Geotechnical Engineering

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Geotechnical Investigation

Proposed Commercial Building
Kanata West Business Park - Phase 4
Block 2 - Nipissing Court
Ottawa, Ontario

Prepared For

1497328 Ontario Inc. c/o
Taggart Realty Management

February 13, 2020

Report: PG5235-1

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 1497328 Ontario Inc. care of Taggart Realty Management to conduct a geotechnical investigation for the proposed commercial building to be located at Block 2 within Phase 4 of the Kanata West Business Park located along Nipissing Court, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

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 its design.

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

2.0 Proposed Development

It is understood that the proposed commercial building will consist of a slab-on-grade commercial building with associated office areas. The office areas are anticipated to occupy the ground and mezzanine levels. It is further understood that associated access lanes, loading and parking areas and some landscaped areas will occupy the remainder of the subject site. It is anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current geotechnical investigation was conducted on January 28, 2020 and consisted of advancing three (3) boreholes to a maximum depth of 6.1 m below the existing ground surface. A previous field program was carried out by Paterson on November 30, 2010 to provide general coverage of the subject site and surrounding area which included one (1) borehole adjacent to the subject site. The existing borehole was advanced to a depth of 7 m below the existing ground surface. The locations of the test holes are shown on Drawing PG5235-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted 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. The test hole procedures consisted of advancing the boreholes to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered during drilling from the auger flights or a 50 mm diameter split-spoon sampler. The split-spoon samples were classified on site and placed in sealed plastic bags. All samples from all field investigations were transported to our laboratory. The depths at which the auger flight and split-spoon samples were recovered are depicted as AU, SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

The thickness of the overburden was evaluated by dynamic cone penetration testing (DCPT) at BH 2-20. The DCPT consists of driving a steel drill road, equipped with a 50 mm diameter cone at the tip, using at 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.

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

Groundwater

Flexible piezometers were installed in all borehole locations to permit the monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson personnel. The location and ground surface elevations at the borehole locations were surveyed and referenced to a geodetic datum using a Trimble GPS unit. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG5235-1 Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the boreholes and visually examined in our laboratory to review the 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

Generally, the ground surface across the subject site is relatively flat and covered with brush and grassed areas. The ground surface located within Nipissing Court is approximately 1 m above the grade of the subject site. The site is currently bordered by a distribution center to the west, vacant land to the north and east and a storm water management pond to the south.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of topsoil underlain by a loose to compact, silty sand to sandy silt layer. A glacial till deposit was encountered underlying the sandy silty in all of the test holes. The glacial till deposit was observed to consist of a compact to dense, grey silty sand with clay, gravel, cobbles and boulders. Practical refusal to augering was encountered in BH1-10 at a depth of 7.0 m. Practical refusal to DCPT was encountered at a depth of 6.8 m at BH2-20. 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.

Bedrock

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation with an overburden drift thickness ranging from 10 to 15 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in the piezometers at the borehole locations on January 28, 2020 and December 22, 2010 for current and previous investigations, respectively. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Measured Groundwater Levels				
Test Hole Location	**Ground Surface Elevation (m)**	**Groundwater Level**		**Date**
		Depth (m)	**Elevation (m)**	
BH 1-20	105.38	2.90	102.48	February 4, 2020
BH 2-20	104.86	blocked	-	February 4, 2020
BH 3-20	105.11	blocked	-	February 4, 2020
BH 1-10	104.87	0.90	103.97	December 22, 2010
Note: - The ground surface elevations are referenced to a geodetic datum.				

It should be noted that groundwater measurements can be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater table can also be estimated based on consistency, moisture levels and colour of the recovered soil samples. Based on these observations, the long-term groundwater level is estimated at a depth ranging between 2 to 3 m below existing grade. It should be noted that the groundwater is subject to seasonal fluctuations and therefore, groundwater could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed development. It is expected that the proposed building will be constructed with conventional shallow footings bearing on an undisturbed, compact sandy silt/silty sand bearing surface.

