PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

1649 Montreal Road and 741 Blair Road
Ottawa, Ontario

Prepared for 10869279 Canada Inc.

Report PG5663-1 Revision 1 dated August 29, 2022

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 10869279 Canada Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1649 Montreal Road and 741 Blair Road, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

☐ determine the subsoil and groundwater conditions at this site by means of test holes.

☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the proposed development was not part of the scope of work.

2.0 Proposed Development

It is understood that the proposed development will consist of a multi-storey building over an underground parking structure, which will extend 2 or more levels and cover the majority of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the investigation was carried out on October 15 and 16, 2020. As part of the investigation, seven (7) boreholes were completed across the subject site extending to a maximum 6.5 m depth. The test hole locations were placed in a manner to provide general coverage of the subject site. The test hole locations are illustrated on Drawing PG5663-1 - Test Hole Location Plan presented in Appendix 2.

The boreholes were completed using a low-clearance auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. The bedrock was cored to assess the bedrock quality. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores (RC) were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1

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

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

Groundwater

Monitoring wells were installed in the boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test holes were located in the field by Paterson personnel using a GPS unit, and the ground surface elevation at the test hole locations from the current investigation were referenced to a geodetic datum. The ground surface elevation and location of the test holes are presented on Drawing PG5663-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the field investigation were examined in our laboratory to review field notes and soil samples. All samples will be stored in the laboratory for a period of one month after issuance of the report. They will then be discarded unless we are otherwise directed.

4.0 Observations

4.1 Surface Conditions

Currently, a car repair shop and its associated parking lot occupy the south part of the site, while a small house along with its associated landscaped area occupy the north western corner of the site. The remaining part of the site is heavily treed. The majority of the ground surface across the subject site is relatively flat and is gradually sloping downwards towards the south.

The site is bound by single houses and landscaped areas to the north, a one floor church to the east, Montreal Road to the south, and Blair Road to the west.

4.2 Subsurface Profile

Overburden

Generally, the soil conditions encountered at the test hole locations consist of a concrete slab/asphaltic concrete layer followed by fill overlying a compact glacial till layer. The fill material consisted of brown silty sand with crushed stone. Compact brown silt and silty sand were occasionally encountered below the fill. Practical refusal to augering was generally encountered at depths ranging between 2 to 6 m below existing ground surface. Bedrock consisting of poor to fair quality grey limestone interbedded with shale was cored at BH2-20, BH3-20, BH6-20, and BH7-20 beginning at depths of 1.8 to 3.8m below existing ground surface.

Bedrock

Based on available geological mapping, the bedrock in this area mostly consists of interbedded limestone and dolomite of the Gull River formation with an overburden drift thickness of 1 to 5 m depth.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

4.3 Groundwater

Groundwater levels (GWL) were measured in the monitoring wells installed in five boreholes and results are summarized in Table 1. It should be noted that surface water can become perched within a backfilled borehole, which can lead to higher than normal groundwater level readings. The long-term groundwater level can also be estimated based on moisture levels and colour of the recovered soil samples. Based on these observations, the long-term groundwater table is expected to be located below the bedrock surface. 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 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1	97.76	2.42	95.34	October 19, 2020
BH 2	97.48	2.59	94.89	October 19, 2020
BH 3	97.66	2.31	95.35	October 19, 2020
BH 6	97.92	1.35	96.57	October 19, 2020
BH 7	97.98	1.81	96.17	October 21, 2020

5.0 Discussion

5.1 Geotechnical Assessment

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

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming, rock grinding and controlled blasting will be required to remove the bedrock for the underground parking levels. 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 the 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 or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding 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 near vertical sidewalls. 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. The 1 m horizontal ledge set back can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Fill Placement

Excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. Alternatively, an engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings.

Bedrock Excavation Face Reinforcement

Horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. 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.

5.2.1 Vibration Monitoring and Control Plan

An automatic emailer, set up by Paterson, will provide real-time results to the blasting consultant, construction team and Paterson for immediate review. Following each recorded event, an email will be sent out containing the following:

☐ A breakdown of the vibration event including the PPV, dominant frequency, and zero cross frequency in each direction (transverse, vertical, longitudinal).

☐ The monitor serial, calibration date, and the location of the monitor recording the event.

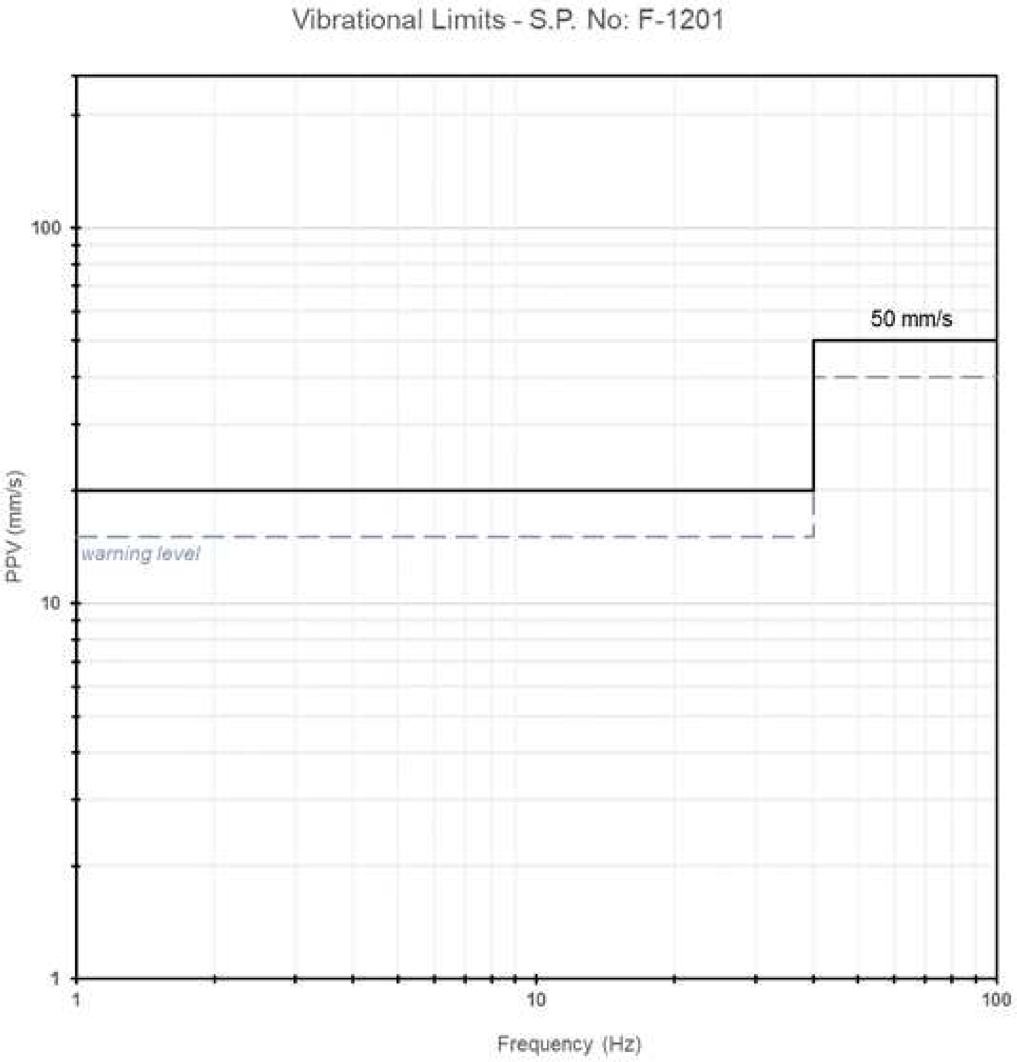
☐ A statement indicating if the vibration is either within the agreed upon limits or in exceedance.

☐ A PDF attachment containing the full waveforms and FFT report.

For warning or exceedance level events, the procedures described below will be followed.

Proposed Vibration Limits

The excavation operations should be planned and conducted under the supervision of a licensed professional engineer. The vibration limits for the associated infrastructure, outlined in the figure below, are 20 mm/s for frequencies below 40 Hz, and 50 mm/s for frequencies 40 Hz and higher. The warning level limits are 10 mm/s for frequencies below 40 Hz, and 40 mm/s for frequencies 40 Hz and higher.



Monitoring Data

The monitoring protocol should include the following information:

Warning Level Event

☐ Review the waveforms of the event to determine the cause of the event and confirm monitor function.
☐ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.
☐ A site visit may be required to confirm the monitor placement, source of exceedance, provide mitigation recommendations and/or to review the field conditions.

Exceedance Level Event

☐ Review the waveforms of the event to determine the cause of the event and confirm monitor function.
☐ Paterson will notify the blasting contractor of the exceedance.
☐ A site visit may be required to confirm the monitor placement, source of exceedance, provide mitigation recommendations and/or to review the field conditions.
☐ An exceedance report will be created and issued.

Incident/Exceedance Reporting

In case an incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

☐ Identify the location of the vibration exceedance,
☐ The date, time and nature of the exceedance/incident,
☐ Purpose of the exceeded monitor and current vibration criteria,
☐ Identify the likely cause of the exceedance/incident,
☐ Describe the initial response action that has been completed to date,
☐ Describe the proposed measures to address the exceedance/incident and provide an immediate action plan and prevention measures to eliminate the cause of the exceeded vibrations(s) during future work.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean surface sounded bedrock surface at the proposed founding elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **5,000 kPa** incorporating a geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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.

For a foundation placed on bedrock, the total and differential settlement are expected to be negligible.

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).

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 2 and 3, which are presented in Appendix 2 of this report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5663-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones 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 were 15, 1.5 and 1.0 m away from the first and last geophone and at the centre of the seismic array.

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, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building is **2,033 m/s** provided the footings are placed directly on bedrock. The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30}= \frac{30\,m}{\left(\frac{30\,m}{2{,}033\,m/s}\right)}$$

$$V_{s30}= 2{,}033\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30}, is 2,033 m/s. Therefore, a **Site Class A** is applicable for the design of proposed building bearing on bedrock, 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

The basement floor slab at the final underground level will be placed over a minimum 200 mm thick layer OPSS Granular A crushed stone followed by Granular B Type II to the top of the bedrock subgrade. The sub-floor granular material within the footprint of the building will be placed in maximum 300 mm thick lifts and compacted to at least 98% of the material's SPMDD.

An underfloor drainage system is required between the finished floor and the underlying bedrock subgrade to direct water infiltration to the building sump pit.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a composite drainage system, which will be placed against the exposed bedrock face. A nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

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.

Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Static Conditions

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 Conditions

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 (City Hall) area is 0.281 g according to OBC 2012 (R2019). 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 of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

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

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

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

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

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

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 60 and 120 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

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 in Table 3. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
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	1.2	0.6	1.8	250
	1.9	1.0	2.9	500
	3	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.9	2.4	500
	2.6	1.0	3.6	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

It is anticipated that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that the foundation wall will be blind poured against a drainage system and waterproofing system fastened against the shoring system and bedrock face.

A waterproofing membrane, such as a granular bentonite sheeting, will be required to lessen the effect of water infiltration. The waterproofing membrane can be placed and fastened to the shoring system and should extend between 3 m below finished grade and to the bottom of the excavation at the founding level of the perimeter footings. The waterproofing should be extended a minimum of 600 mm below the underside of the perimeter footings.

It is recommended that the composite drainage system, such as Delta Drain 6000 or equivalent, extend from the exterior finished grade to the founding elevation. The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footings interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, we recommend that 150 mm diameter perforated, corrugated pipes be placed along the interior perimeter of the foundation wall and one drainage line within each bay. The spacing of the underfloor drainage system should be confirmed at the time of backfilling the floor completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose.

Adverse Effects from Dewatering on Adjacent Structures

Based on the native subsoil conditions encountered at the subject site (glacial till deposit overlying shallow bedrock), no overburden soil was encountered that is considered sensitive to dewatering. Surrounding structures are currently founded on bedrock or the dense glacial till deposit. Therefore, in our opinion, no adverse effects from short term and long term dewatering are expected for surrounding structures.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the 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

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for the majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

A trench box is recommended 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.

Temporary Shoring

It is anticipated that temporary shoring is required to complete the required excavation where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the 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 to the approved shoring design system should be reported immediately to the owner’s structural designer 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. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The 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. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 4 - Soil Parameters for Shoring System Design

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

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

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

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

6.4 Pipe Bedding and Backfill

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 99% 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 crushed stone, 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 99% of the SPMDD.

Generally, it should be possible to re-use the site material above the cover material if the excavation and backfilling operations are completed in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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

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

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low to negligible (less than 20,000 L/day).

Impacts on Neighboring Structures

It is understood that at least two levels of underground parking are planned for the proposed building. Based on the existing groundwater level and considering the proposed building will be surrounded by a waterproofing membrane, long-term groundwater lowering will be minimal and take place within a limited range of the proposed building. Based on the proximity of neighboring buildings and minimal zone impacted by the groundwater lowering, 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 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. Additional information could be provided, if required.

7.0 Recommendations

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

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Inspection of the foundation waterproofing and all foundation drainage systems.

☐ Sampling and testing of the concrete and fill materials placed.

☐ 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.

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

8.0 Statement of Limitations

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

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, we request notification 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 10869279 Canada Inc or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Patey

Nicole Patey, B.Eng.

LICENSED PROFESSIONAL ENGINEER
August 29, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

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
1649 Montreal Road and 741 Blair Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 15, 2020

FILE NO. PG5663

HOLE NO. BH 1-20

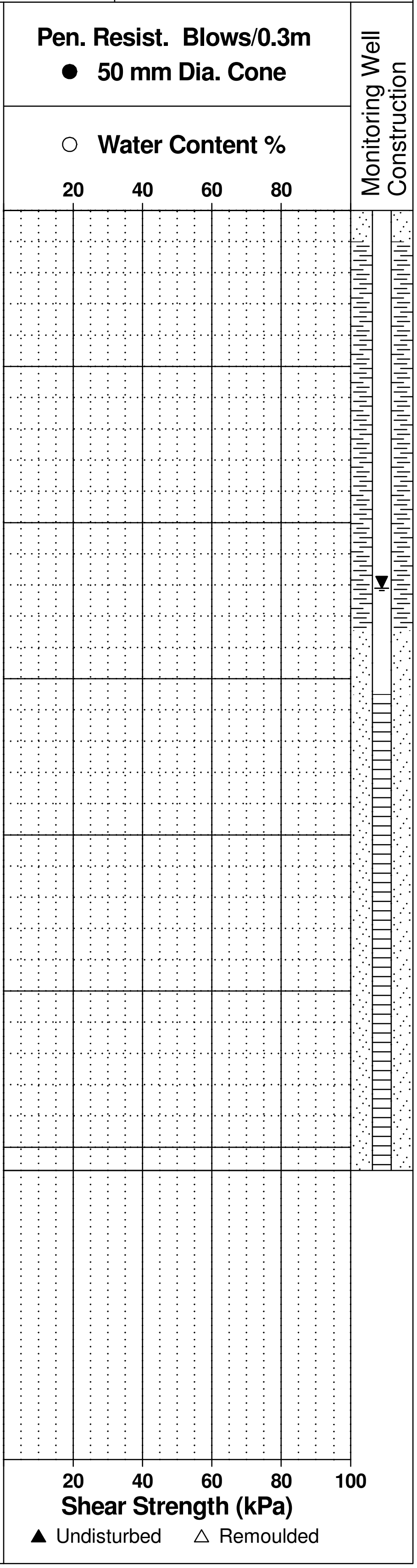
SOIL DESCRIPTION	Bottom depth (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.66
FILL: Brown silty sand with clay	1.37
FILL: Brown silty sand	3.81
GLACIAL TILL: Loose to very dense, grey silty sand with gravel	5.60
Weathered **BEDROCK**	6.15
End of Borehole	

Practical refusal to augering at 6.15m depth

(GWL @ 2.42m - Oct. 19, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	29	7
SS	3	33	7
SS	4	58	2
SS	5	17	W
SS	6	75	5
SS	7	25	14
SS	8	91	50+
SS	9	50	50+

DEPTH (m)	ELEV. (m)
0	97.76
1	96.76
2	95.76
3	94.76
4	93.76
5	92.76
6	91.76



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 15, 2020

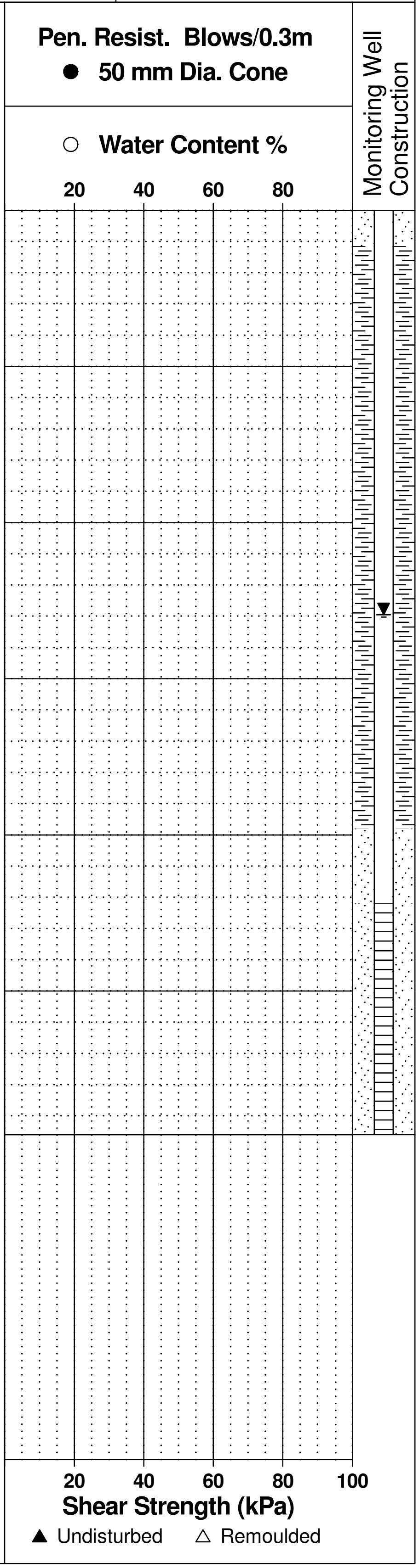
FILE NO. PG5663

HOLE NO. BH 2-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand, trace crushed stone	0.56	AU	1		
FILL: Grey/black silty sand with gravel and crushed stone	1.83	SS	2	54	10
Grey **SILTY CLAY,** some sand	2.13	SS	3	33	7
GLACIAL TILL: Compact to dense, brown silty sand with clay and gravel	3.61	SS SS	4 5	50 25	14 50+
BEDROCK: Poor to fair quality, grey limestone	5.92	RC RC	1 2	100 100	45 58
End of Borehole					

(GWL @ 2.59m - Oct. 19, 2020)

DEPTH (m)	ELEV. (m)
0	97.48
1	96.48
2	95.48
3	94.48
4	93.48
5	92.48



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Ottawa, Ontario

DATUM Geodetic

REMARKS

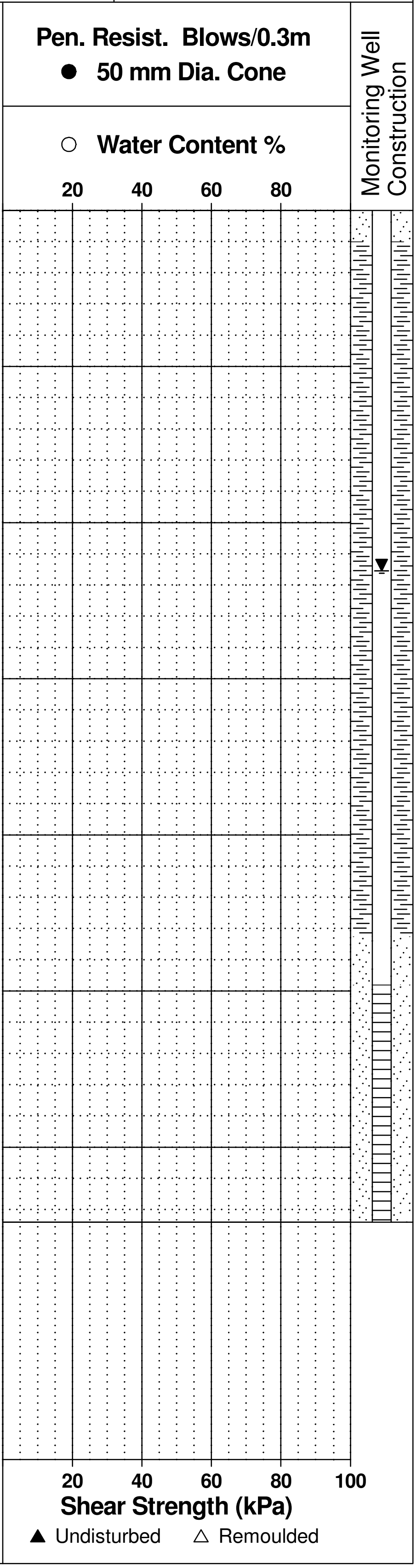
BORINGS BY CME-55 Low Clearance Drill

DATE October 15, 2020

FILE NO. PG5663

HOLE NO. BH 3-20

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	97.66
Asphaltic concrete 0.08						
FILL: Brown silty sand with crushed stone	AU	1				
	SS	2	62	27	1	96.66
	SS	3	54	32	2	95.66
3.05	SS	4	46	18	3	94.66
GLACIAL TILL: Brown silty sand with gravel	SS	5	17	3		
3.83	SS	6	0	50+		
BEDROCK: Poor quality, grey limestone	RC	1	57	45	4	93.66
	RC	2	100	48	5	92.66
6.48					6	91.66
End of Borehole						
(GWL @ 2.31m - Oct. 19, 2020)						



