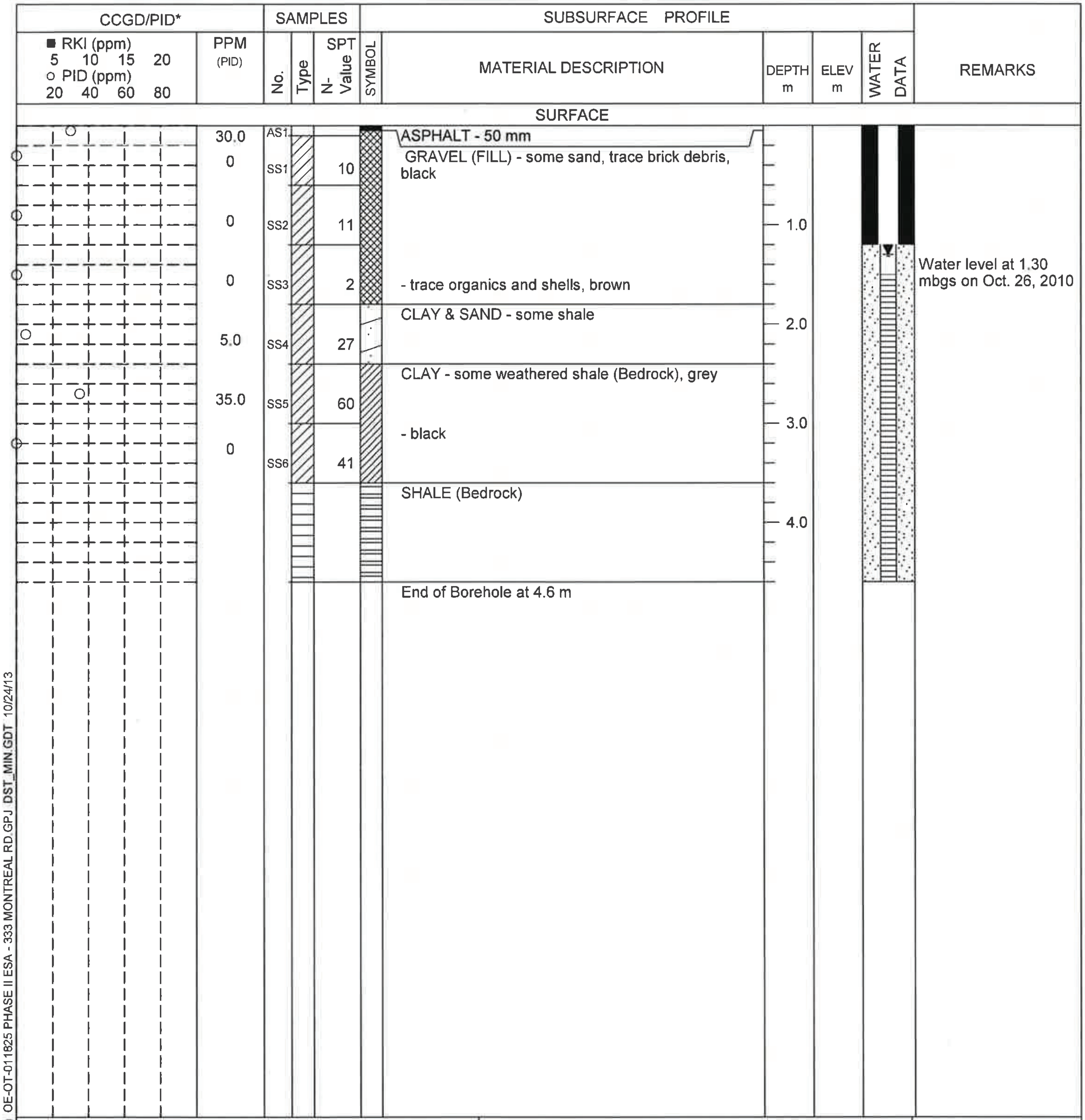
PIEZOMETER CONSTRUCTION



LOG OF BOREHOLE BHMW1-2010

DST REF. No.: **OE-OT-011825**
CLIENT: **Concorde Motel**
PROJECT: **Phase II ESA**
LOCATION: **333 Montreal Rd, Ottawa, Ontario**

Drilling Data
METHOD: **CCME 75**
DIAMETER: **50.8 mm**
DATE: **October 21, 2010**



PPM (PID)	No.	Type	SPT N-Value	MATERIAL DESCRIPTION	DEPTH m	REMARKS
30.0	AS1			ASPHALT - 50 mm		
0	SS1	▨	10	GRAVEL (FILL) - some sand, trace brick debris, black		
0	SS2	▨	11		1.0	
0	SS3	▨	2	- trace organics and shells, brown		Water level at 1.30 mbgs on Oct. 26, 2010
5.0	SS4	▨	27	CLAY & SAND - some shale	2.0	
35.0	SS5	▨	60	CLAY - some weathered shale (Bedrock), grey		
				- black	3.0	
0	SS6	▨	41			
				SHALE (Bedrock)	4.0	
				End of Borehole at 4.6 m		

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OTTAWA, ONTARIO, K1G 5T9
PH: (613)748-1415
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Web: www.dstgroup.com

* Catalytic Combustible Gas Detector / Photo Ionization Detector

SAMPLE TYPE LEGEND

- ☐ Auger Sample
- ▨ Split Spoon Sample
- ▯ 70mm Thin Wall Tube
- ▩ Rock Core
- ▧ Side Sampler
- ░ Sand
- ■ Bentonite

