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Geotechnical Investigation

Proposed Building Expansion
4837 Albion Road
Ottawa, Ontario

Prepared For

Hard Rock Ottawa

October 30, 2019

Report PG4315-2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Hard Rock Ottawa to conduct a geotechnical investigation for the proposed expansion of the existing building located at 4837 Albion Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ❏ determine the subsurface soil and groundwater conditions based on test hole information.
- ❏ 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. This report contains the findings and includes geotechnical recommendations pertaining to the design and construction of the development as understood at the time of writing this report.

2.0 Proposed Project

Based on conceptual drawings provided, it is our understanding that the proposed building expansion will consist of a multi-storey structure. An associated parking garage, car parking areas, access lanes and landscaped areas are also anticipated for this expansion.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between October 2 and October 8, 2019. At that time, a total of eleven (11) boreholes (BH1-19 to BH11-19) and three (3) test pits (TP1-19 to TP3-19) were completed within the proposed expansion footprint. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and existing underground utilities. Previous geotechnical investigations were completed in November 2017 and August 2018. The locations of the boreholes and test pit are shown on Drawing PG4315-2 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using either a truck or track-mounted auger drill rig operated by a two person crew. The test pits were completed using a rubber-tired backhoe. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The field procedure for test holes consisted of augering or excavating to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). Soil grab samples (G) were recovered at regular depth intervals from the test pit sidewalls. All soil samples were visually inspected and initially classified on site, placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the split-spoon, auger and grab samples were recovered from the test holes are shown as SS, AU and G, 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 thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at BH 1-19, BH 5-19 and BH 7-19. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Diamond drilling was completed at one borehole location to assess the bedrock quality. Rock samples were recovered from BH 4-19 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which the rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

Flexible standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations and ground surface elevation at the test hole locations were surveyed by Paterson field personnel. Ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top of a manhole located within the grassed area north of the existing building. A geodetic elevation of 112.16 m was provided for the TBM by Novatech Engineering Consultants Ltd. The location of the boreholes and the ground surface elevation at each borehole location are presented on Drawing PG4315-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

One (1) 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 sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is occupied by a two storey building with a walk-out basement and horse track, as well as access lanes, parking and landscaped areas. The ground surface at the subject site is relatively flat within the parking area and gradually slopes down eastward towards the horse track within the grass/treed area.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of a thin layer of topsoil or asphalt underlain by a crushed stone fill material with silty sand and gravel. The above noted material is primarily underlain by a compact to dense sand to silty sand deposit. A glacial till deposit was encountered below the silty sand deposit, and was observed to consist of silty sand with gravel, cobbles and boulders. A stiff to very stiff brown silty clay with occasional sand seams was observed within the aforementioned sand deposit at BH 6-19. Practical refusal to DCPT was encountered at depths of 12.1 m, 16.7 m and 16.1 m at BH 1-19, BH 5-19 and BH 7-19, respectively. Specific details of the soil profile at borehole locations are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Bedrock was cored at BH 4-19. Bedrock core samples were observed to consist of interbedded limestone and dolostone, with some shale partings. The recovery values range from 90 to 95%, while the RQD values varied between 81 and 93%. Based on the results the bedrock quality is good to excellent.

Based on available geological mapping, the local bedrock consists of dolomite of the Oxford formation with an anticipated overburden thickness of 15 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the open boreholes following completion of the drilling program. The groundwater level observations are provided on the Soil Profile and Test Data sheets in Appendix 1.

Based on the groundwater and soil observations, such as moisture levels and colour of the recovered soil samples, the long-term groundwater table can be expected at a depth greater than 7 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed building expansion. It is expected that the proposed building can be founded by conventional shallow foundations provided the bearing resistance values are sufficient to handle the design building loads. Alternatively, a raft foundation or end bearing piled foundation should be considered for the proposed building.

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 those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the building structures 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 building areas should be compacted to at least 98% of the 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. Site excavated, brown silty clay under dry conditions and above freezing temperatures and approved by the geotechnical consultant at the time of placement can be used to build up the subgrade level for areas to be paved. The brown silty clay should be placed in maximum 300 mm loose lifts and compacted to a minimum density of 95% of its SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Conventional Shallow Foundation

Footings placed on an undisturbed, very stiff silty clay and/or dense sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**.

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

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.

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 soil bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1.5H:1V, passing through in situ soil or engineered fill of equal or higher capacity as the soil.

Raft Foundation

It is anticipated that a raft foundation will be required to support the multi-storey portions of the proposed structure. For our design calculations, one basement level was assumed. It is expected that the excavation will extend between 3 to 4 m below existing ground surface. The SLS contact pressure can be taken to be **350 kPa**. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken to be **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **350 kPa**. The proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

End Bearing Piled Foundation

Alternatively, consideration could also be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are tabulated below. A resistance factor of 0.4 has been incorporated into the factored at ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 1 - Summary of Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	1100	1720	10	44
245	12	1200	2000	10	53
245	13	1300	2270	10	58

Permissible Grade Raise

A permissible grade raise restriction has been determined for areas of the subject site where a silty clay deposit is present. Based on the testing results, a permissible grade raise restriction of **2 m** above existing ground surface is recommended for areas of the subject site where clay is present.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building addition in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The results of the shear wave velocity testing are attached to the current report.

Field Program

The field program was completed on October 10, 2019 by Paterson personnel. The shear wave velocity testing array was placed near the western side of the subject site, oriented approximately east-west as shown on Drawing PG4315-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the ground surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 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 parallel to the geophone array, which creates a polarized shear wave. The hammer shots are repeated 4 to 8 times at each shot location to improve signal to noise ratio. The shots are also completed in forward and reverse directions (i.e. striking both sides of the I-beam seated parallel to the geophone array). The shots are located 3, 4.5 and 26 m away from the first geophone, 3, 4.5 and 28 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the proposed building addition foundations. 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 shear wave velocity due ot 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 available data, the bedrock is approximately 19.8 m below ground surface. It is understood that the building footings will be placed approximately 3 to 4 m below ground surface. Therefore, the overburden thickness between the underside of footings and the bedrock surface is estimated to be 16.8 m. Based on the test results, the average overburden shear wave velocity is **235 m/s**. Through interpretation, the bedrock shear wave velocity is **2,217 m/s**.

The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012, as presented below.

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

$$V_{s30} = \frac{30m}{\left(\frac{16.8m}{235m/s} + \frac{13.2m}{2217m/s}\right)}$$

$$V_{s30} = 388m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30} for foundations at this site is **388 m/s**. Therefore, a **Site Class C** is applicable for design of the proposed building addition. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Floor Slab

With the removal of topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed building expansion, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill materials within the footprint of the proposed building should be placed in maximum 200 mm thick loose layers and compacted to at least 98% of its SPMDD. It is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone.

5.6 Basement Wall

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 dry unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil can be taken as 13 kN/m^3, where applicable.

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

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 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 $P_o = 0.5 \cdot K_o \cdot \gamma \cdot H^2$, 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 Pavement Structure

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

Table 2 - 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 3 - 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 sub-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 with suitable vibratory equipment.

Parking Lots A and B

For Parking Lots A and B, it is understood that it is proposed to place 200 mm of OPSS Granular A and a double surface treatment over the existing gravel surface. It is further understood that the double surface treatment will be a temporary surface with a service life of 2 years and will be utilized for light duty commercial vehicle traffic.

To evaluate the existing subgrade at Parking Lots A and B, five (5) hand excavated test pits were advanced in this area on September 26, 2019. The test pits were excavated to approximate depths of 0.3 to 0.5 m below the existing ground surface.

The subsurface profile observed within the test pits generally consisted of approximately 100 to 225 mm of OPSS Granular A underlain by a compact, light brown silty sand with trace to some gravel. Groundwater was not observed within the completed test pits.

Based on the results of the test pits completed at the site, the existing OPSS Granular A subgrade at Parking Lots A and B is considered suitable to support the proposed 200 mm thickness of OPSS Granular A and a double surface treatment.

However, prior to placement of the 200 mm of OPSS Granular A, a vibratory drum roller should complete several passes over the existing subgrade surface as a proof-rolling program. Paterson should be on-site to inspect the proof-rolling program, and any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II.

The 200 mm thick layer of OPSS Granular A should then be placed and compacted to at least 98% of its SPMDD, followed by the double surface treatment.

Parking Lot C

It is understood that Parking Lot C will be located at the southwest end of the site, in what is currently a vacant, landscaped area. It is further understood that Parking Lot C will be utilized as a temporary parking lot and staging area during proposed construction works at the site. It is planned to place 250 mm of OPSS Granular B followed by 50 mm of OPSS Granular A for this purpose.

Based on test pits previously completed by Paterson, the subsurface conditions in this portion of the site consist of topsoil underlain by an approximate 0.5 m thickness of granular fill, which is then underlain by a compact silty sand with some cobbles and boulders. The granular fill and/or silty sand are considered suitable subgrades for the temporary Parking Lot C, provided the following recommendations are followed:

❏ Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under the proposed Parking Lot C.