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

REMARKS

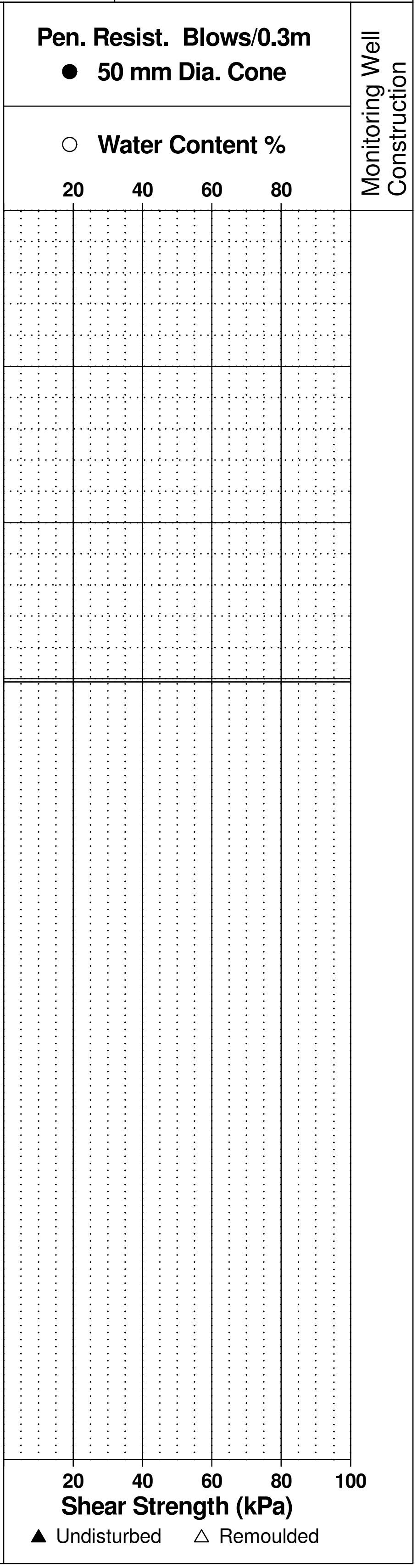
BORINGS BY CME-55 Low Clearance Drill

DATE October 15, 2020

FILE NO. PG5663

HOLE NO. BH 4-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	97.77
Asphaltic concrete 0.08						
FILL: Brown silty sand, some crushed stone 0.51	AU	1				
Compact, brown **SILTY SAND** with gravel	SS	2	71	16	1	96.77
	SS	3	62	26		
2.29					2	95.77
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders 3.02	SS		46	15		
End of Borehole Practical refusal to augering at 3.02m depth.					3	94.77



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Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 15, 2020

FILE NO. PG5663

HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	97.93		
Asphaltic concrete 0.08									
FILL: Brown silty sand with crushed stone 0.60		AU	1						
FILL: Brown silty sand with gravel 1.52		SS	2	33	7	1	96.93		
Brown **SILT,** some sand and gravel 1.96		SS	3	59	50+				
End of Borehole Practical refusal to augering at 1.96m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1649 Montreal Road and 741 Blair Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

DATE October 16, 2020

FILE NO. PG5663

HOLE NO. BH 6-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	97.92
Concrete slab (0.16)						
FILL: Brown silty sand	SS	1	83			
	SS	2	67		1	96.92
(1.75)	SS	3	60			
BEDROCK: Poor to fair quality, grey limestone interbedded with shale	RC	1	100	42		
	RC	2	100	36	2	95.92
	RC	3	100	47		
	RC	4	100	0	3	94.92
	RC	5	100	0		
	RC	6	69	0		
(4.47)	RC	7	100	55	4	93.92
End of Borehole						

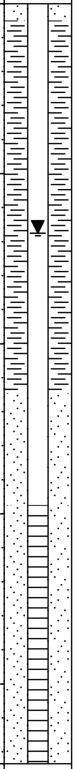
(GWL @ 1.35m - Oct. 19, 2020)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction



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 Geodetic

REMARKS

BORINGS BY Portable Drill

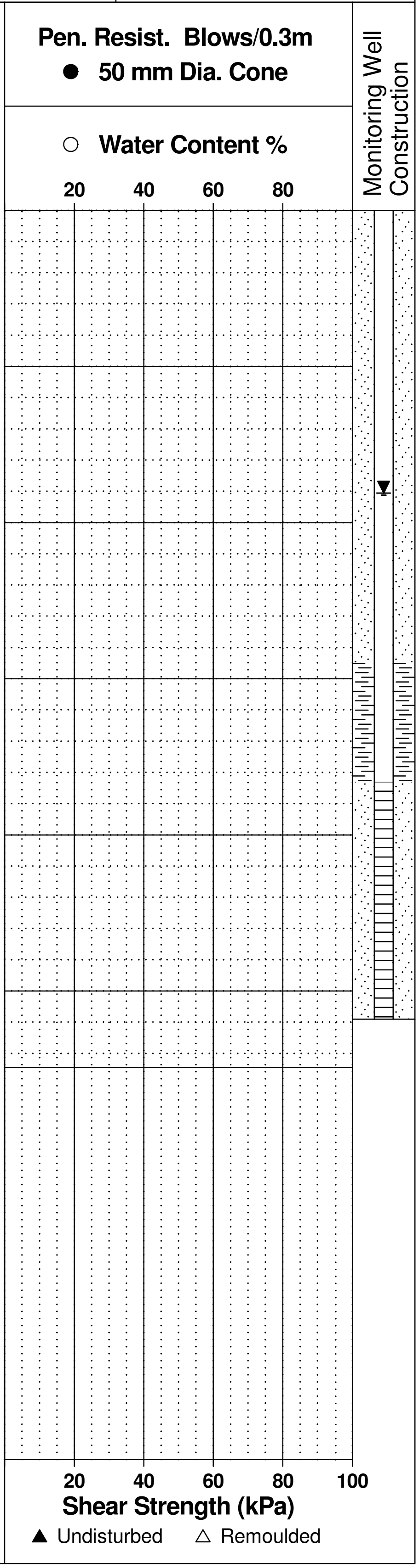
DATE October 16, 2020

FILE NO. PG5663

HOLE NO. BH 7-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
Concrete slab 0.13				
FILL: Brown silty sand with crushed stone 0.56	SS	1	83	
FILL: Brown silty sand, trace gravel, clay and crushed stone 1.80	SS	2	67	
	SS	3	42	
Compact, brown **SILT,** some sand, occasional gravel 2.74	SS	4	58	
	SS	5	58	
BEDROCK: Poor to fair quality, grey limestone interbedded with shale 5.49	RC	1	100	70
	RC	2	100	0
	RC	3	100	50
	RC	4	100	42
	RC	5	100	31
	RC	6	100	30
	RC	7	100	42
	RC	8	100	33
	RC	98	100	50
End of Borehole				
(GWL @ 1.81m - Oct. 21, 2020)				

DEPTH (m)	ELEV. (m)
0	97.98
1	96.98
2	95.98
3	94.98
4	93.98
5	92.98



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SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
1649 Montreal Road
Gloucester, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 6 DEC 00

FILE NO. E2071

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Light brown to brown sand		AU	1		
		SS	2	50	2
	1.83	SS	3	75	4
Firm, grey **SILTY CLAY,** trace sand	2.13				
Compact, grey to brown **SILTY SAND,** some gravel		SS	4	67	21
	3.50	SS	5	33	8

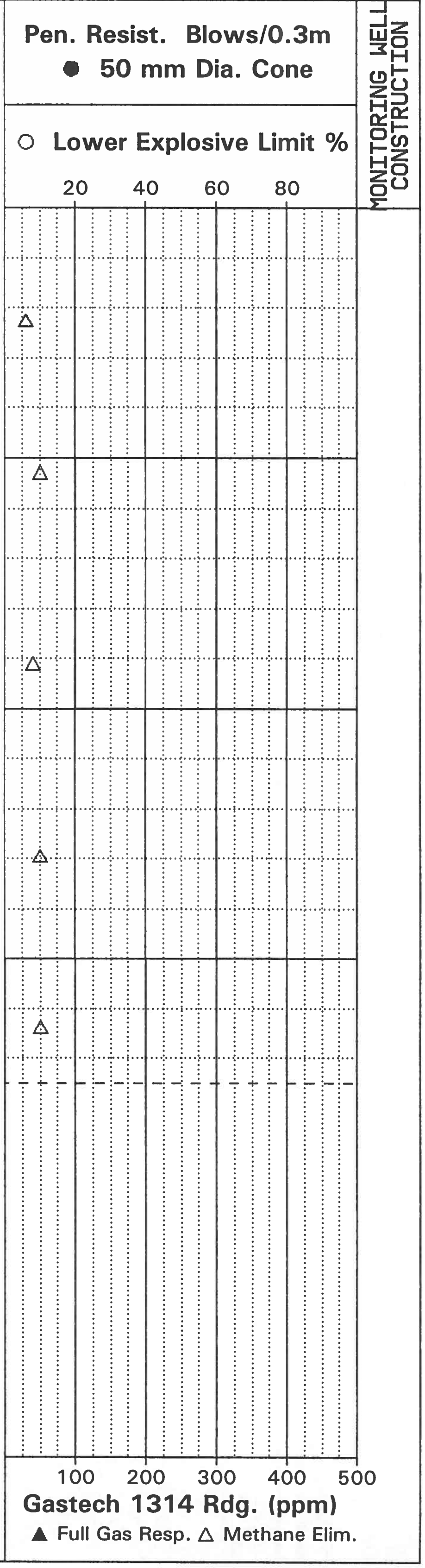
End of Borehole

Spoon refusal @ 3.50m depth

(Open hole WL @ 0.8m depth)

DEPTH (m): 0, 1, 2, 3

ELEV. (m)