APPENDIX B

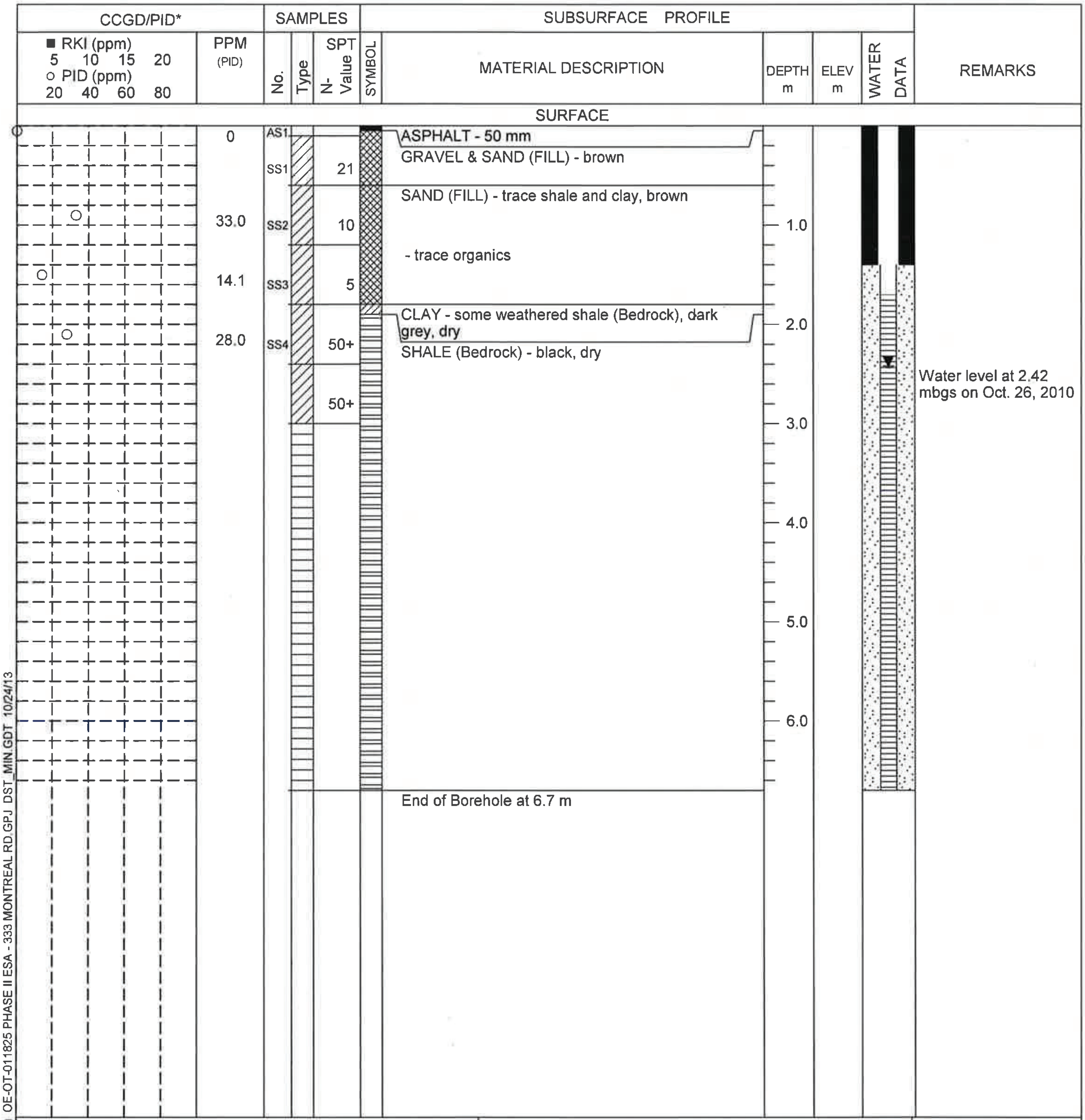
PAGE 1 OF 1

GASTECBH (OTTAWA) OE-OT-011825 PHASE II ESA - 333 MONTREAL RD.GPJ DST_MIN.GDT 10/24/13

LOG OF BOREHOLE BHMW2-2010

DST REF. No.: **OE-OT-011825**
CLIENT: **Concorde Motel**
PROJECT: **Phase II ESA**
LOCATION: **333 Montreal Rd, Ottawa, Ontario**

Drilling Data
METHOD: **CCME 75**
DIAMETER: **50.8 mm**
DATE: **October 21, 2010**



PPM (PID)	No.	SPT N-Value	MATERIAL DESCRIPTION	REMARKS
0	AS1		ASPHALT - 50 mm	
	SS1	21	GRAVEL & SAND (FILL) - brown	
33.0	SS2	10	SAND (FILL) - trace shale and clay, brown	
14.1	SS3	5	- trace organics	
28.0	SS4	50+	CLAY - some weathered shale (Bedrock), dark grey, dry SHALE (Bedrock) - black, dry	Water level at 2.42 mbgs on Oct. 26, 2010
		50+		
			End of Borehole at 6.7 m	

GASTECBH (OTTAWA) OE-OT-011825 PHASE II ESA - 333 MONTREAL RD.GPJ DST_MIN.GDT 10/24/13

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* Catalytic Combustible Gas Detector / Photo Ionization Detector