❏ The granular fill or silty sand subgrade should then be proof-rolled using several passes of a vibratory drum roller. Paterson should be on-site to inspect the proof-rolling program, and any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II.

❏ The 250 mm of OPSS Granular B should then be placed and compacted to at least 98% of its SPMDD, followed by the 50 mm of OPSS Granular A which is compacted to at least 98% of its SPMDD.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed building expansion and connected to the existing system. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

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

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as 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

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

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

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

Groundwater Control for Building Construction

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) is not anticipated to be required based on current building details. However, 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.

Also, if typical ground or surface water volumes are 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 MOECC review of the PTTW application.

Impacts on Neighbouring Structures

Based on the existing groundwater level, a groundwater lowering will not take place due to construction of the proposed building expansion. 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 installation 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 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 difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

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 samples 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 a low corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

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

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

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project.

A geotechnical investigation 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, Paterson requests notification immediately in order to permit reassessment of the 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 Hard Rock Ottawa 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.

Nathan Christ

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 2

FILE NO. PG4315

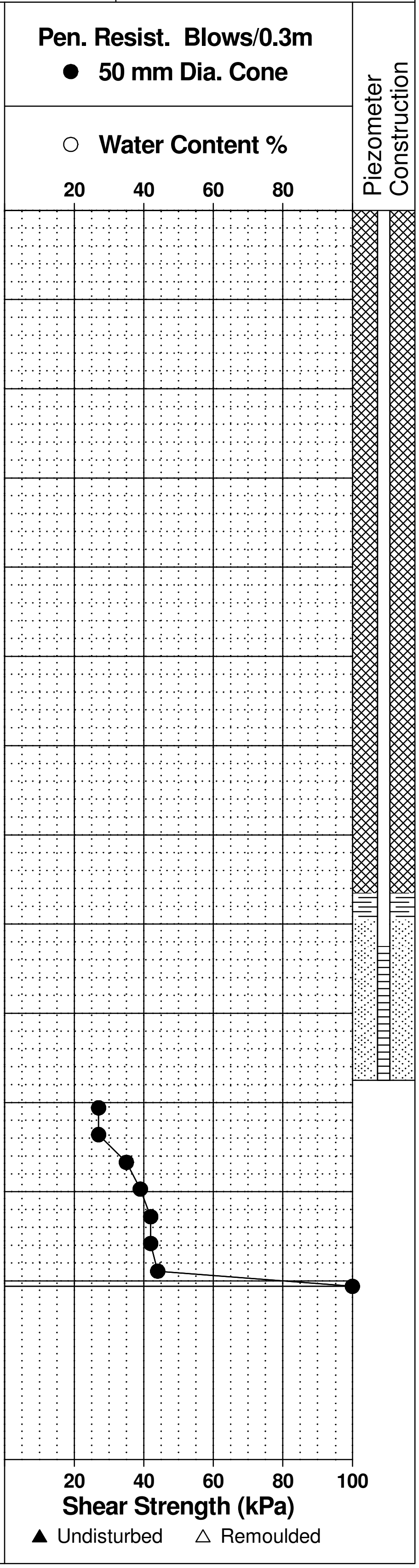
HOLE NO. BH 1-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	114.06
Asphaltic concrete 0.08		AU	1				
FILL: Brown sand with gravel 0.60							
FILL: Brown sand, some silt and gravel 1.62		SS	2	54	22	1	113.06
Compact, brown **SAND** - trace silt by 2.4m depth		SS	3	67	15	2	112.06
		SS	4	62	15	3	111.06
		SS	5	79	13	4	110.06
		SS	6	71	16	5	109.06
		SS	7	79	15	6	108.06
		SS	8	92	17	7	107.06
		SS	9	79	22	8	106.06
		SS	10	75	15	9	105.06
		SS	11	71	25	10	104.06
9.75		SS	13	83	22	11	103.06
Dynamic Cone Penetration Test commenced at 9.75m depth. 12.06						12	102.06

End of Borehole

Practical DCPT refusal at 12.06m depth

(Piezometer dry/blocked at 10.11m depth - Oct. 10, 2019)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 2

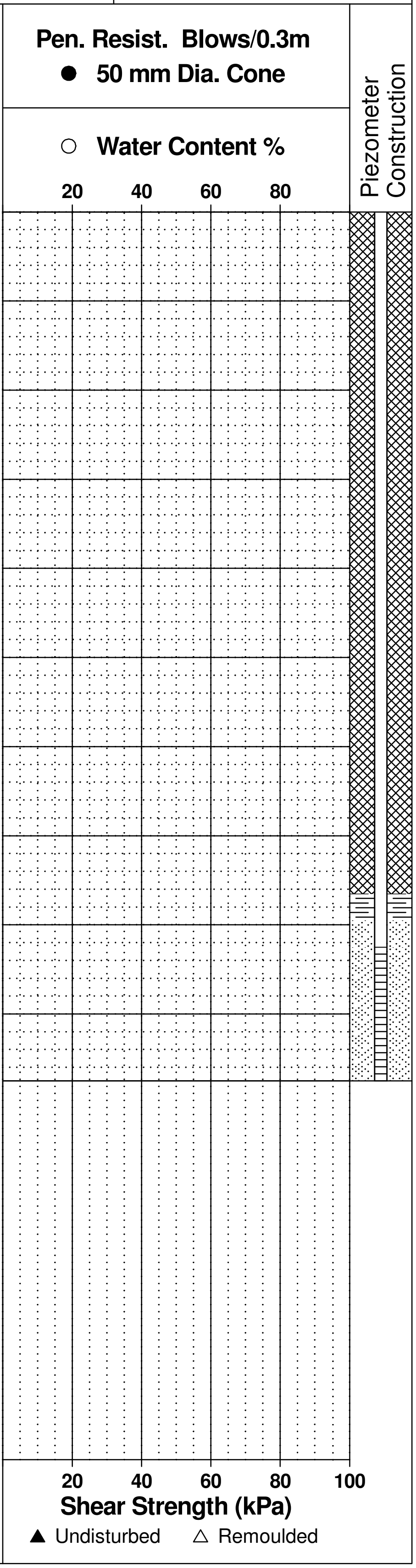
FILE NO. PG4315

HOLE NO. BH 2-19

SOIL DESCRIPTION	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08	AU	1		
FILL: Brown sand, some gravel	0.60				
FILL: Brown silty sand with gravel	1.52	SS	2	71	14
Compact to dense, brown **SAND**	7.06	SS	3	71	17
		SS	4	67	16
		SS	5	75	17
		SS	6	67	26
		SS	7	58	42
		SS	8	67	37
		SS	9	58	41
Dense, brown **SILTY SAND**	9.75	SS	10	75	37
		SS	11	62	26
		SS		88	14
		SS	13	88	34
End of Borehole					

(Piezometer dry/blocked at 9.05m depth - Oct. 10, 2019)

DEPTH (m)	ELEV. (m)
0	114.02
1	113.02
2	112.02
3	111.02
4	110.02
5	109.02
6	108.02
7	107.02
8	106.02
9	105.02



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 2

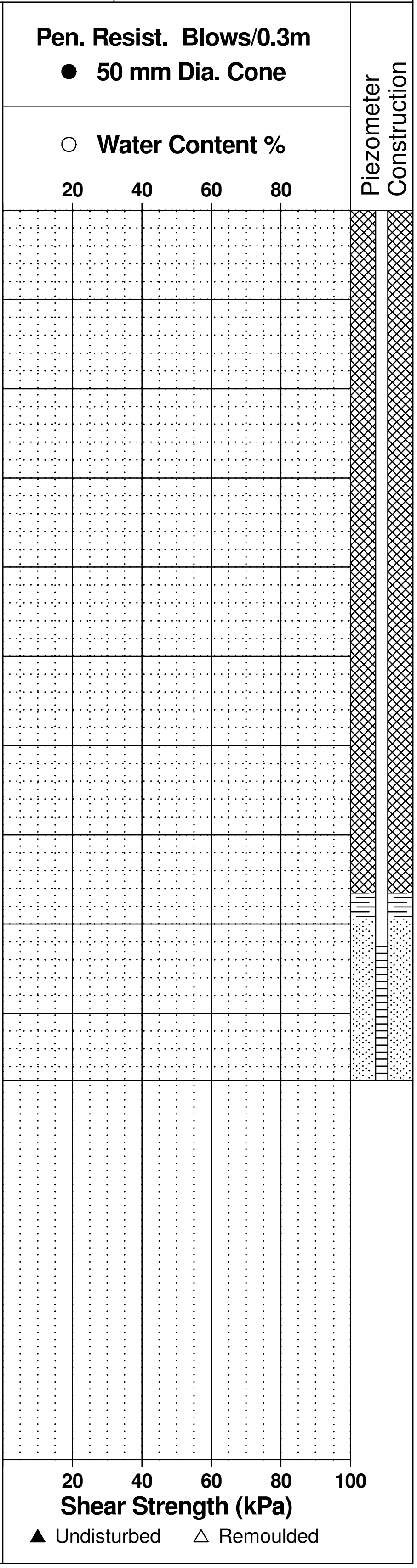
FILE NO. PG4315

HOLE NO. BH 3-19

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown sand, some silt and gravel	0.76
FILL: Brown sand with gravel, trace cobbles and boulders	1.70
Compact, brown **SAND,** trace silt	2.29
Compact, brown **SAND**	7.80
Compact, brown **SANDY SILT**	9.75
End of Borehole (Piezometer dry/blocked at 8.40m depth - Oct. 10, 2019)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	35
SS	3	83	21
SS	4	58	14
SS	5	71	11
SS	6	88	12
SS	7	71	17
SS	8	62	14
SS	9	67	14
SS	10	62	15
SS	11	88	17
SS	12	83	15
SS	13	75	22