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DATUM

REMARKS

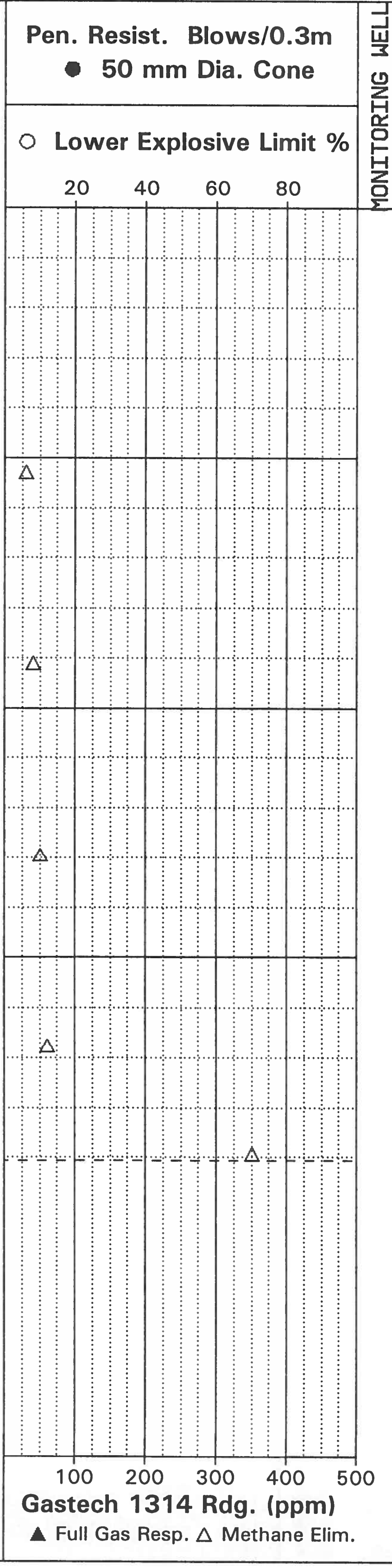
BORINGS BY CME 55 Power Auger

DATE 7 DEC 00

FILE NO. **E2071**

HOLE NO. **BH 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Dark brown sandy **TOPSOIL** 0.25							
FILL: Brown sand, some gravel, trace silt		SS	6	50	10	1	
		SS	7	50	11	2	
		SS	8	58	17	3	
		SS	9	33	2		
3.81		SS	10	100			
End of Borehole Spoon refusal @ 3.81m depth (Open hole WL @ 2.0m depth)							



MONITORING WELL CONSTRUCTION

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1649 Montreal Road
Gloucester, Ontario

DATUM

REMARKS

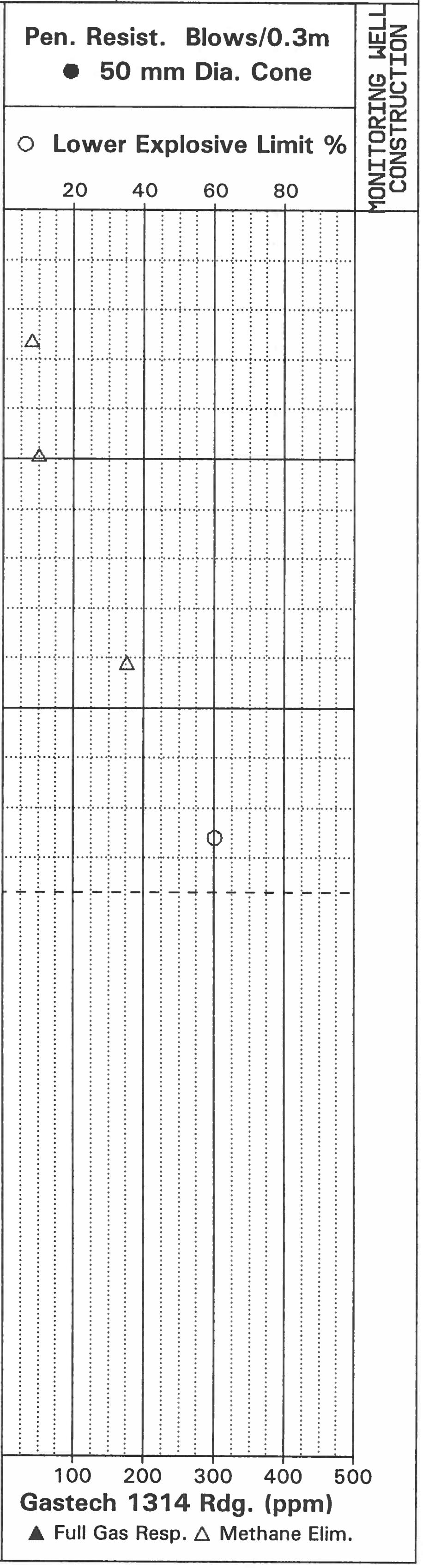
BORINGS BY CME 55 Power Auger

DATE 6 DEC 00

FILE NO. **E2071**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Asphaltic concrete 0.05							
FILL: Light brown sand to sand, clay and gravel mixture		AU	11				
		SS	12	44	12	1	
1.37							
Stiff, dark brown SILTY **CLAY** 1.68							
Compact, dark grey SILTY **SAND-GRAVEL**		SS	13	67	13	2	
		SS	14		4		
2.74							
End of Borehole Spoon refusal @ 2.74m depth (Open hole WL @ 1.8m depth)							



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Phase II Environmental Site Assessment
1649 Montreal Road
Gloucester, Ontario

DATUM

REMARKS

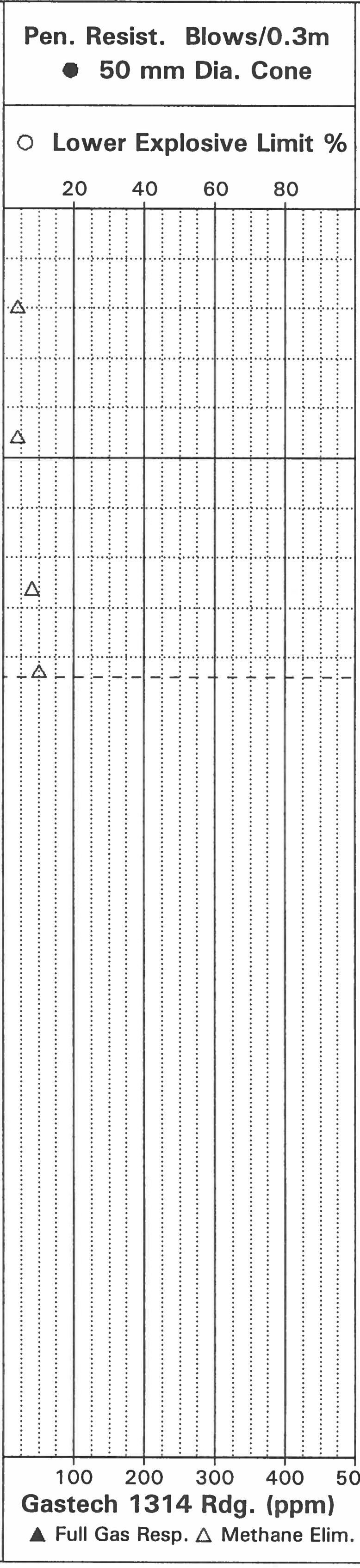
BORINGS BY Portable Drill

DATE 7 DEC 00

FILE NO. E2071

HOLE NO. CH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0		
Concrete slab 0.18								
FILL: Light brown to brown sand, trace to some gravel		SS	1		20			
		SS	2		10	1		
		SS	3		22			
1.88		SS	4					
End of Corehole Spoon refusal @ 1.88m depth								



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BORINGS BY Portable Drill

DATE 7 DEC 00

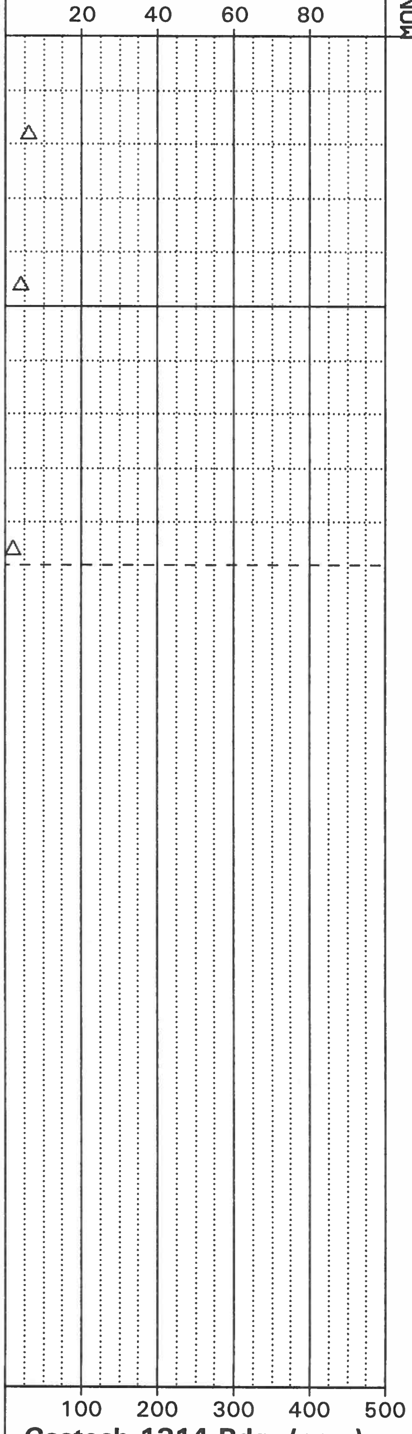
FILE NO. **E2071**

HOLE NO. **CH 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete slab 0.10							
FILL: Reddish brown to brown sand		SS	5		3		
		SS	6		2	1	
1.96		SS	7				
End of Corehole Spoon refusal @ 1.96m depth							