SAMPLE TYPE LEGEND

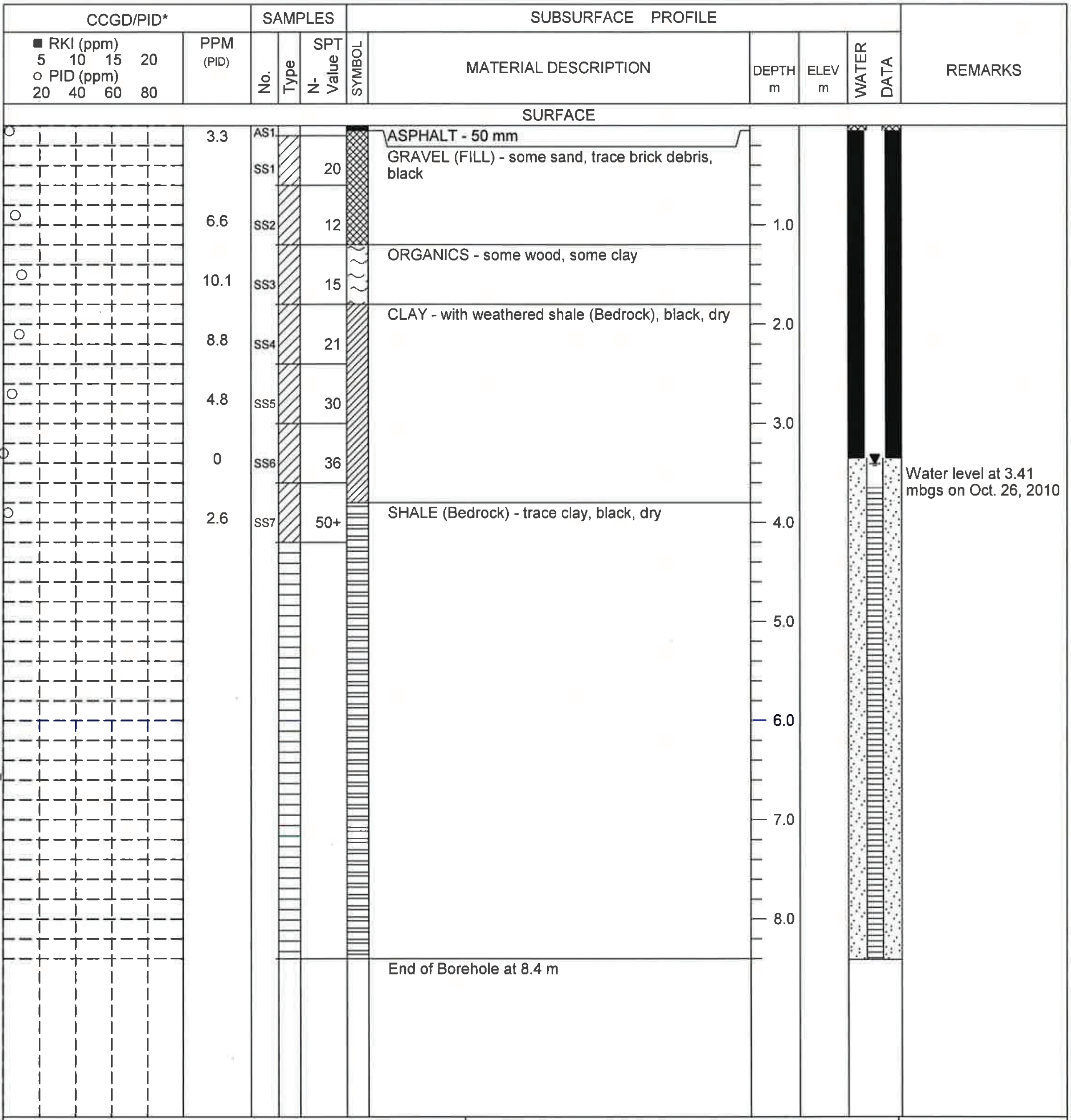
▤	Auger Sample	▩	Rock Core	⁘	Sand
▨	Split Spoon Sample	▧	Side Sampler	■	Bentonite
▯	70mm Thin Wall Tube				

APPENDIX B

LOG OF BOREHOLE BHMW3-2010

DST REF. No.: **OE-OT-011825**
CLIENT: **Concorde Motel**
PROJECT: **Phase II ESA**
LOCATION: **333 Montreal Rd, Ottawa, Ontario**

Drilling Data
METHOD: **CCME 75**
DIAMETER: **50.8 mm**
DATE: **October 21, 2010**



PPM (PID)	No.	Type	SPT N-Value	MATERIAL DESCRIPTION	DEPTH m	ELEV m	REMARKS
3.3	AS1			ASPHALT - 50 mm			
	SS1		20	GRAVEL (FILL) - some sand, trace brick debris, black			
6.6	SS2		12		1.0		
10.1	SS3		15	ORGANICS - some wood, some clay			
8.8	SS4		21	CLAY - with weathered shale (Bedrock), black, dry	2.0		
4.8	SS5		30		3.0		
0	SS6		36				Water level at 3.41 mbgs on Oct. 26, 2010
2.6	SS7		50+	SHALE (Bedrock) - trace clay, black, dry	4.0		
				End of Borehole at 8.4 m			

GASTECBH (OTTAWA) OE-OT-011825 PHASE II ESA - 333 MONTREAL RD.GPJ DST_MIN.GDT 10/24/13

DST consulting engineers

DST Consulting Engineers Inc.
203 - 2150 THURSTON DRIVE
OTTAWA, ONTARIO, K1G 5T9
PH: (613)748-1415
FX: (613)748-1356
Email: ottawa@dstgroup.com
Web: www.dstgroup.com

* Catalytic Combustible Gas Detector / Photo Ionization Detector

SAMPLE TYPE LEGEND

☰	Auger Sample	▦	Rock Core	░	Sand
▨	Split Spoon Sample	▧	Side Sampler	■	Bentonite
☐	70mm Thin Wall Tube				