DEPTH (m)	ELEV. (m)
0	113.98
1	112.98
2	111.98
3	110.98
4	109.98
5	108.98
6	107.98
7	106.98
8	105.98
9	104.98



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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 3

FILE NO. PG4315

HOLE NO. BH 4-19

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.18
FILL: Brown sand, some gravel	1.52
Compact, light brown **SAND**	1.90
Compact, brown **SANDY SILT**	2.57
Compact, light brown **SAND**	2.82
Compact, brown **SANDY SILT**	4.17
Compact, brown **SAND,** some silt	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	67	9
SS	2	71	9
SS	3	79	14
SS	4	83	30
SS	5	96	24
SS	6	96	23
SS	7	71	25
SS	8	75	19
SS	9	67	38
SS	10	75	44
SS	11	62	37
SS	12	75	26
SS	13	79	26
SS	14	88	26
SS	15		
SS	16	83	12
SS	17	67	26
SS	18	75	19

DEPTH (m)	ELEV. (m)
0	113.71
1	112.71
2	111.71
3	110.71
4	109.71
5	108.71
6	107.71
7	106.71
8	105.71
9	104.71
10	103.71
11	102.71
12	101.71
13	100.71
14	99.71

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

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REMARKS

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DATE 2019 October 3

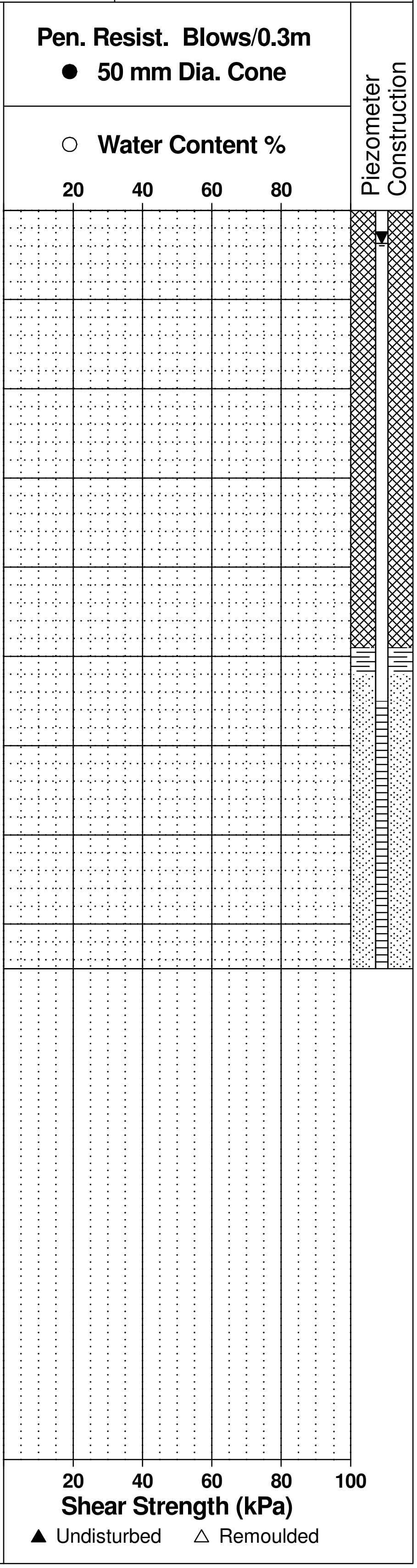
FILE NO. PG4315

HOLE NO. BH 4-19

SOIL DESCRIPTION	Layer bottom (m)
GROUND SURFACE	
Compact, brown **SAND,** some silt	16.66
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders	19.81
BEDROCK: Interbedded limestone and dolostone, with some shale partings	22.50
End of Borehole (GWL @ 14.37m - Oct. 10, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	1	12	0
RC	2	52	81
RC	3	90	80
RC	4	95	93

DEPTH (m)	ELEV. (m)
14	99.71
15	98.71
16	97.71
17	96.71
18	95.71
19	94.71
20	93.71
21	92.71
22	91.71



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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 3

FILE NO. PG4315

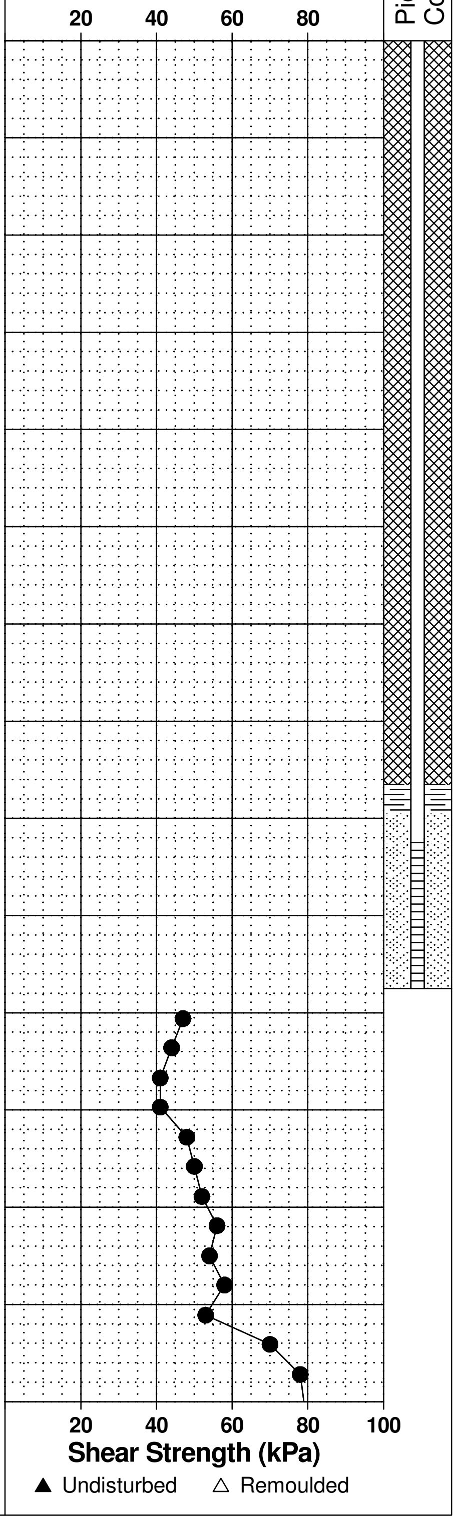
HOLE NO. BH 5-19

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.05	AU	1		
FILL: Brown sand with gravel, some silt 0.76				
Compact, brown **SILTY SAND** 1.52	SS	2	88	13
Compact to dense, brown **SAND,** trace silt	SS	3	96	20
	SS	4	92	20
	SS	5	88	18
	SS	6	83	28
	SS	7	62	40
	SS	8	75	35
	SS	9	67	33
	SS	10	83	28
	SS	11	62	32
	SS	12	75	30
9.75	SS	13	83	43
Dynamic Cone Penetration Test commenced at 9.75m depth				

DEPTH (m)	ELEV. (m)
0	113.42
1	112.42
2	111.42
3	110.42
4	109.42
5	108.42
6	107.42
7	106.42
8	105.42
9	104.42
10	103.42
11	102.42
12	101.42
13	100.42
14	99.42

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

○ **Water Content %**



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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REMARKS

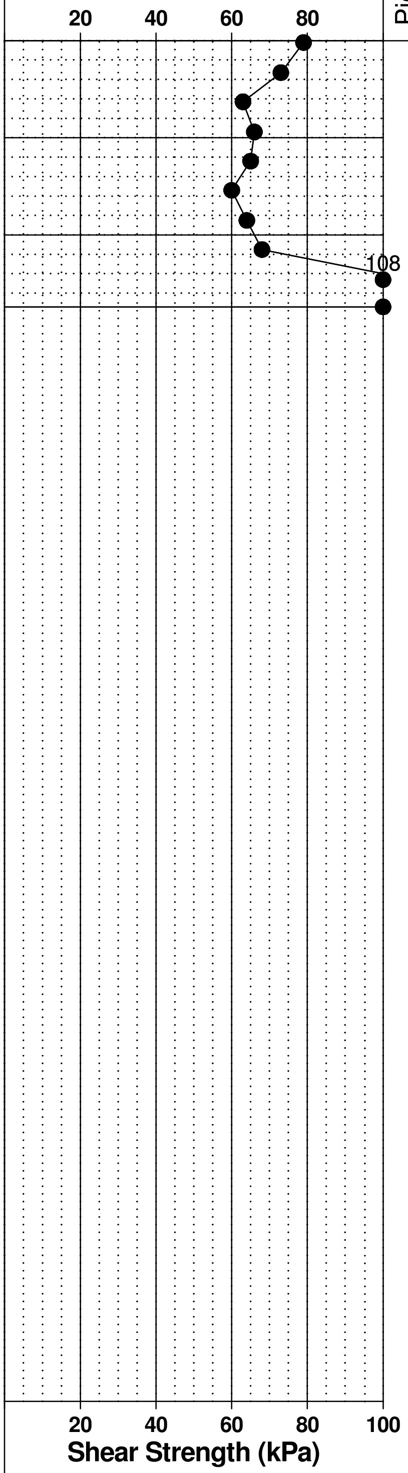
BORINGS BY CME 55 Power Auger

DATE 2019 October 3

FILE NO. PG4315

HOLE NO. BH 5-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						14	99.42		
						15	98.42		
						16	97.42		
16.74									
End of Borehole Practical DCPT refusal at 16.74m depth (Piezometer dry/blocked at 8.96m depth - Oct. 10, 2019)									