Due to the absence of sensitive silty clay within the overburden, permissible grade raise restrictions are not applicable for the subject site from a geotechnical perspective.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil 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 footprints, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. 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

Shallow Footings

Footings placed on an undisturbed, compact sandy silt/silty sand bearing surface can be designed using a factored bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a bearing resistance value at ultimate limit states (ULS) of **180 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Where the sandy silt/silty sand bearing surface is found to be in a loose state of compactness, the area should be proof-rolled using a vibratory compactor and approved by the geotechnical consultant prior to placing footings.

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

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

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 compact sandy silt/silty sand above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. Two shear wave velocity profiles analysed as part of our study are presented in Appendix 2.

Field Program

The seismic array testing location was placed within the east portion of the site as depicted on Drawing PG5235-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 18 horizontal 4.5 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 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, which creates a polarized shear wave. The hammer shots are repeated between five to ten times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3, 4.5 and 12 m away from the first, 3, 4.5 and 30 m away from the last geophone, and at the centre of the seismic array.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Based on the test results, the overburden soils have an average shear wave velocity of 265 m/s. The bedrock has an average shear wave velocity of 2,593m/s. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012.

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

$$V_{s30} = \frac{30m}{\left(\frac{6m}{265m/s} + \frac{24m}{2{,}593m/s}\right)}$$

$$V_{s30} = 940m/s$$

Based on the results of the seismic shear wave velocity testing, the average shear wave velocity, Vs_{30}, was calculated to be 940 m/s for an anticipated underside of footing at an approximate geodetic elevation of 104.70 m. Although the average shear wave velocity is sufficient for a Site Class B, as per Note 1 of Table 4.1.8.4.A of the OBC 2012, "site Classes A and B, hard rock and rock are not to be used if there is more than 3 m of softer materials between the rock and the underside of footing or mat foundations." Therefore, a **Site Class C** is applicable for design of the proposed building. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious materials, within the proposed building footprint, the native soil or approved engineered fill, free of organic and deleterious materials, and approved by the geotechnical consultant at the time of construction is considered to be an acceptable subgrade surface on which to commence backfilling for slab on grade construction. The upper 200 mm of sub-slab fill should consist of an OPSS Granular A crushed stone for slab-on-grade construction. All backfill material within the proposed building footprint should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the SPMDD.

Any loose areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

Where required at the subject site, the pavement structures for car only parking areas, access lanes and heavy truck parking areas are shown on Tables 2 and 3.

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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure
Access Lanes and Heavy Truck Loading/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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended but is considered optional for the proposed slab-on-grade building. If implemented, the system should consist of a 150 mm diameter, geotextile wrapped, 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 structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

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

Concrete Sidewalks Adjacent to Building

To avoid differential settlements within the proposed sidewalls adjacent to the proposed building, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks adjacent to the building footprint to consist of non-frost susceptible material such as OPSS Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to 98% of the material's SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the building's perimeter drainage pipe. Consideration should be given to placing a rigid insulation layer below the granular fill layer to prevent frost heave issues at the building entrances.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 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

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

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 subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

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.

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

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate 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) 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.

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.

6.7 Corrosion Potential and Sulphate

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

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.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation of this nature is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 1497328 Ontario 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.

LICENSED PROFESSIONAL ENGINEER
Feb. 12, 2020
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APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - Block 2
Ottawa, Ontario