APPENDIX B

LOG OF BOREHOLE BH2

DST REF. No.: **OE03761**	<u>Drilling Data</u>
CLIENT: **Concorde Motel, 333 Montreal Road.**	METHOD: **Mobile B47**
PROJECT: **Limited Phase II ESA**	DIAMETER: **140 mm**
LOCATION: **Ottawa, Ontario**	
SURFACE ELEV.: **--/--**	DATE: **December 08 2003**

CCGD *		SAMPLES			SUBSURFACE PROFILE					
○ PPM 40 80 120 160 ▪ % LEL 20 40 60 80	PPM	No.	Type	SPT N-Value	SYMBL	MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						ASPHALT				
					▓	FILL - sand and gravel, compact, dark brown				
○	10	SS1	▨	20	▓	FILL - sand, silty, compact, light brown				
							1			
○	10	SS2	▨	20	▓	FILL - clay, silty, gravely, trace brick, compact, dark grey				
○	5	SS3	▨	16	▨	LIMESTONE - possible bedrock, grey	2			
						End of borehole at 2.1 m depth due to auger refusal.				

DST Consulting Engineers Inc.
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Ottawa, Ontario, K1B 3W8
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Email: ottawa@dstgroup.com

* - Catalytic Combustible Gas Detector

SAMPLE TYPE LEGEND

☐ Auger Sample	▓ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
☐ Thin Wall Tube	☰ Grab Sample	

ATTACHMENT 2

PROJECT: 971-2653

RECORD OF BOREHOLE 97-1

SHEET 1 OF 1

LOCATION: See Plan

BORING DATE: May 21, 1997

DATUM: Local

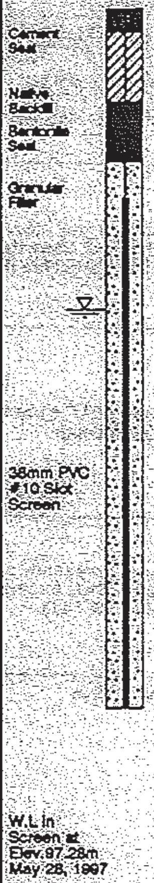
DIP:

SAMPLER HAMMER, 63.5kg; DROP, 760 mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	RECOVERY %	LAB. TESTING	COMBUSTIBLE VAPOUR ( ) ● LEL % ☐	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl
		Asphalt Surface		99.21							
0	Power Auger 200mm Diam (Hollow Stem)	ASPHALTIC CONCRETE		0.00							
		Granular A (FILL)		0.18							
		Loose brown medium to coarse sand and gravel (FILL)		0.34							
1 2		Loose brown to grey brown sandy silt, gravel, occasional cobble (FILL)		98.33 0.88	1 2	50 DO 50 DO	4 10				
3		Crushed limestone (FILL)		96.92 2.29	3 4	50 DO 50 DO	5 4				
4		Very dense grey sandy silt, gravel, shale fragments (GLACIAL TILL)		95.68 3.53	5	50 DO	74				
		Black weathered SHALE		94.97 4.24							
5 6 7 8 9 10		End of Hole		94.64 4.57							

INSTALLATIONS

A B C



DATA INPUT: C:97-1-863.01/B.L

DEPTH SCALE (ALONG HOLE)

1 to 50

Golder Associates

LOGGED: H.E.C.

CHECKED: BS

PROJECT: 971-2863

RECORD OF BOREHOLE 97-2

SHEET 1 OF 1

LOCATION: See Plan

BORING DATE: May 21, 1997

DATUM: Local

DIP:

SAMPLER HAMMER, 63.5kg; DROP, 760 mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES					COMBUSTIBLE VAPOUR ● ( ) / LEL %	HYDRAULIC CONDUCTIVITY, k, cm/s / ☐ WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl	INSTALLATIONS A B C
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	RECOVERY %	LAB. TESTING			
		Asphalt Surface		99.26								
0	Power Auger 200mm Diam (Hollow Stem)	ASPHALTIC CONCRETE		0.00 99.06								
		Granular A (FILL)		0.20								
1		Compact grey brown sandy silt with gravel, trace clay, occasional cobble (FILL)		98.32 0.94	1	50 DO	11					
					2	50 DO	16					
2		Brown PEAT, trace to some shells at lower depth		97.25 2.01	3	50 DO	9					
3		Dense grey sandy silt, some gravel, shale fragments (GLACIAL TILL)		96.00 3.26	4	50 DO	43					
4					5	50 DO	27					
		End of Hole		94.96 4.30								
5												
6												
7												
8												
9												
10												

DATA INPUT: O:97-2-863.drl/B.L

DEPTH SCALE (ALONG HOLE)

1 to 50

Golder Associates

LOGGED: H.E

CHECKED: B

PROJECT: 971-2863

LOCATION: See Plan

DIP:

RECORD OF BOREHOLE 97-3

BORING DATE: May 20, 1997

SHEET 1 OF 1

DATUM: Local

SAMPLER HAMMER, 63.5kg; DROP, 760 mm

BORING METHOD: Power Auger 200mm Diam (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	RECOVERY %	LAB. TESTING
Asphalt Surface	99.37 0.00					
ASPHALTIC CONCRETE	0.08					
Brown medium to coarse sand and gravel (FILL)	98.96 0.41					
Grey brow silty clay, some sand, gravel and wood (FILL)		1	50 DO	5		
	97.42 1.95	2	50 DO	7		
Brown PEAT	96.78 2.59	3	50 DO	14		
Brown ALLUVIUM with shells	96.38 2.99					
Dense to very dense grey sandy silt, some gravel and shale fragments (GLACIAL TILL)		4	50 DO	56		
	95.22 4.15	5	50 DO	>100		
Black weathered SHALE	93.58 5.79					
End of Hole						