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

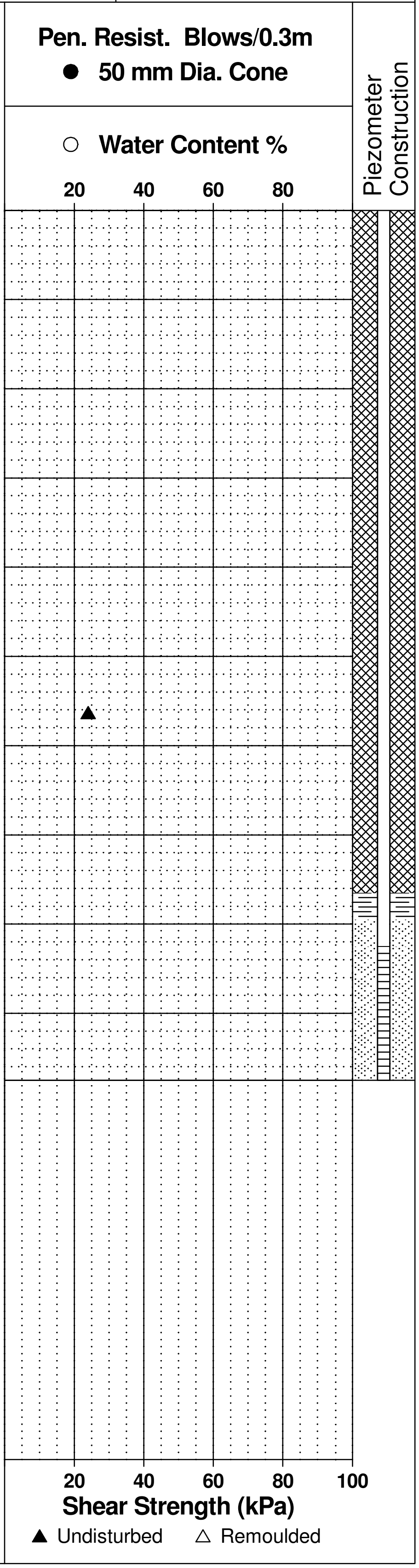
DATE 2019 October 4

FILE NO. PG4315

HOLE NO. BH 6-19

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown sand with gravel	0.60	AU	1		
Compact, light brown **SAND**		SS	2	75	12
	2.13	SS	3	62	19
Brown **SILTY CLAY** with sand seams, trace to some sand		SS	4	88	9
	3.40	SS	5	100	10
Compact, light brown **SAND**	3.56				
Brown **SILTY CLAY,** trace sand		SS	6	79	21
		SS	7	100	8
	5.69	SS	8	92	21
Compact to dense, light brown **SAND**		SS	9	67	42
		SS	10	71	41
	9.75	SS	11	58	28
End of Borehole (Piezometer dry/blocked at 7.76m depth - Oct. 10, 2019)					

DEPTH (m)	ELEV. (m)
0	113.15
1	112.15
2	111.15
3	110.15
4	109.15
5	108.15
6	107.15
7	106.15
8	105.15
9	104.15



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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

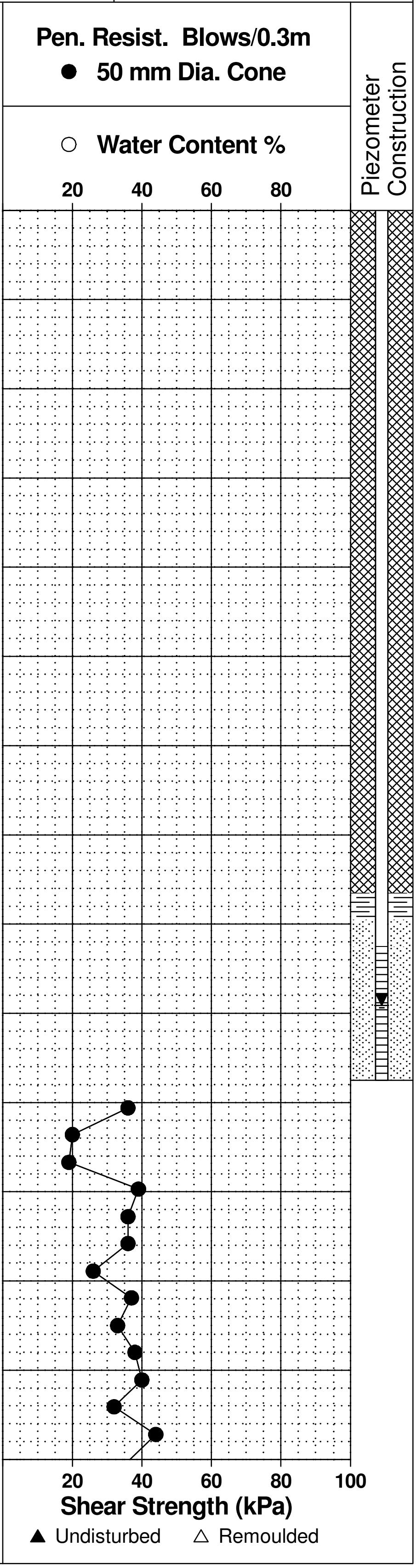
DATE 2019 October 4

FILE NO. PG4315

HOLE NO. BH 7-19

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Brown sand, some gravel	1.37	SS	2	83	11
FILL: Brown silty sand with clay and gravel	1.65	SS	3	75	14
Compact, brown **SAND** - with gravel by 4.6m depth	9.14	SS	4	0	19
		SS	5	67	10
		SS	6	75	14
		SS	7	67	17
		SS	8	71	21
		SS	9	67	34
		SS	10	67	22
Compact, brown **SANDY SILT**	9.75	SS	11	79	17
Dynamic Cone Penetration Test commenced at 9.75m depth					

DEPTH (m)	ELEV. (m)
0	112.78
1	111.78
2	110.78
3	109.78
4	108.78
5	107.78
6	106.78
7	105.78
8	104.78
9	103.78
10	102.78
11	101.78
12	100.78
13	99.78
14	98.78



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

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REMARKS

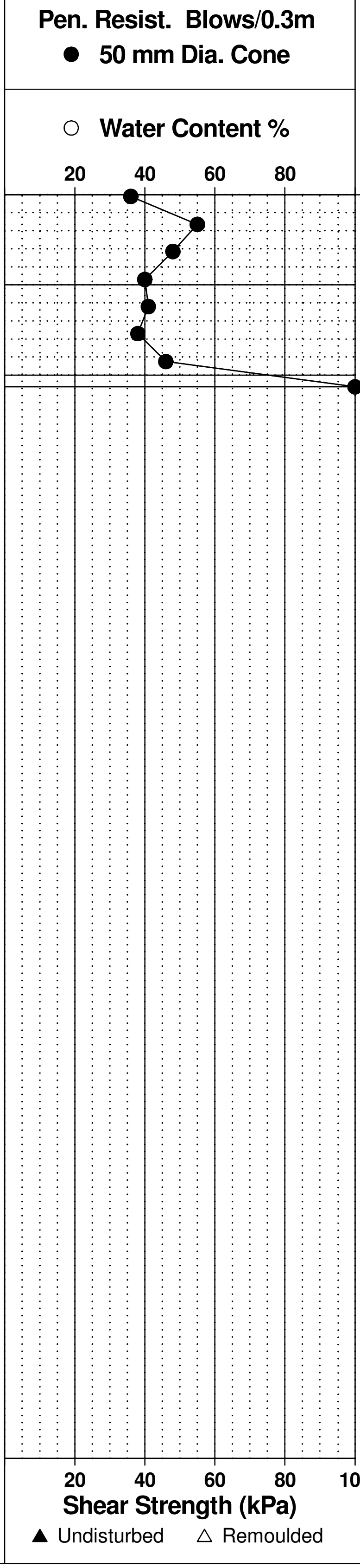
BORINGS BY CME 55 Power Auger

DATE 2019 October 4

FILE NO. PG4315

HOLE NO. BH 7-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
						14	98.78	
						15	97.78	
16.13						16	96.78	
End of Borehole Practical DCPT refusal at 16.13m depth (GWL @ 8.91m - Oct. 10, 2019)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