DATUM Geodetic

REMARKS

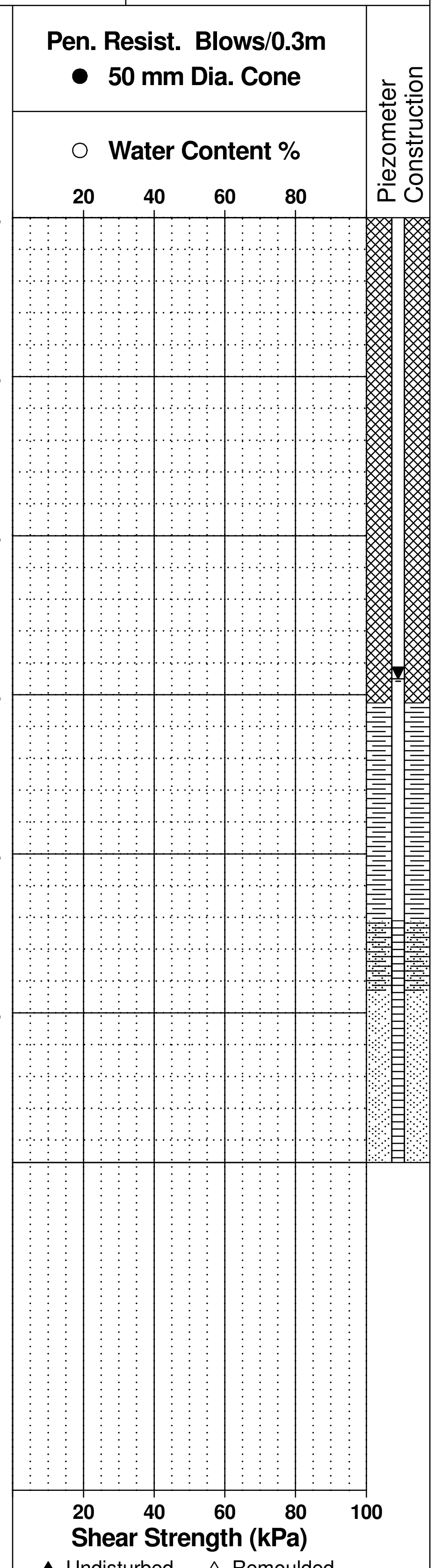
BORINGS BY CME 55 Power Auger

DATE 2020 January 28

FILE NO. PG5235

HOLE NO. BH 1

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.38
TOPSOIL	0.08	AU	1				
Compact **SANDY SILT**		SS	2	54	21	1	104.38
- loose by 1.5 m depth		SS	3	63	4	2	103.38
- grey with gravel and clay by 2.3 m	3.05	SS	4	79	11	3	102.38
GLACIAL TILL: Dense grey silty sand to sandy silt, gravel, cobbles and boulders, trace clay		SS	5	50	31		
		SS	6	64	+50	4	101.38
		SS	7	93	+50	5	100.38
	5.94	SS	8	82	+50		
End of Borehole							
(GWL @ 2.9m depth - Feb. 4, 2020)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - Block 2
Ottawa, Ontario