COMBUSTIBLE VAPOUR ( ) ●

LEL % ☐

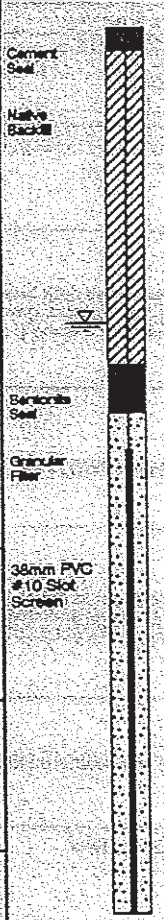
HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

INSTALLATIONS

A B C



W.L. in Screen at Elev.97.46m May 28, 1997

DATA INPUT: O:97-3-863.drf/3.L

DEPTH SCALE (ALONG HOLE)

1 to 50

Golder Associates

LOGGED: H.E.C

CHECKED: BS

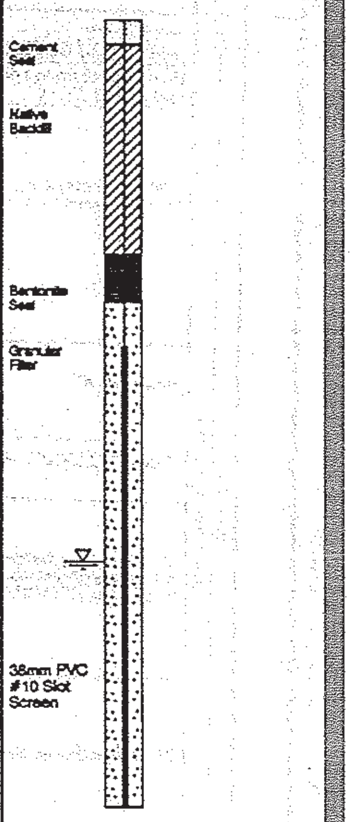
PROJECT: 971-2853
LOCATION: See Plan
DIP:

RECORD OF BOREHOLE 97-4

BORING DATE: May 20, 1997
SHEET 1 OF 1
DATUM: Local
SAMPLER HAMMER, 63.5kg; DROP, 760 mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	RECOVERY %	LAB. TESTING	COMBUSTIBLE VAPOUR ( ) ● LEL % □	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ─○W─ Wl
		Asphalt Surface		98.80							
0	Power Auger 200mm Diam (Hollow Stem)	CONCRETE		0.00							
		VOID		0.12							
1		Very loose medium brown sand (FILL)		98.22 0.58	1	50 DO	4				
2		Very loose dark brown fine to coarse sand, gravel, peat, marl, trace wood (FILL)		98.55 1.25	2	50 DO	2				
3		Brown PEAT, trace sand		97.51 2.29	3	50 DO	5				
4		Compact to dense grey sandy silt, gravel and shale fragments (GLACIAL TILL)		96.60 3.20	4	50 DO	12				
					5	50 DO	26				
5		Black weathered SHALE		95.47 4.33							
		End of Hole		94.62 5.18							
6											
7											
8											
9											
10											

INSTALLATIONS



W.L in Screen at Elev.96.27m May 28, 1997

DATA INPUT: O:97-4-863.drf/8.L

DEPTH SCALE (ALONG HOLE)
1 to 50

Golder Associates

LOGGED: H.E
CHECKED: /5

PROJECT: 971-2653

RECORD OF BOREHOLE 97-5

SHEET 1 OF 1

LOCATION: See Plan

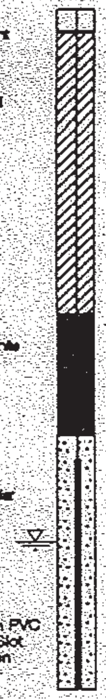
BORING DATE: May 17, 1997

DATUM: Local

DIP:

SAMPLER HAMMER, 63.5kg; DROP, 760 mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	RECOVERY %	LAB. TESTING	COMBUSTIBLE VAPOUR ● ( ) LEL % ☐	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl	INSTALLATIONS A B C
		Asphalt Surface		98.64								
0	Power Auger 200mm Diam (Hollow Stem)			0.05						0.00-0.05 - ASPHALTIC CONCRETE		
		CONCRETE		0.15								Cement Seal
1 2		Brown medium to coarse sand, trace to some gravel (FILL)		97.36 2.28	1 2	50 DO 50 DO	5 2					Native Backfill Bentonite Seal
		Brown PEAT		96.90 2.74	3	50 DO	2					
3		Dense dark grey sandy silt with shale fragments, occasional gravel (GLACIAL TILL)		95.74 3.90	4 5	50 DO 50 DO	34 >100					Granular Filter
4	Rotary Drill HQ Core	Black weathered SHALE (Hydrocarbon Odour)		95.19 4.45	6 7	HQ RC HQ RC	— —			T.C.R. 93.8% S.C.R. 80.4% R.Q.D. 74.1% T.C.R. 92.9% S.C.R. 92.9% R.Q.D. 0%		38mm PVC #10 Slot Screen
5 6 7 8 9 10		End of Hole										W.L. in Screen at Elev.96.19m May 28, 1997



DATA INPUT: O:97-8-463.drt/B.L

DEPTH SCALE (ALONG HOLE)
1 to 50

Golder Associates

LOGGED: H.E.C

CHECKED: BS

Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 20895

Report Date: 13-Dec-2016
Order Date: 7-Dec-2016
Project Description: PG3970

	Client ID:	BH1-SS5	-	-	-
	Sample Date:	06-Dec-16	-	-	-
	Sample ID:	1650245-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.72	-	-	-
Resistivity	0.10 Ohm.m	36.7	-	-	-
Anions					
Chloride	5 ug/g dry	22	-	-	-
Sulphate	5 ug/g dry	96	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SHEAR WAVE VELOCITY PROFILES