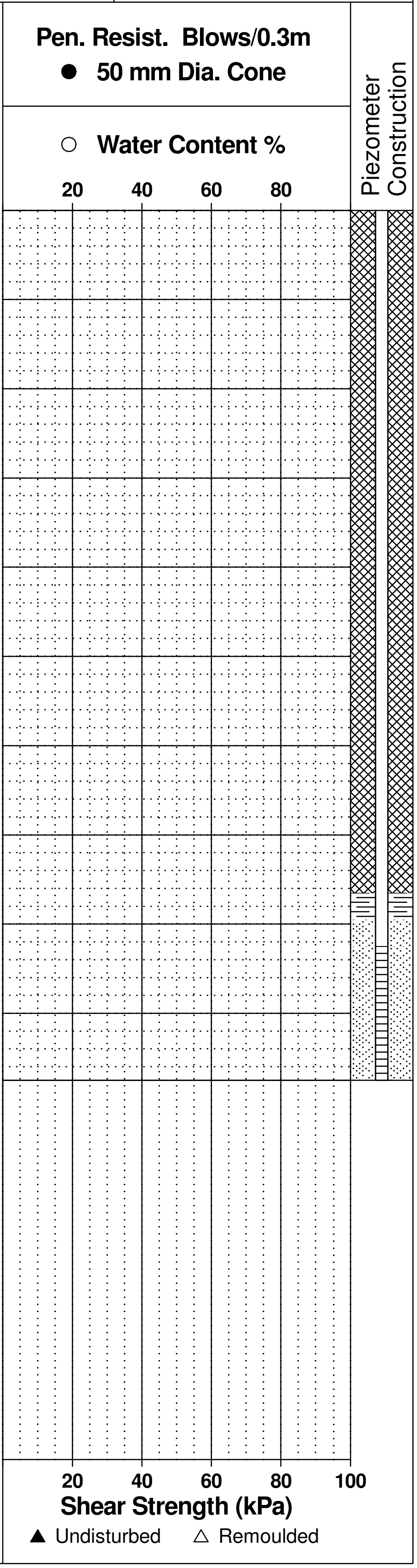
DATE 2019 October 4

FILE NO. PG4315

HOLE NO. BH 8-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete 0.05					
FILL: Brown sand with gravel 0.60		AU	1		
Compact to dense, brown **SAND**		SS	2	58	24
		SS	3	67	22
		SS	4	75	25
		SS	5	79	38
		SS	6	75	58
		SS	7	83	46
		SS	8	75	30
		SS	9	79	36
- some silt by 7.6m depth		SS	10	88	49
9.75		SS	11	92	38
End of Borehole (Piezometer dry/blocked at 8.85m depth - Oct. 10, 2019)					

DEPTH (m)	ELEV. (m)
0	113.03
1	112.03
2	111.03
3	110.03
4	109.03
5	108.03
6	107.03
7	106.03
8	105.03
9	104.03



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 4

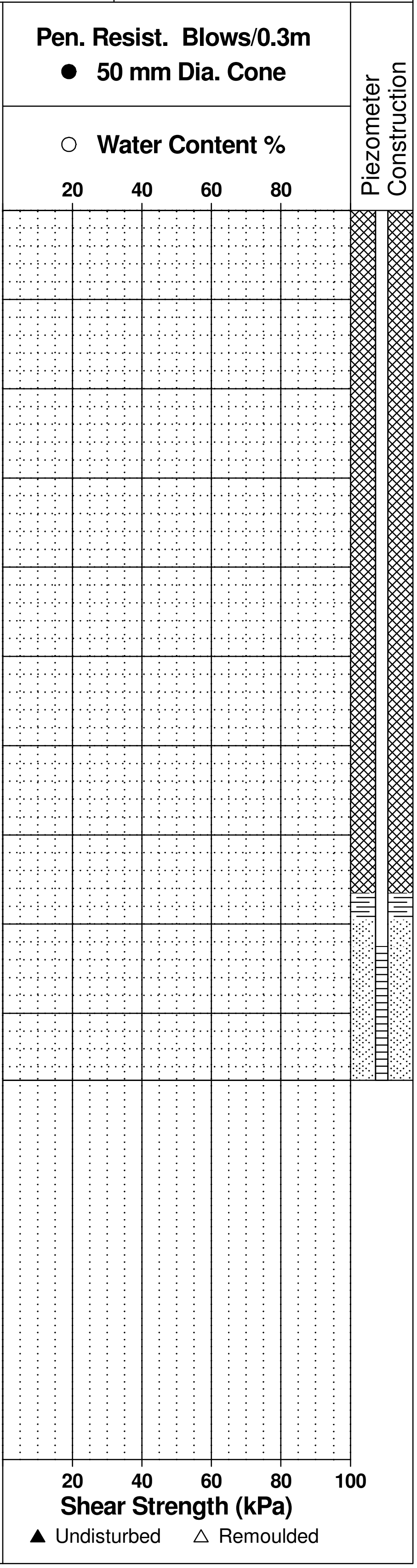
FILE NO. PG4315

HOLE NO. BH 9-19

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	113.62
Asphaltic concrete 0.20	AU	1				
FILL: Brown sand with gravel 1.52	SS	2	62	21	1	112.62
Compact to dense, brown **SAND,** trace silt	SS	3	83	20	2	111.62
	SS	4	83	23	3	110.62
	SS	5	79	28	4	109.62
	SS	6	75	27	5	108.62
	SS	7	88	41	6	107.62
	SS	8	62	38	7	106.62
	SS	9	79	35	8	105.62
	SS	10	62	39	9	104.62
9.75	SS	11	79	30		

End of Borehole

(Piezometer dry/blocked at 8.85m depth - Oct. 10, 2019)



SOIL PROFILE AND TEST DATA

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Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 8

FILE NO. PG4315

HOLE NO. BH10-19

GROUND SURFACE

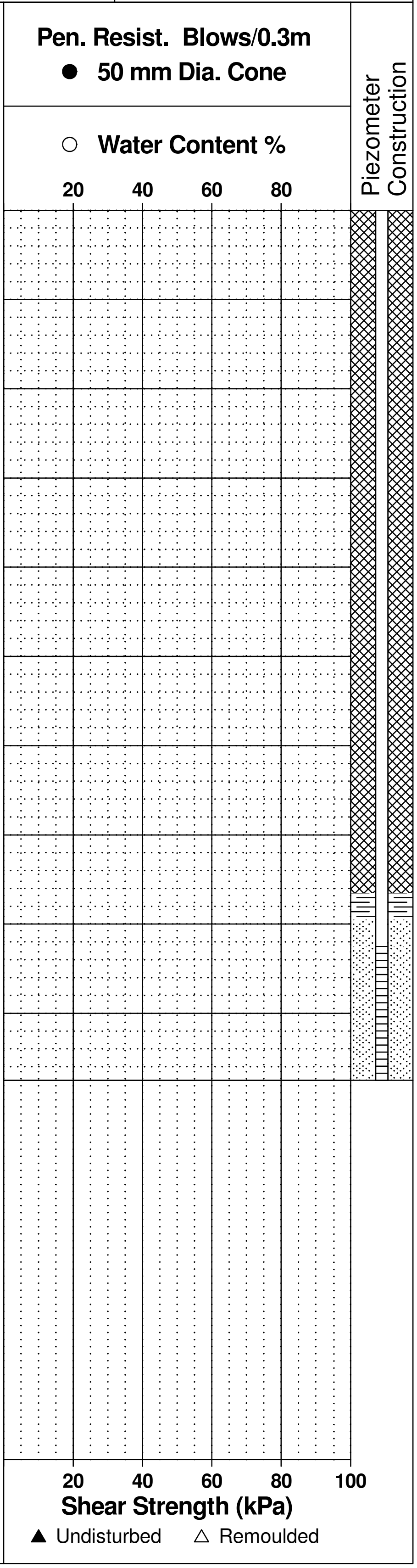
SOIL DESCRIPTION	Depth to base (m)
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel	0.60
FILL: Brown silty sand with clay and gravel	1.17
FILL: Brown sand with gravel	1.52
Compact to dense, light brown **SAND,** trace silt	9.24
Compact, brown **SILT,** some sand	9.75

End of Borehole

(Piezometer dry/blocked at 9.03m depth - Oct. 10, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	18
SS	3	67	29
SS	4	58	31
SS	5	71	19
SS	6	83	22
SS	7	67	25
SS	8	62	30
SS	9	79	29
SS	10	67	34
SS	11	92	21