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

○ Lower Explosive Limit %



Gastech 1314 Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

MONITORING WELL CONSTRUCTION

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DATUM

REMARKS

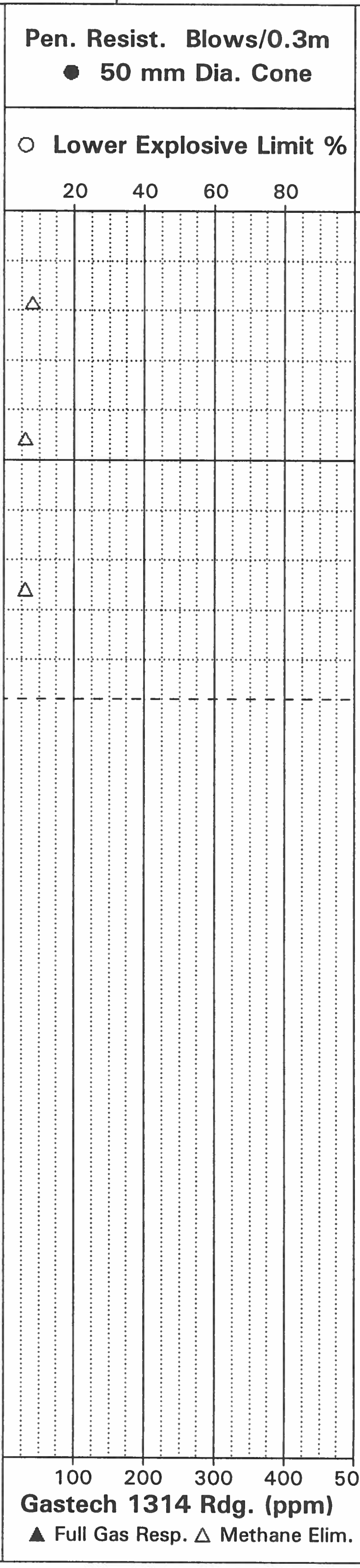
BORINGS BY Portable Drill

DATE 7 DEC 00

FILE NO. **E2071**

HOLE NO. **CH 6**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete slab 0.13							
FILL: Light brown to brown sand		SS	8		15		
		SS	9		8	1	
		SS	10				
1.96 End of Corehole Spoon refusal @ 1.96m depth							



MONITORING WELL CONSTRUCTION

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SOIL PROFILE & TEST DATA

Phase II-Environmental Site Assessment
1651 Montreal Road and 741 Blair Road
Ottawa (Gloucester), Ontario

DATUM

REMARKS

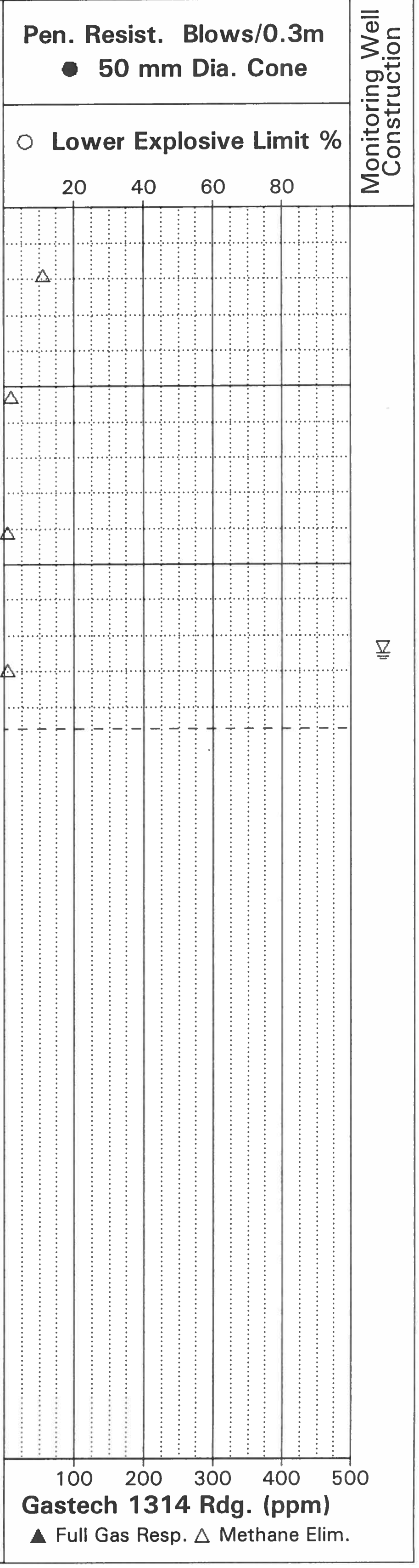
BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

FILE NO. E2662

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL 0.18							
FILL: Brown silty fine to medium sand with crushed gravel 0.70		AU	1				
Very dense, brown **SILTY SAND-GRAVEL** with cobbles		SS	2	33	72	1	
		SS	3	50	68	2	
2.92		SS	4	38	69		
End of Borehole Practical refusal to augering @ 2.92m depth (Open hole WL @ 2.5m depth)							



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SOIL PROFILE & TEST DATA

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1651 Montreal Road and 741 Blair Road
Ottawa (Gloucester), Ontario

DATUM

REMARKS

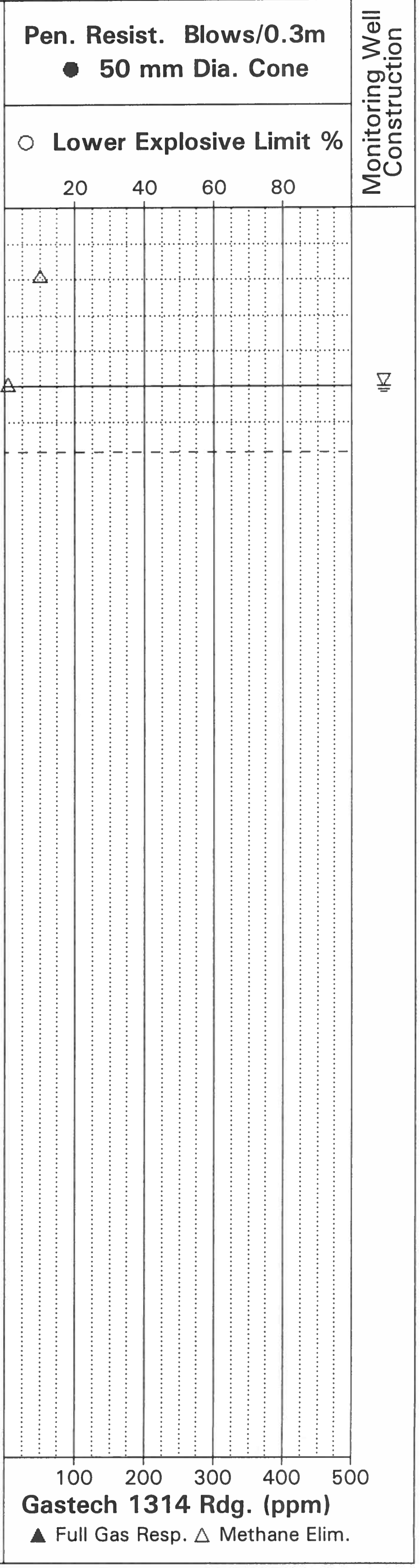
BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

FILE NO. E2662

HOLE NO. BH 2A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
TOPSOIL 0.23						0	
FILL: Dark brown silty fine to medium sand with organic matter		AU	5				
		SS	6	27	50+	1	
1.37 End of Borehole Practical refusal to augering @ 1.37m depth (Open hole WL @ 1.0m depth)							



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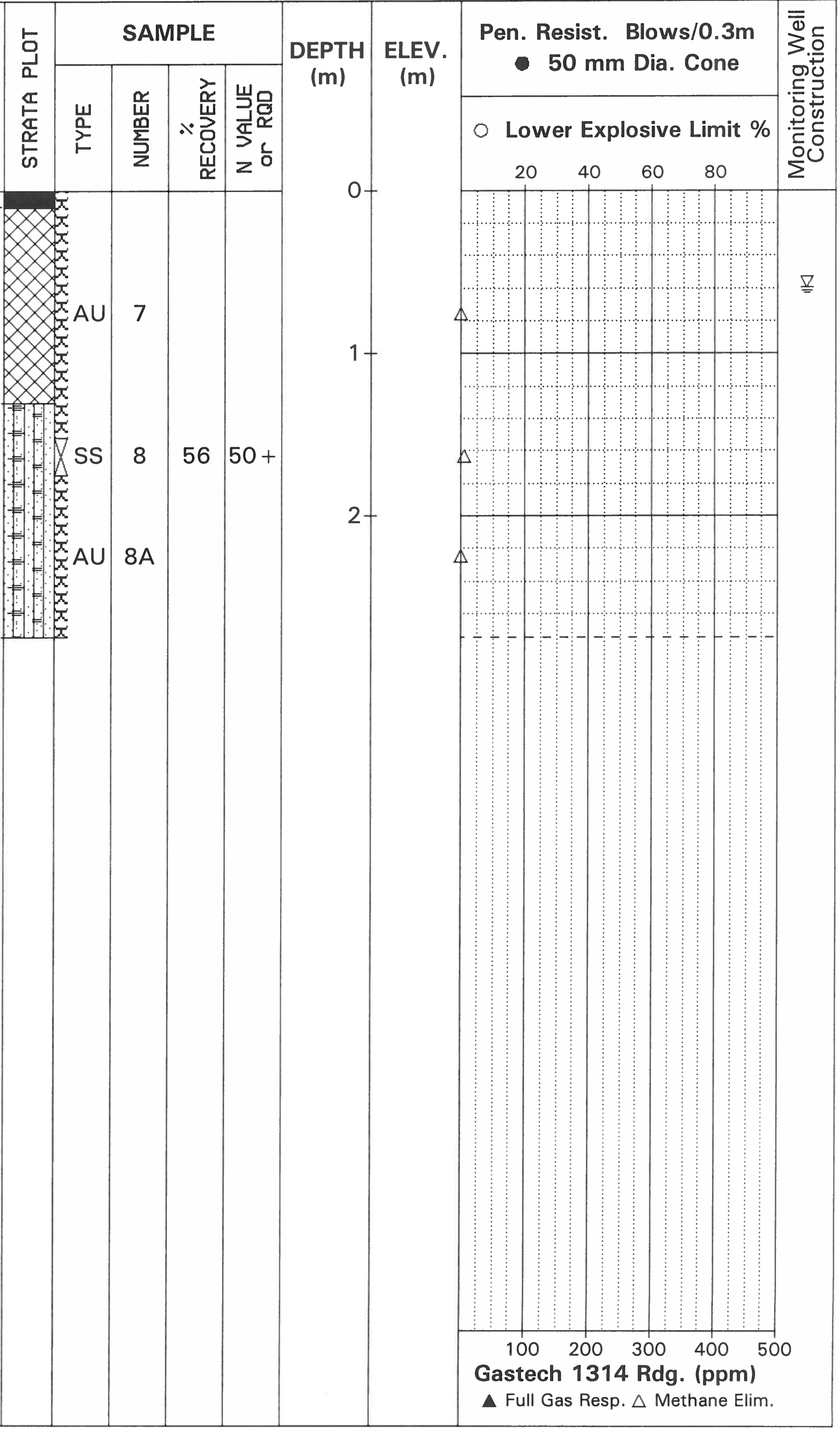
DATE 17 JUN 03