DATUM Geodetic

REMARKS

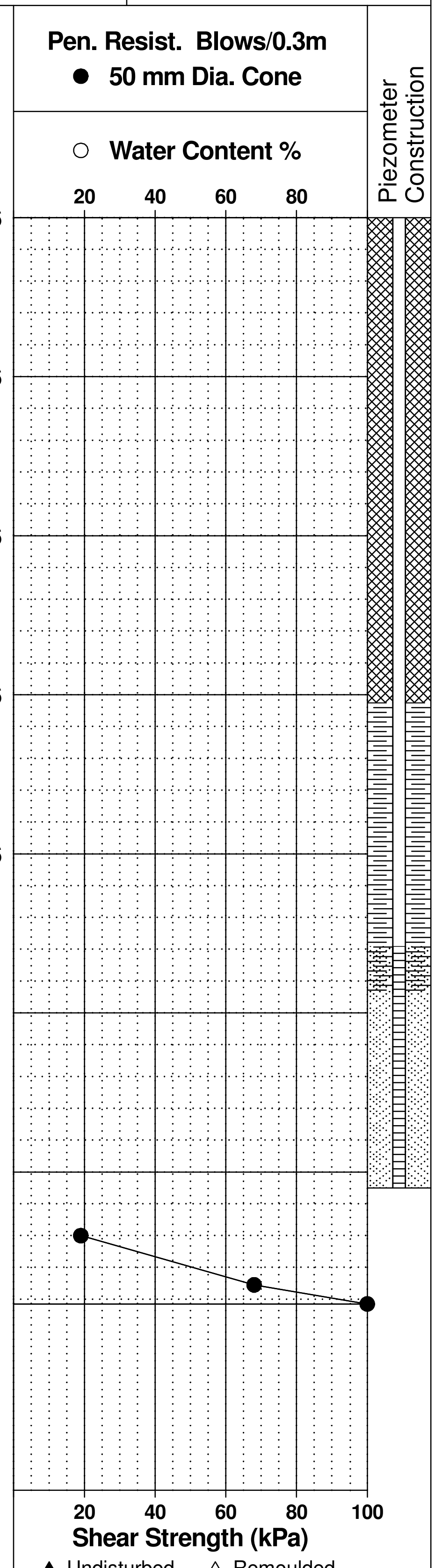
BORINGS BY CME 55 Power Auger

DATE 2020 January 28

FILE NO. PG5235

HOLE NO. BH 2

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.86
TOPSOIL	0.08	AU	1				
Compact, brown **SANDY SILT**		SS	2	88	19	1	103.86
		SS	3	67	14	2	102.86
- grey by 2.1 m depth		SS	4	54	15	3	101.86
	3.35	SS	5	83	17		
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders, some clay		SS	6	4	44	4	100.86
		SS	7	100	63	5	99.86
		SS	8	100	+50		
	6.10	SS	9	0	+50	6	98.86
Dynamic Cone Penetration Test commenced at 6.40m depth.							
	6.83						
End of Borehole							
Practical refusal to DCPT @ 6.83m depth							
(Piezometer blocked - Feb. 4, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - Block 2
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 January 28

FILE NO. PG5235

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.11
TOPSOIL	0.05						
Brown **SILTY SAND**	2.13	AU	1				
		SS	2	54	17	1	104.11
		SS	3	54	13	2	103.11
Compact, brown **SANDY SILT** - grey by 2.7 m depth	3.66	SS	4	33	9		
		SS	5	8	15	3	102.11
GLACIAL TILL: Loose to compact, grey silty sand with gravel, cobbles and boulders, some clay	5.94	SS	6	92	4	4	101.11
		SS	7	17	4	5	100.11
		SS	8	100	26		
End of Borehole (Piezometer blocked - Feb. 4, 2020)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

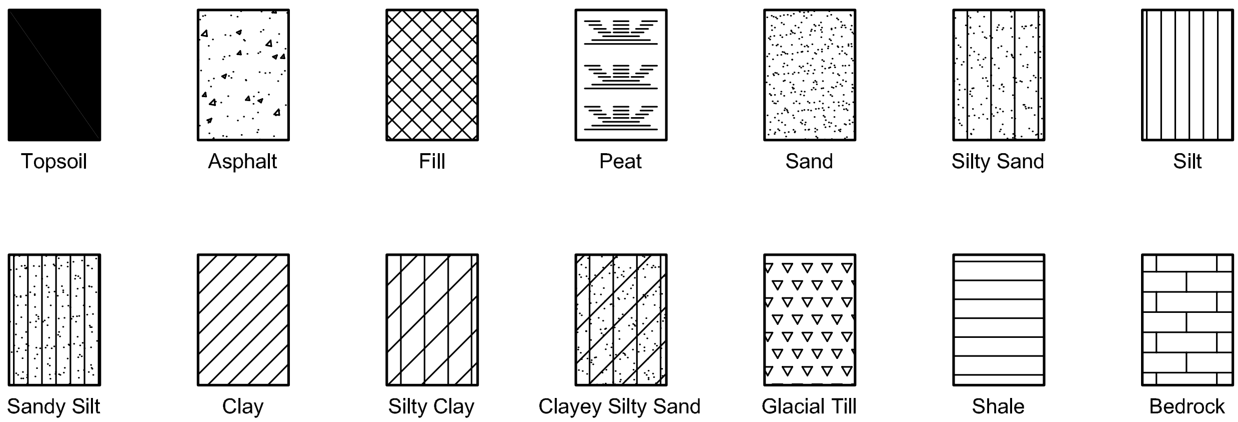
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