DEPTH (m)	ELEV. (m)
0	114.13
1	113.13
2	112.13
3	111.13
4	110.13
5	109.13
6	108.13
7	107.13
8	106.13
9	105.13



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 8

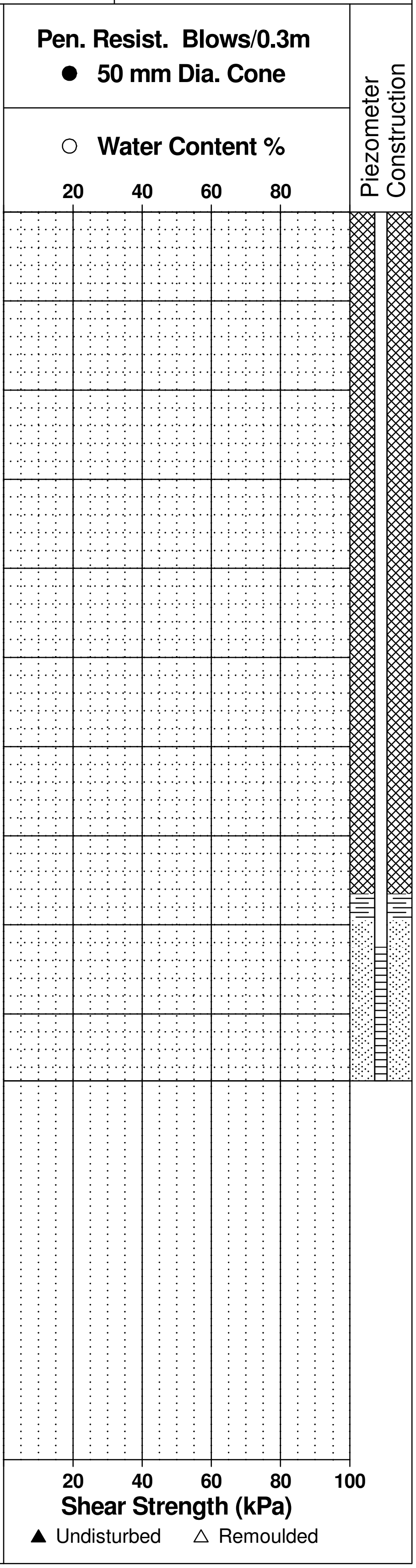
FILE NO. PG4315

HOLE NO. BH11-19

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with gravel	0.60
FILL: Brown sand, some gravel	1.14
Compact, light brown **SAND,** trace silt	8.03
Compact, brown **SILT,** some sand	9.75
End of Borehole (Piezometer dry/blocked at 8.42m depth - Oct. 10, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	23
SS	3	71	20
SS	4	67	24
SS	5	79	24
SS	6	75	33
SS	7	83	24
SS	8	71	25
SS	9	79	24
SS	10	62	22
SS	11	100	24

DEPTH (m)	ELEV. (m)
0	113.62
1	112.62
2	111.62
3	110.62
4	109.62
5	108.62
6	107.62
7	106.62
8	105.62
9	104.62



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REMARKS

BORINGS BY Backhoe

DATE 2019 October 2

FILE NO. PG4315

HOLE NO. TP 1-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	113.91		
TOPSOIL	■	G	1						
0.30									
FILL: Brown silty sand with gravel, some cobbles	▨								
	▨					1	112.91		
	▨	G	2						
1.88									
End of Test Pit (TP dry upon completion)									

20 40 60 80

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

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Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 2019 October 2

FILE NO. **PG4315**

HOLE NO. **TP 2-19**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	114.01		
TOPSOIL	■	G	1						
0.30									
End of Test Pit									

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
Proposed Building Addition - 4837 Albion Road
Ottawa, Ontario

DATUM TBM - Top of manhole cover. Geodetic elevation = 112.16m as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 2019 October 2

FILE NO. PG4315

HOLE NO. TP 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	113.92		
TOPSOIL	■	G	1						
0.30									
FILL: Brown silty sand with gravel, some cobbles	▨	G	2			1	112.92		
1.83									
End of Test Pit (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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)

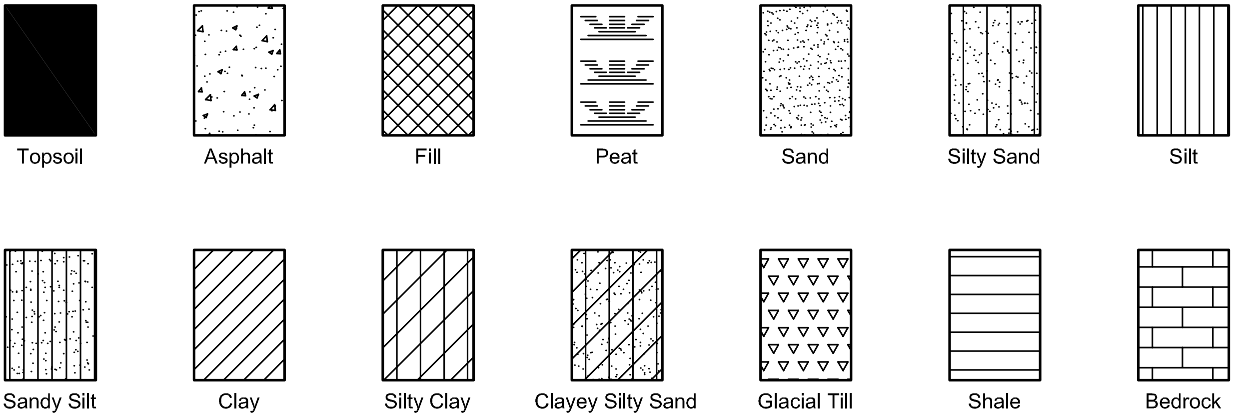
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

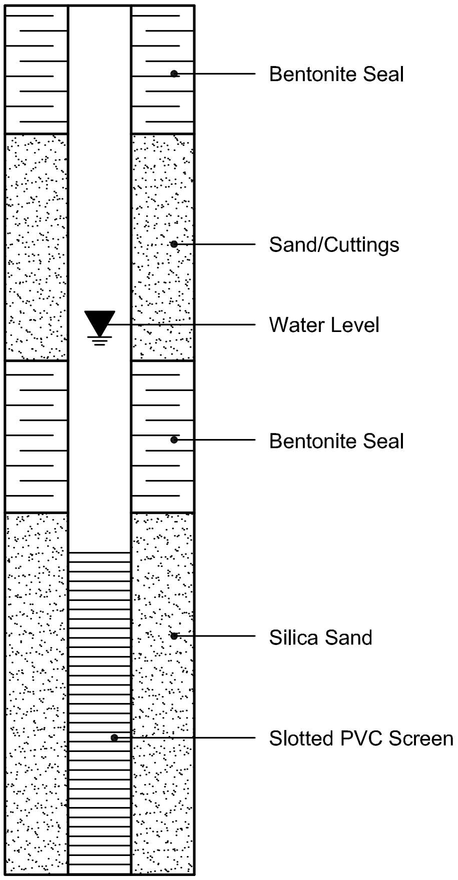
SYMBOLS AND TERMS (continued)

STRATA PLOT

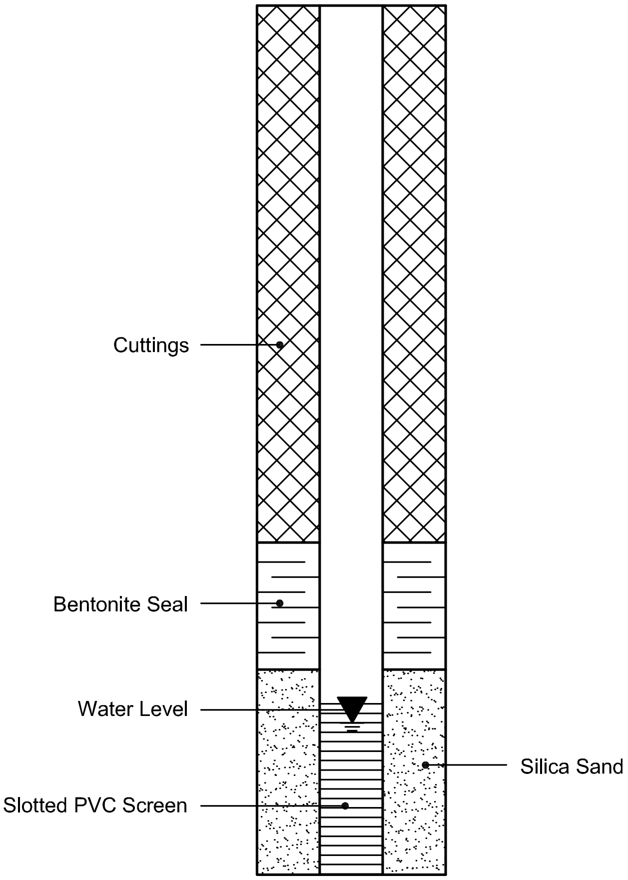


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 23087

Report Date: 10-Nov-2017

Order Date: 8-Nov-2017

Project Description: PG4315

	Client ID:	BH6-SS3	-	-	-
	Sample Date:	08-Nov-17	-	-	-
	Sample ID:	1745368-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	94.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.12	-	-	-
Resistivity	0.10 Ohm.m	10.1	-	-	-
Anions					
Chloride	5 ug/g dry	299	-	-	-
Sulphate	5 ug/g dry	82	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4315-1 - TEST HOLE LOCATION PLAN