DRAWING PG3970-1 - TEST HOLE LOCATION PLAN

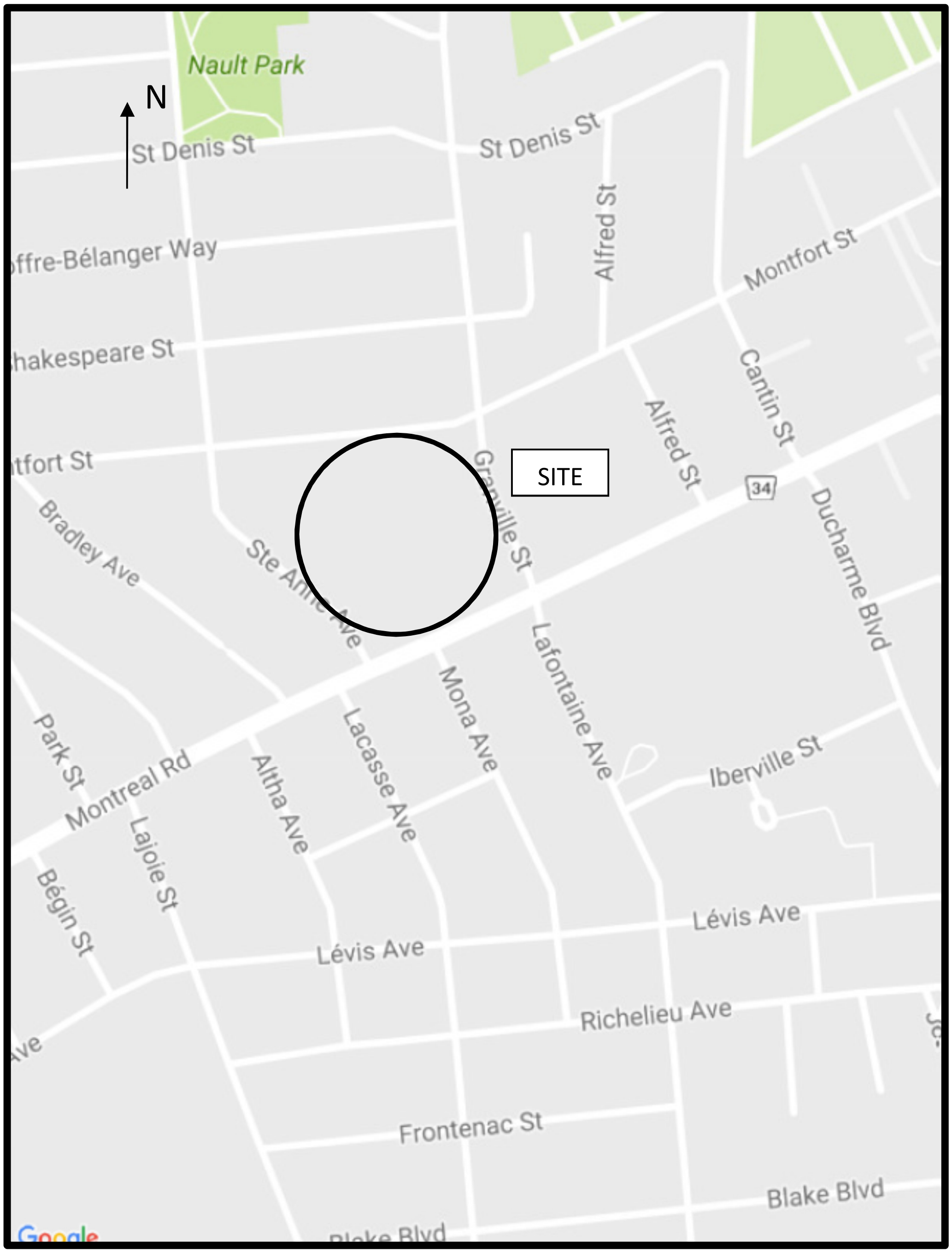


FIGURE 1

KEY PLAN

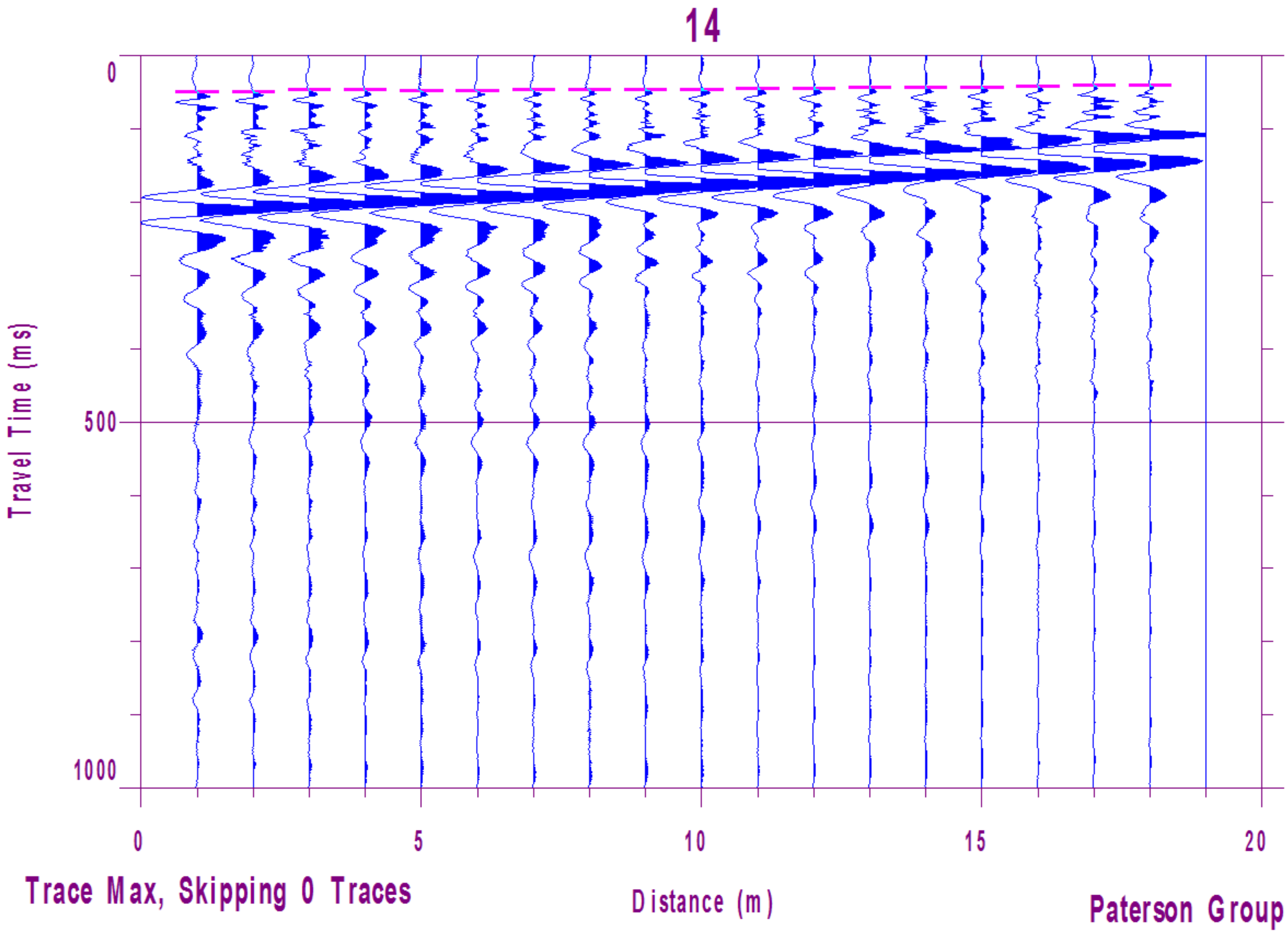


Figure 2 – Shear Wave Velocity Profile at Shot Location +16.5 m

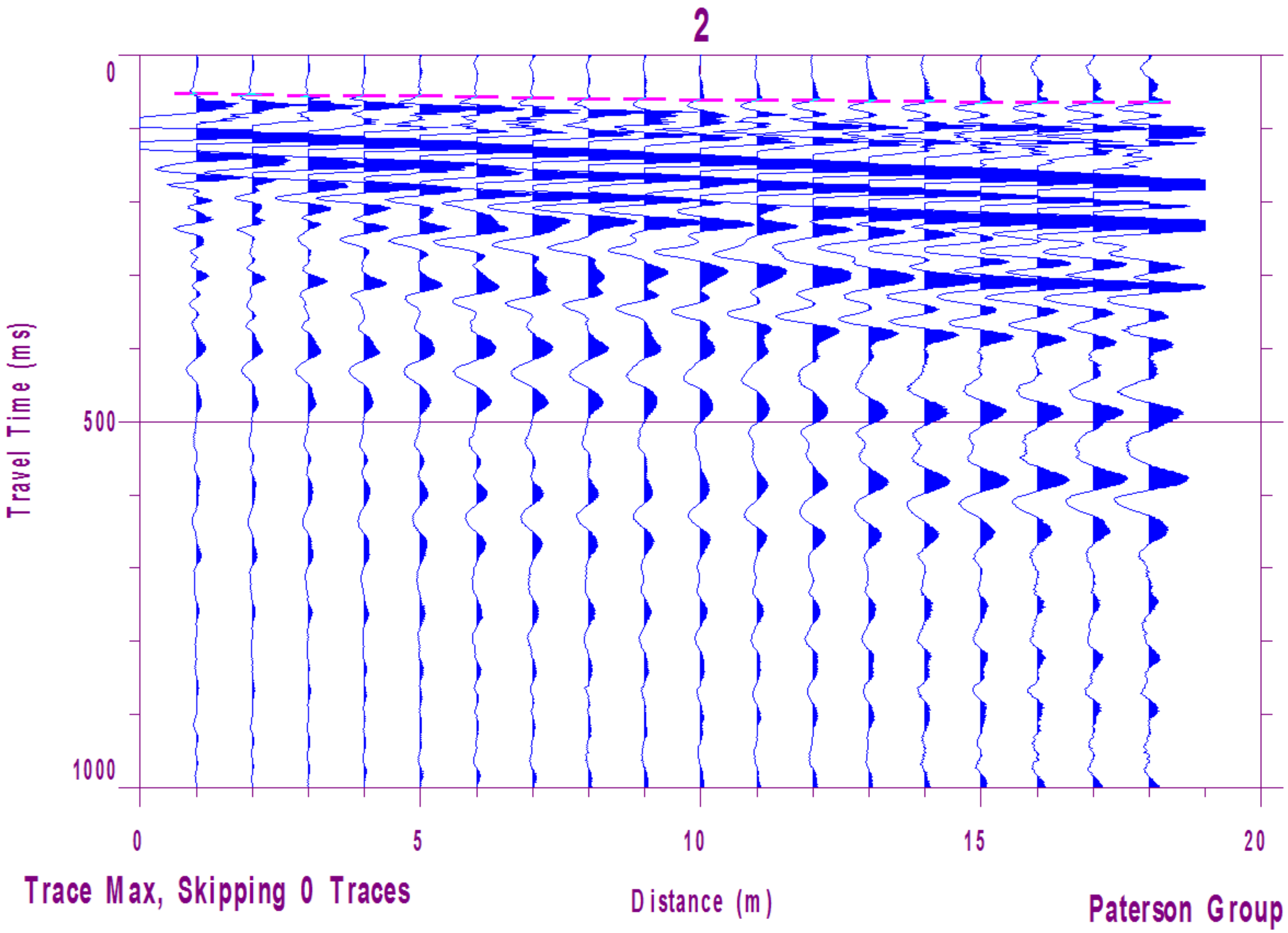
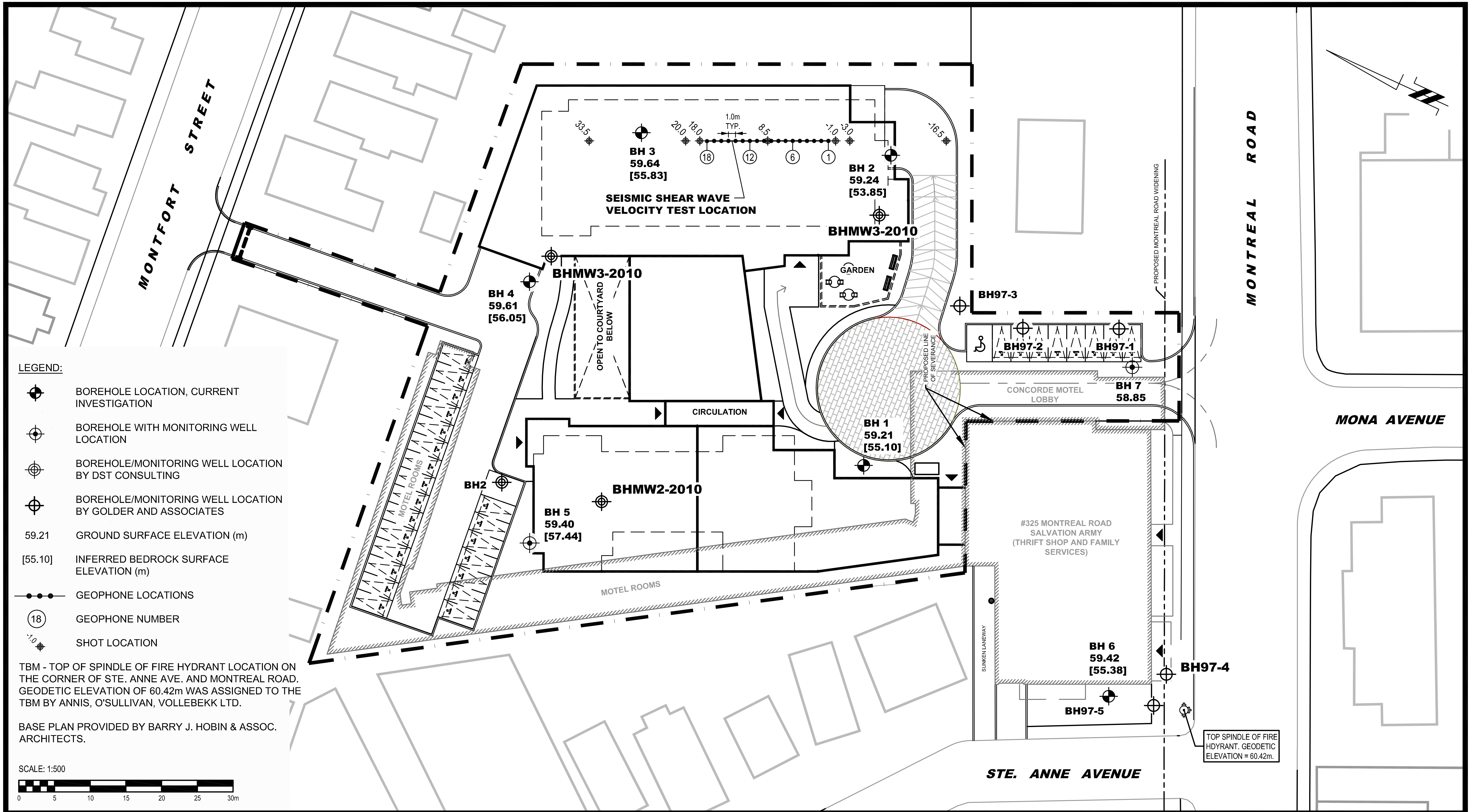


Figure 3 – Shear Wave Velocity Profile at Shot Location -16.5 m



LEGEND:

⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE/MONITORING WELL LOCATION BY DST CONSULTING
⊕	BOREHOLE/MONITORING WELL LOCATION BY GOLDER AND ASSOCIATES
59.21	GROUND SURFACE ELEVATION (m)
[55.10]	INFERRED BEDROCK SURFACE ELEVATION (m)
—●—●—●—	GEOPHONE LOCATIONS
(18)	GEOPHONE NUMBER
-1.0 ⊕	SHOT LOCATION

TBM - TOP OF SPINDLE OF FIRE HYDRANT LOCATION ON THE CORNER OF STE. ANNE AVE. AND MONTREAL ROAD. GEODETIC ELEVATION OF 60.42m WAS ASSIGNED TO THE TBM BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

BASE PLAN PROVIDED BY BARRY J. HOBIN & ASSOC. ARCHITECTS.

SCALE: 1:500

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

THE SALVATION ARMY
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 333 MONTREAL ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale:	1:500
Drawn by:	RCG
Checked by:	SB
Approved by:	DJG
Date:	12/2016
Report No.:	PG3970-1
Dwg. No.:	PG3970-1
Revision No.:	0

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