FILE NO. E2662

HOLE NO. BH 2B

SOIL DESCRIPTION	Depth of layer base	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL	0.10						
FILL: Brown silty fine to medium sand, some gravel	1.30	AU	7			1	
Very dense, brown **SILTY SAND-GRAVEL** with cobbles	2.74	SS	8	56	50 +	2	
		AU	8A				
End of Borehole Practical refusal to augering @ 2.74m depth (Open hole WL @ 0.6m depth)							



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Consulting Engineers
28 Concourse Gate, Unit 1, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Phase II-Environmental Site Assessment
1651 Montreal Road and 741 Blair Road
Ottawa (Gloucester), Ontario

DATUM

REMARKS

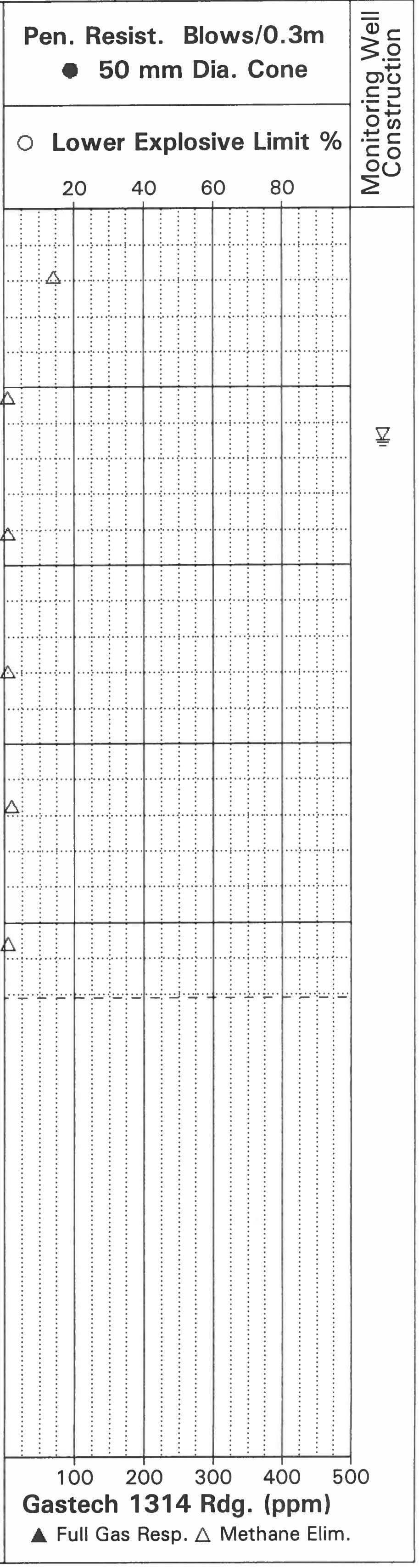
BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

FILE NO. **E2662**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Dark brown silty fine to medium sand with organic matter and rock fragments 0.66		AU	9				
FILL: Brown sandy silty clay, trace gravel 0.86							
Very stiff, brown **SILTY CLAY** 1.65		SS	10	83	11	1	
Loose to very dense, brown to greyish brown **SILTY** fine **SAND**, occasional cobbles		SS	11	79	27	2	
		SS	12	62	34		
		SS	13	58	52	3	
4.42		SS	14	29	3	4	
End of Borehole (Open hole WL @ 1.3m depth)							



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DATUM

REMARKS

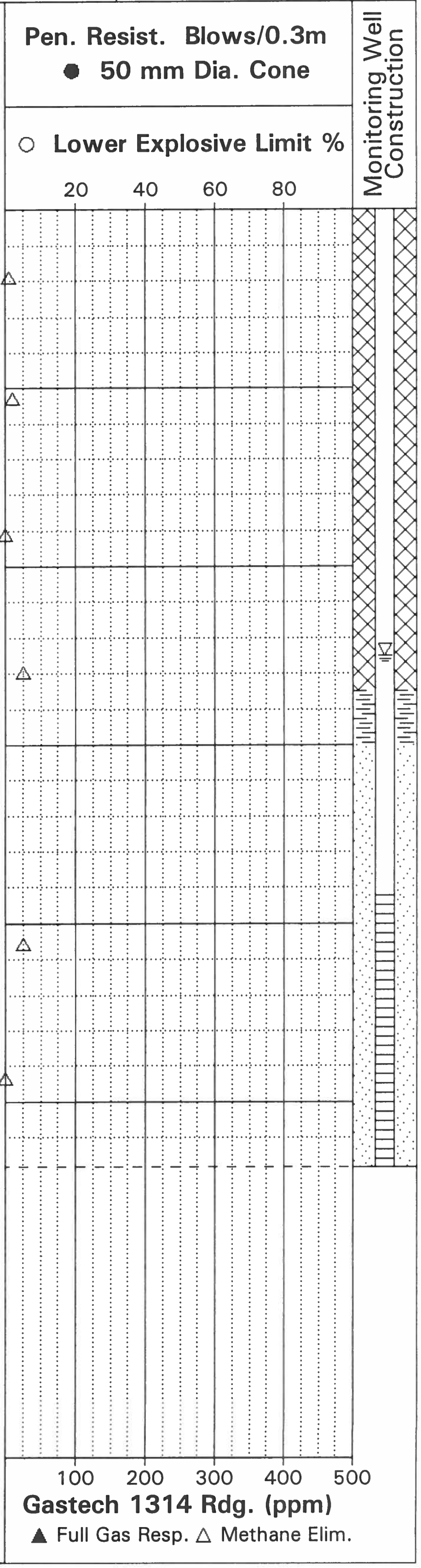
BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

FILE NO. **E2662**

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	
Asphaltic concrete 0.05							
FILL: Dark brown clayey silty sand, trace gravel		AU	15				
		SS	16	17	22	1	
		SS	17	17	18	2	
2.54							
Very stiff, brown **SILTY CLAY**, occasional sand seams		SS	18	58	16		
		SS	19	0	18	3	
		SS	20	92	14	4	
4.60							
Compact, brown SANDY **SILT**		SS	21	50	16	5	
5.36							
End of Borehole							
Practical refusal to augering @ 5.36m depth							
(GWL @ 3.50m-June 17/03)							



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DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

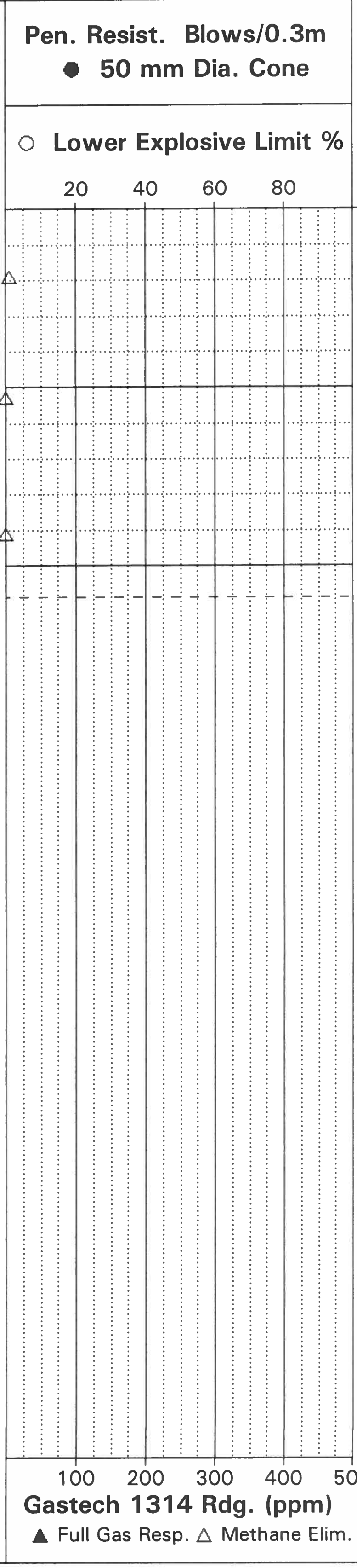
FILE NO. **E2662**

HOLE NO. **BH 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0		
FILL: Brown silty fine to medium sand with crushed gravel, cobbles and asphalt pieces	▨	AU	22					
1.17	▨	SS	23	33	29	1		
FILL: Crushed rock, some silty sand	▨	SS	24	21	65+	2		
2.18								

End of Borehole

Practical refusal to augering @ 2.18m depth.



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SOIL PROFILE & TEST DATA

Phase II-Environmental Site Assessment
1651 Montreal Road and 741 Blair Road
Ottawa (Gloucester), Ontario

DATUM

REMARKS

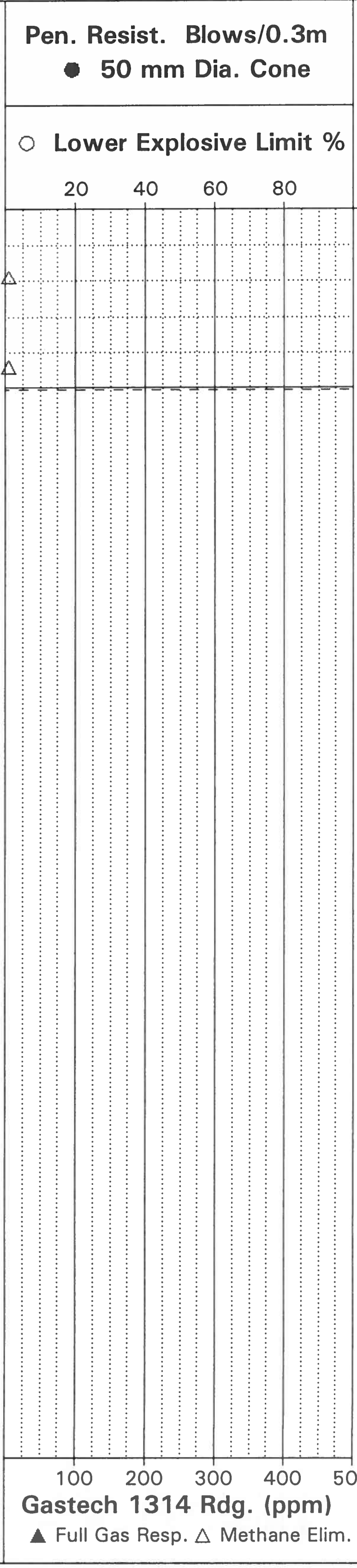
BORINGS BY CME 55 Power Auger

DATE 17 JUN 03

FILE NO. **E2662**

HOLE NO. **BH 6**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Asphaltic concrete 0.05							
FILL: Brown sand with crushed gravel	▨	AU	25				
1.01	▨	AU	26			1	
End of Borehole Practical refusal to augering @ 1.01m depth							



Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

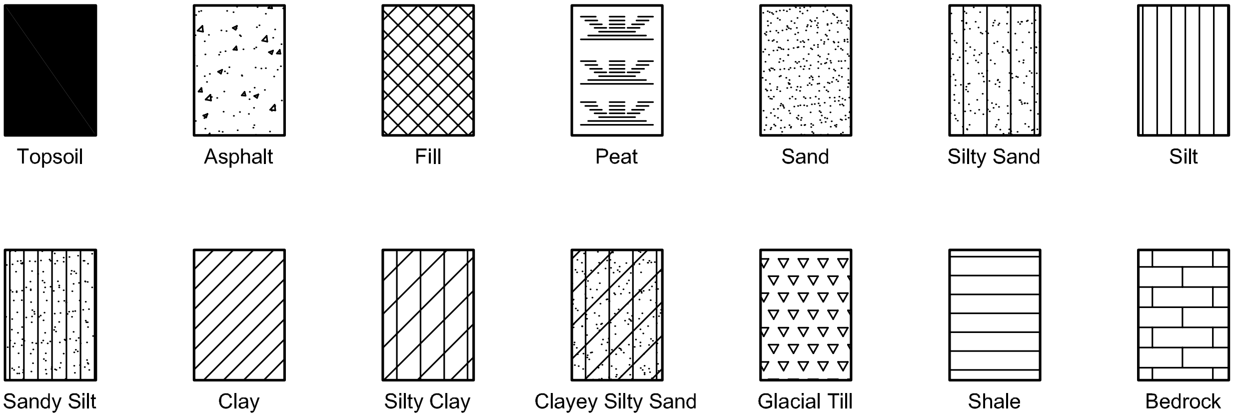
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

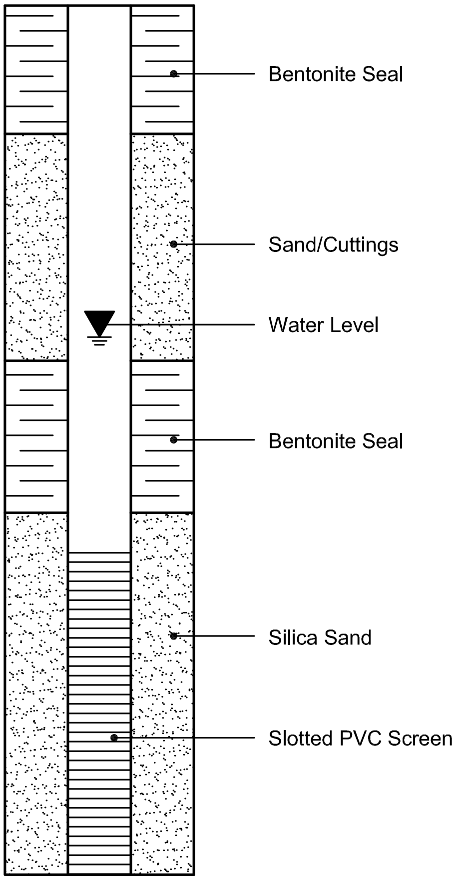
SYMBOLS AND TERMS (continued)

STRATA PLOT

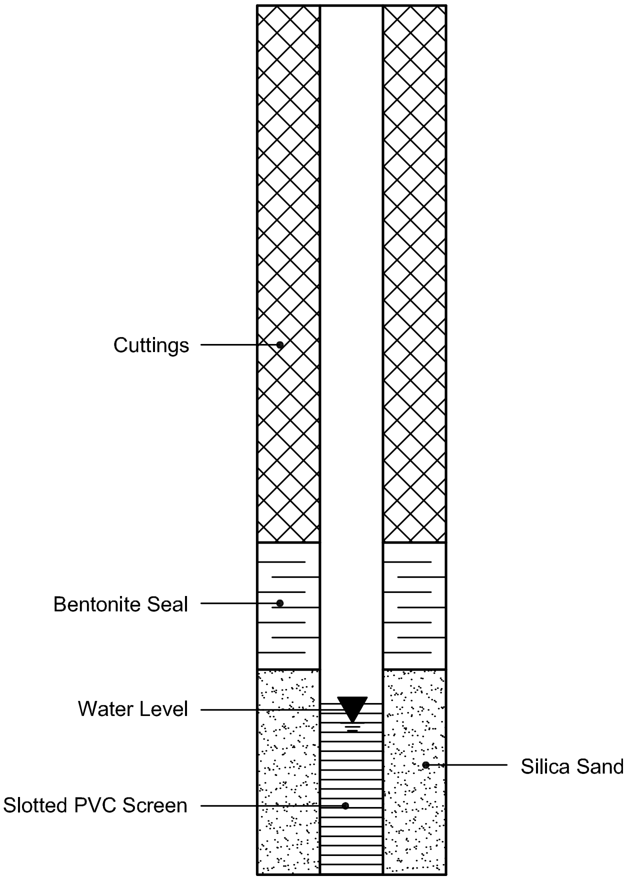


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5663-1 - TEST HOLE LOCATION PLAN