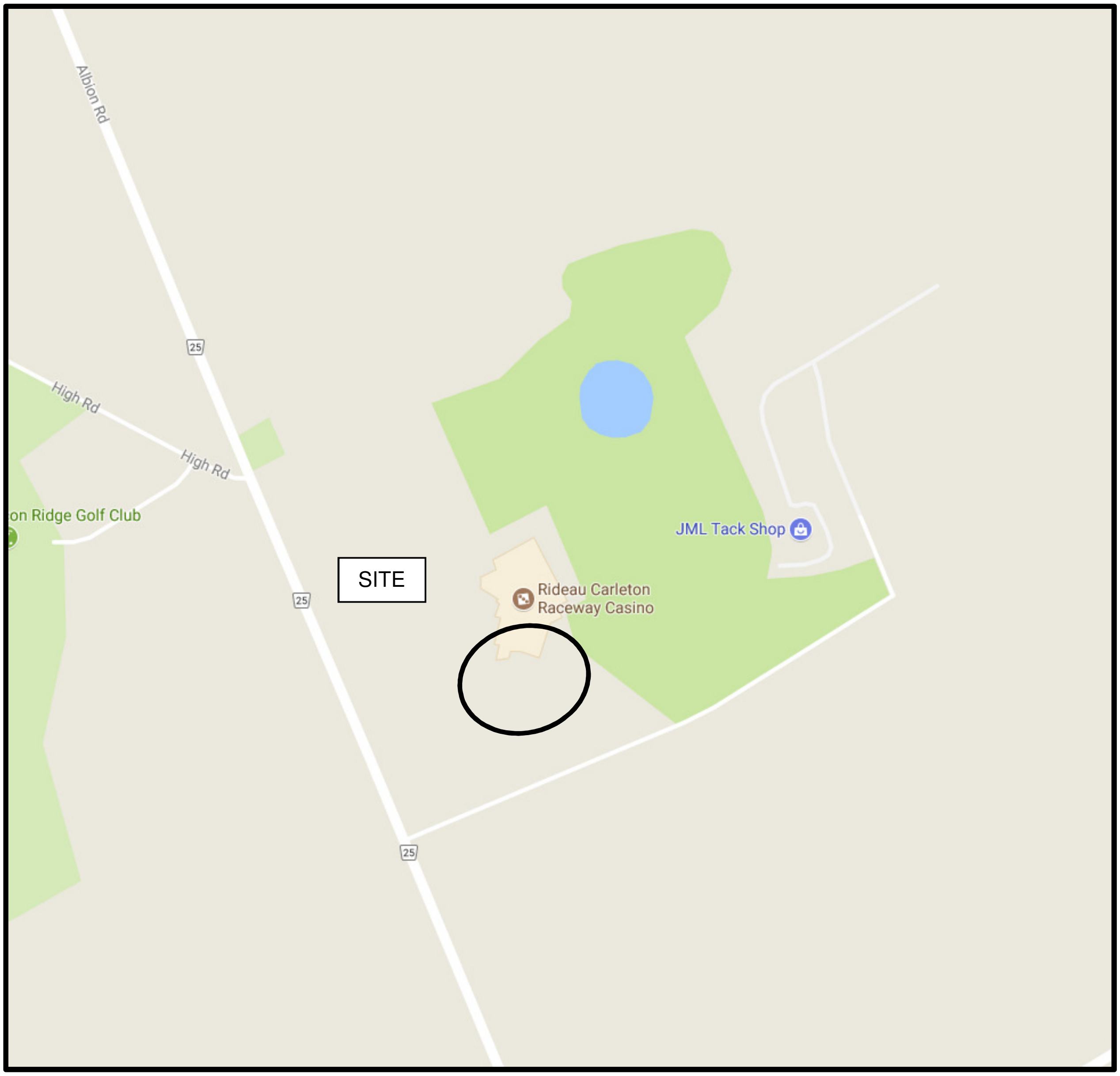


FIGURE 1

KEY PLAN

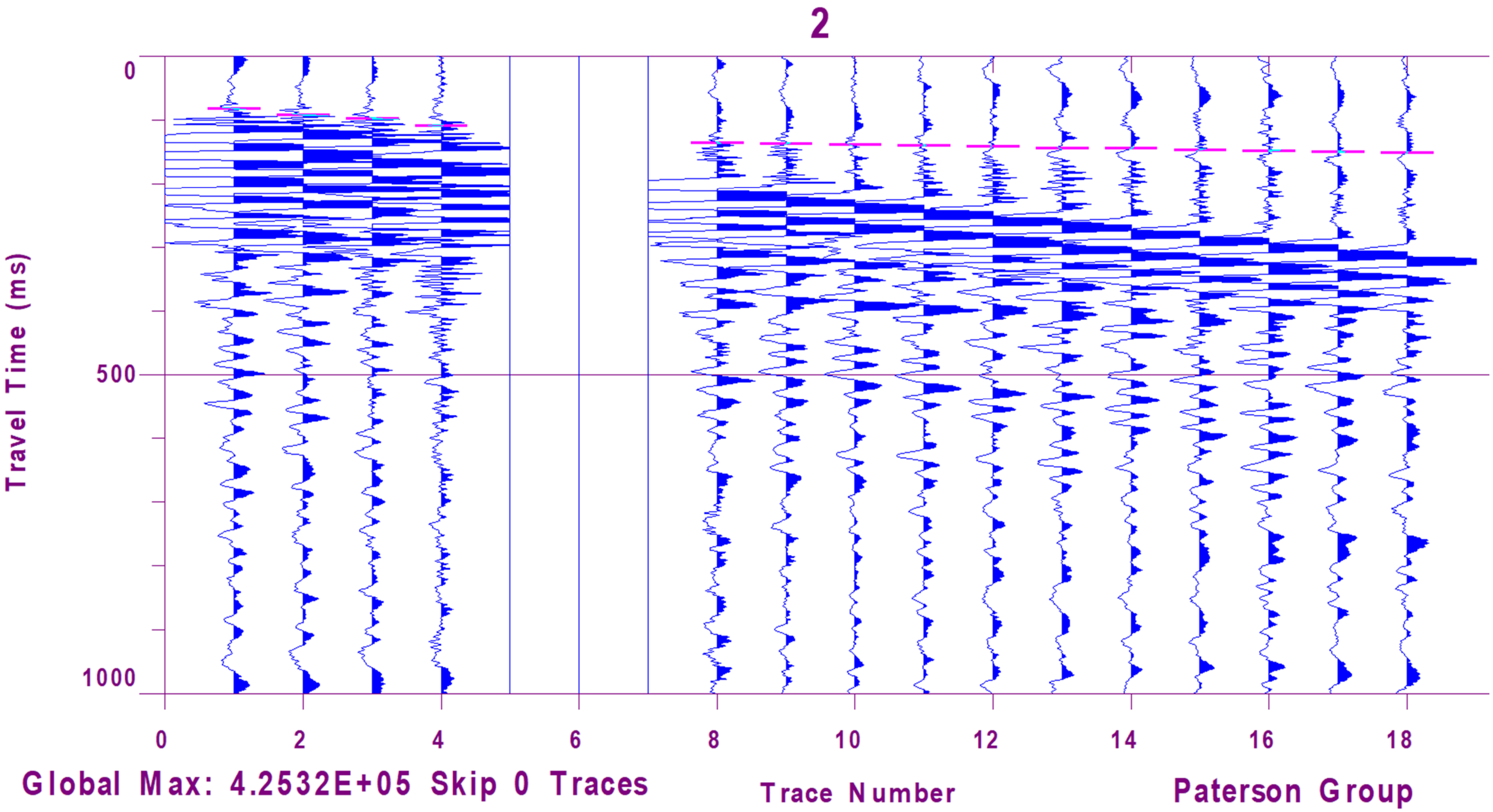


FIGURE 2 – Shear Wave Velocity Profile at Shot Location -26 m

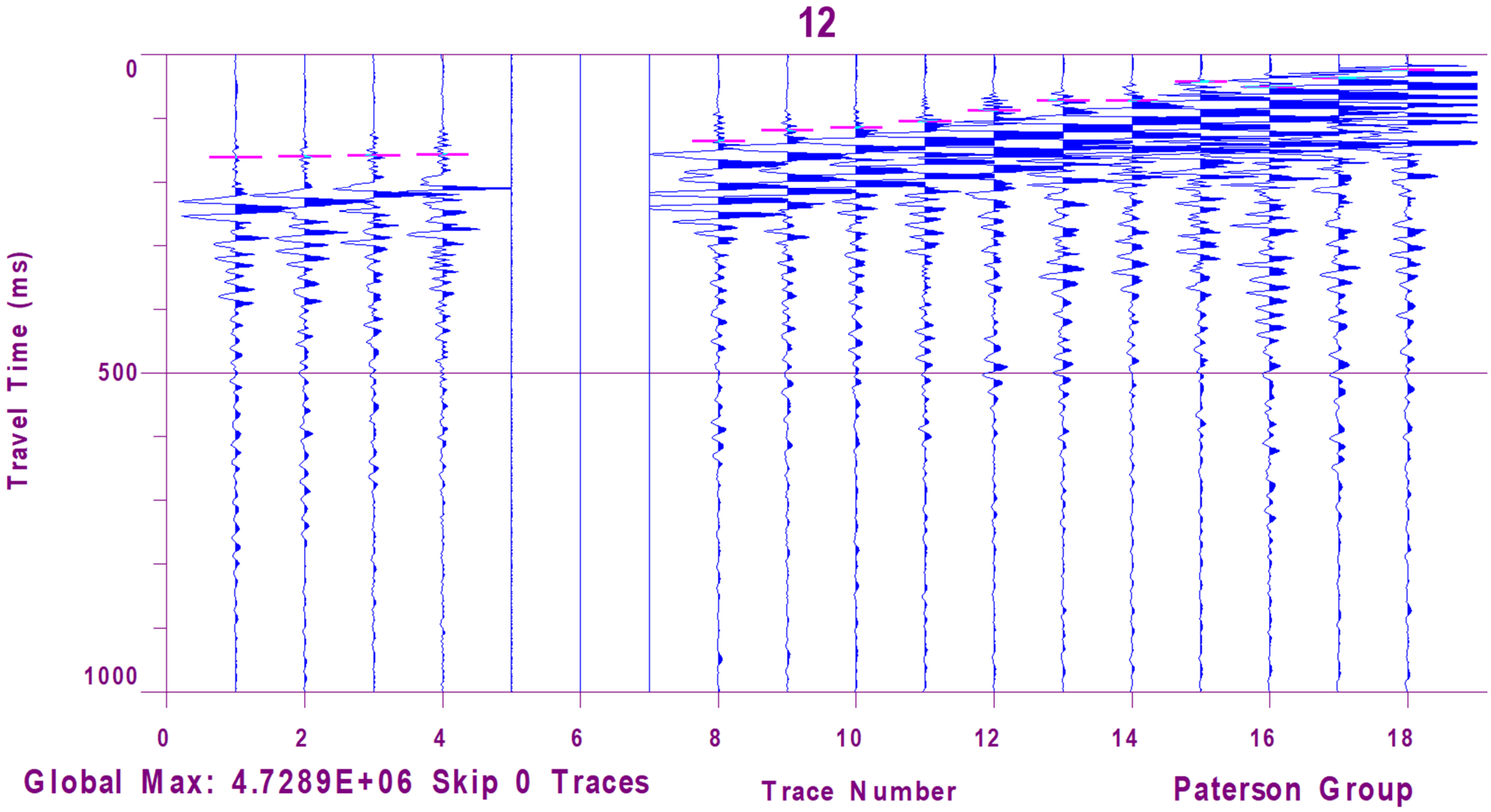
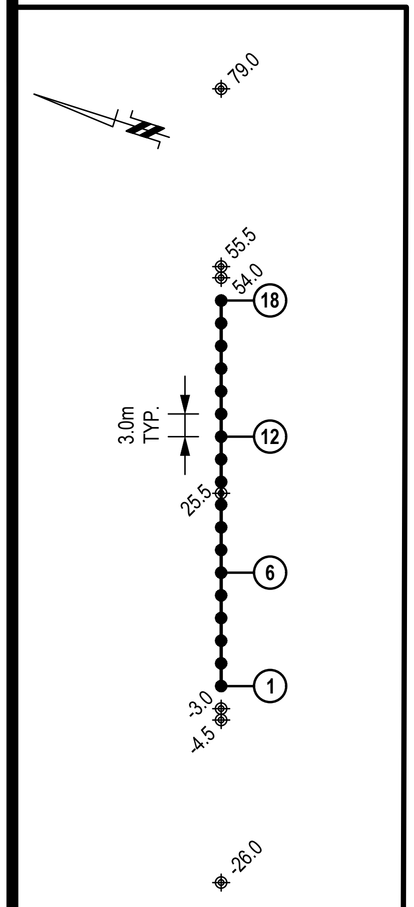
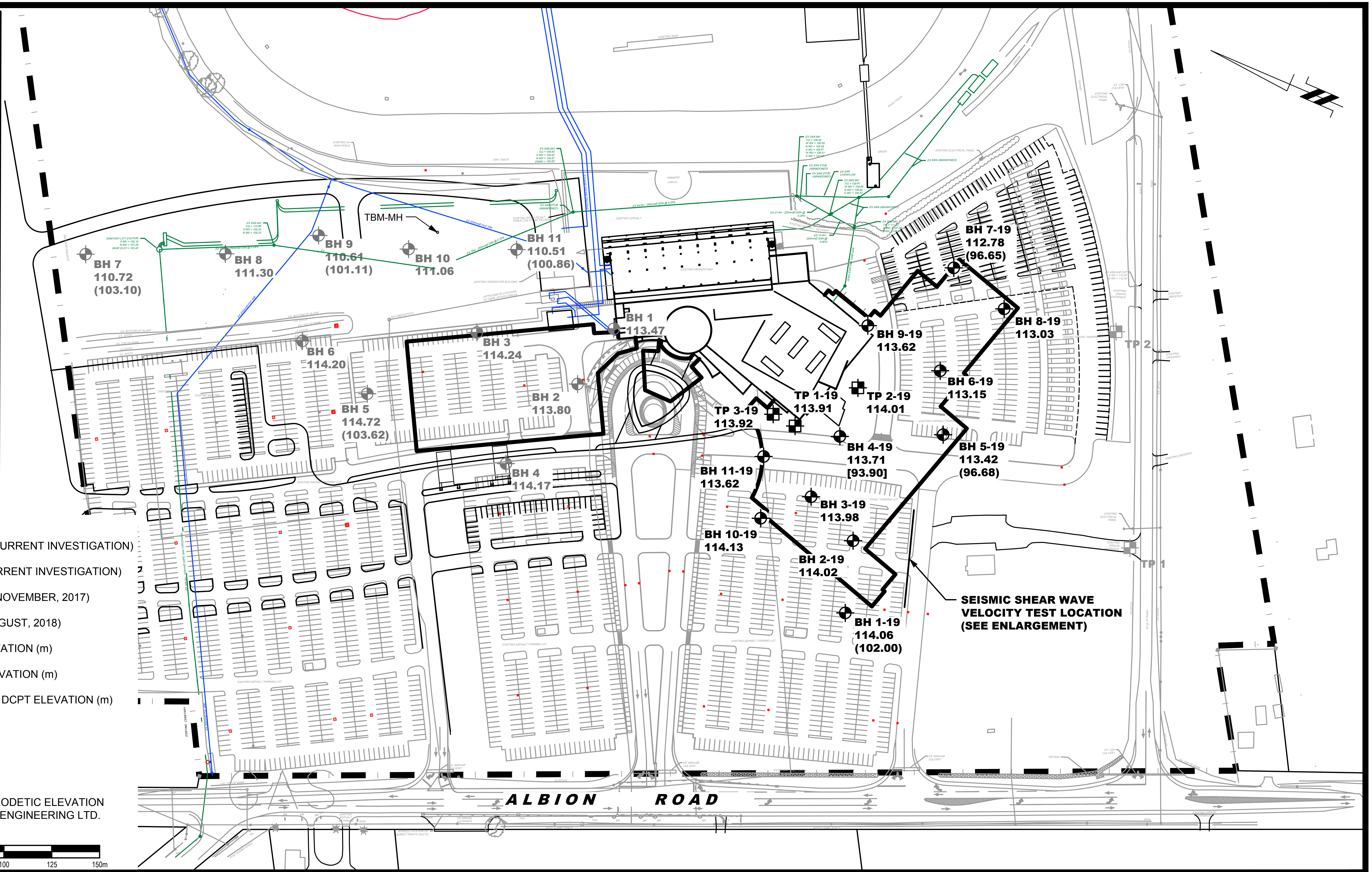


FIGURE 3 – Shear Wave Velocity Profile at Shot Location 55.5 m



ENLARGEMENT
1:1000



LEGEND:

⌖	BOREHOLE LOCATION (CURRENT INVESTIGATION)
▣	TEST PIT LOCATION (CURRENT INVESTIGATION)
⌖	BOREHOLE LOCATION (NOVEMBER, 2017)
▣	TEST PIT LOCATION (AUGUST, 2018)
114.06	GROUND SURFACE ELEVATION (m)
[93.90]	BEDROCK SURFACE ELEVATION (m)
(102.0)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
⑱	GEOPHONE NUMBER
⌖ -3.0	SHOT LOCATION

TBM - TOP OF MANHOLE COVER . GEODETIC ELEVATION =112.16m. PROVIDED BY NOVATECH ENGINEERING LTD.

SCALE: 1:2000

0 25 50 75 100 125 150m

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NO.	REVISIONS	DATE	INITIAL
1	NEW BOREHOLES AND TEST PITS ADDED	10/22/2019	NC

HARD ROCK OTTAWA
GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING EXPANSION - 4837 ALBION ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:2000
Date: 11/2017
Drawn by: YA
Report No.: PG4315-1
Checked by: NC
Dwg. No.: PG4315-1
Approved by: DJG
Revision No.: 1

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