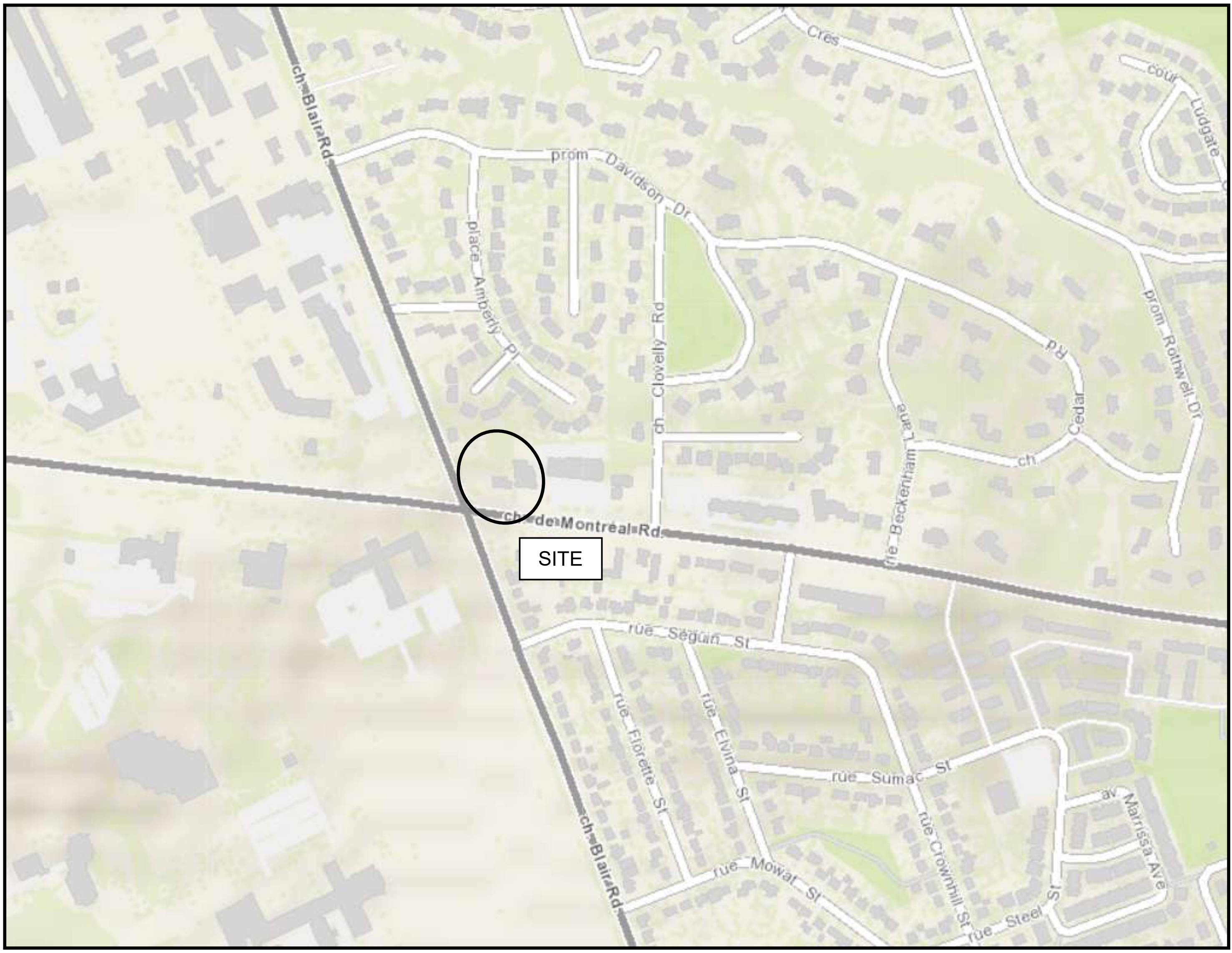


FIGURE 1

KEY PLAN

PATERSON GROUP

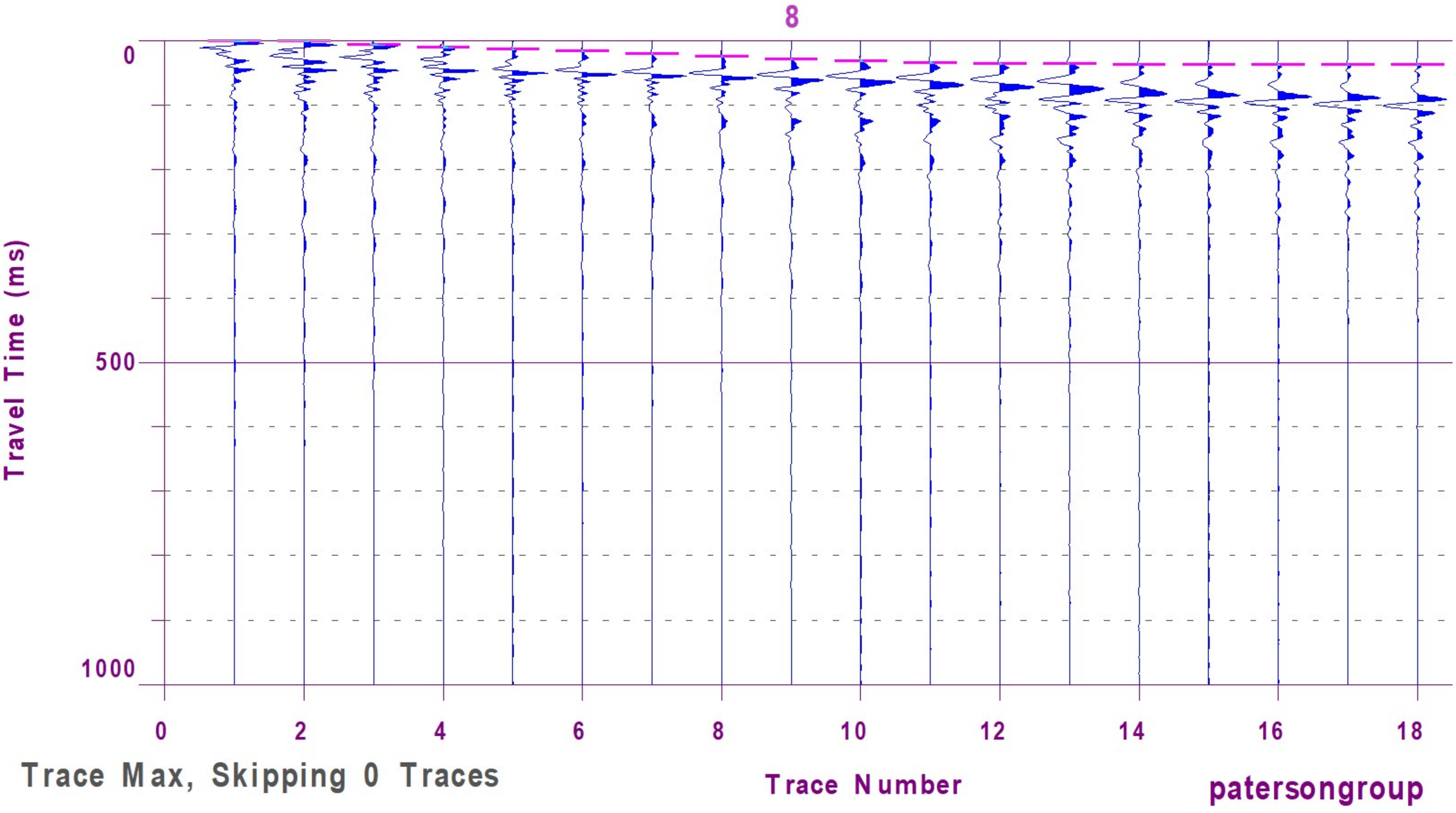


Figure 2 – Shear Wave Velocity Profile at Shot Location -1.5 m

PATERSON
GROUP

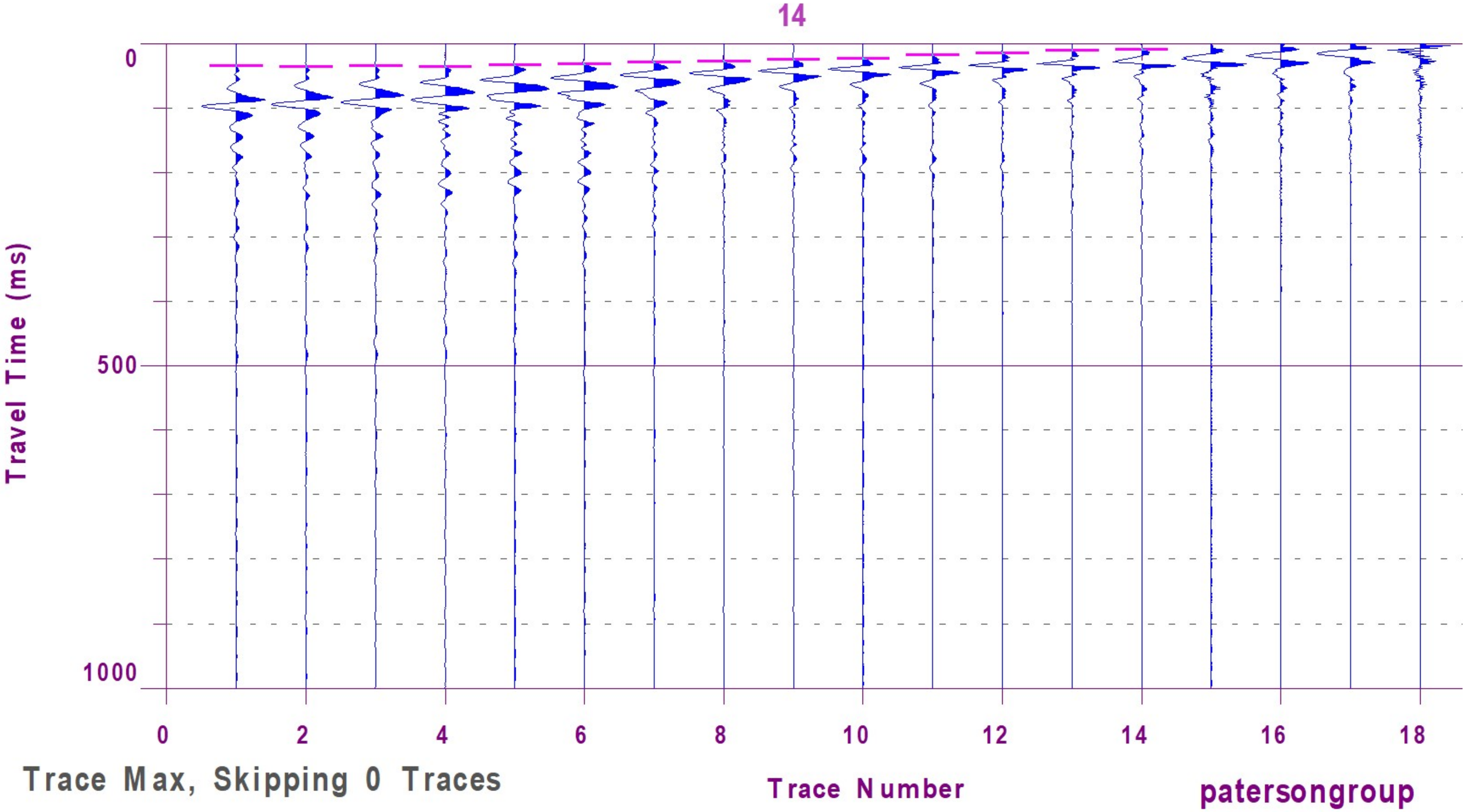
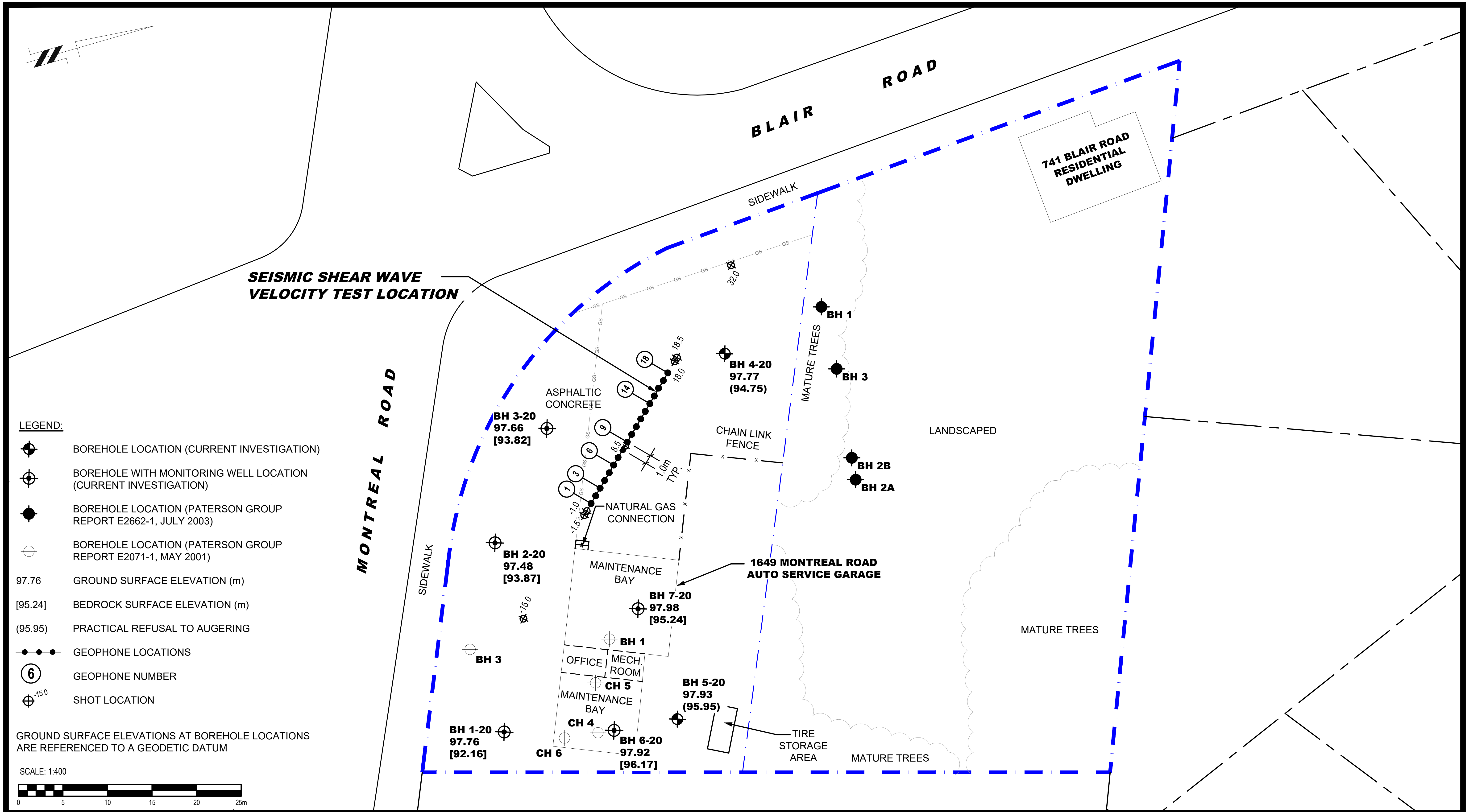


Figure 3 – Shear Wave Velocity Profile at Shot Location 18 m



LEGEND:

⌖	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
●	BOREHOLE LOCATION (PATERSON GROUP REPORT E2662-1, JULY 2003)
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT E2071-1, MAY 2001)
97.76	GROUND SURFACE ELEVATION (m)
[95.24]	BEDROCK SURFACE ELEVATION (m)
(95.95)	PRACTICAL REFUSAL TO AUGERING
●─●─●	GEOPHONE LOCATIONS
⑥	GEOPHONE NUMBER
⌖ -15.0	SHOT LOCATION

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

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7S9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
1	ADDED SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	08/08/2022	FC

10869279 CANADA INC.

GEOTECHNICAL INVESTIGATION
1649 MONTREAL ROAD AND 741 BLAIR ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 01/2021
Drawn by: YA
Report No.: PG5663-1
Checked by: MS
Dwg. No.: PG5663-1
Approved by: DJG
Revision No.: 1

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