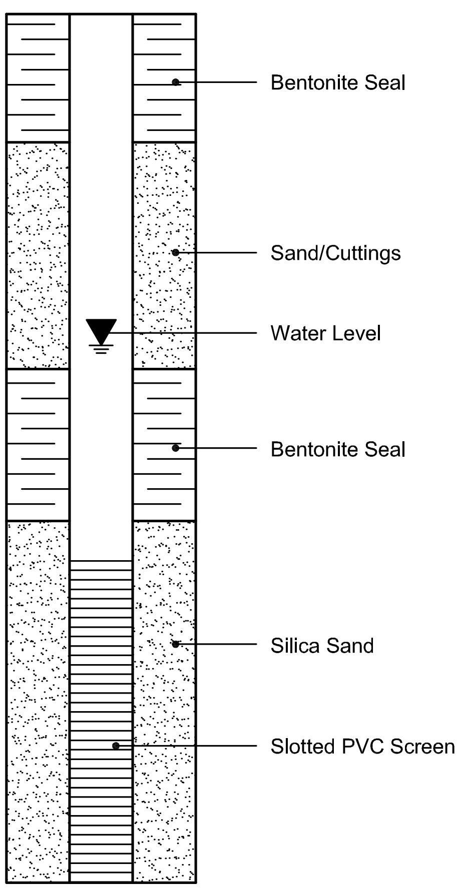
SYMBOLS AND TERMS (continued)

STRATA PLOT

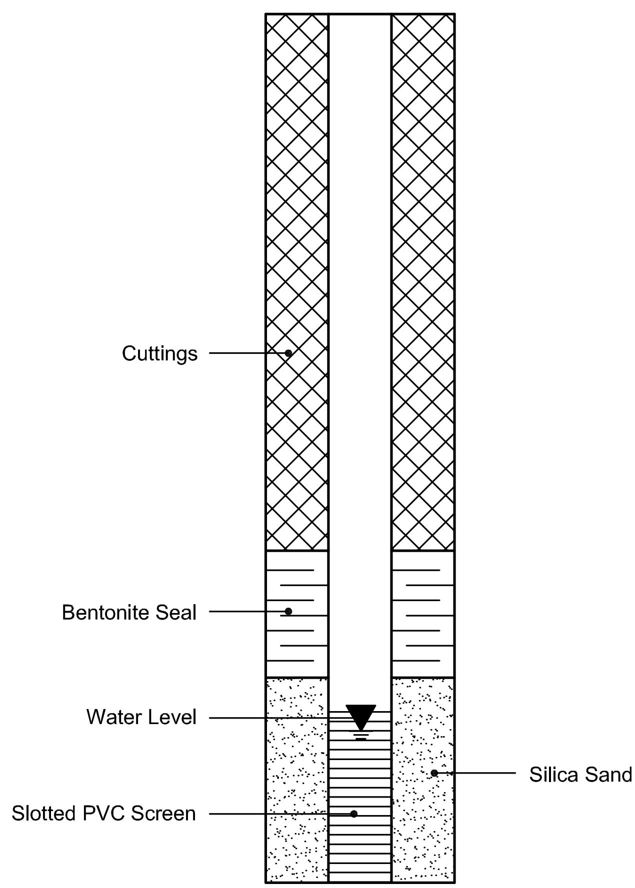


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25640

Report Date: 10-Feb-2020

Order Date: 4-Feb-2020

Project Description: PG5235

	Client ID:	BH3-SS3	-	-	-
	Sample Date:	28-Jan-20 13:00	-	-	-
	Sample ID:	2006206-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	83.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.88	-	-	-
Resistivity	0.10 Ohm.m	77.5	-	-	-
Anions					
Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	31	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SHEAR WAVE VELOCITY TESTING PROFILES

DRAWING PG5235-1 - TEST HOLE LOCATION PLAN

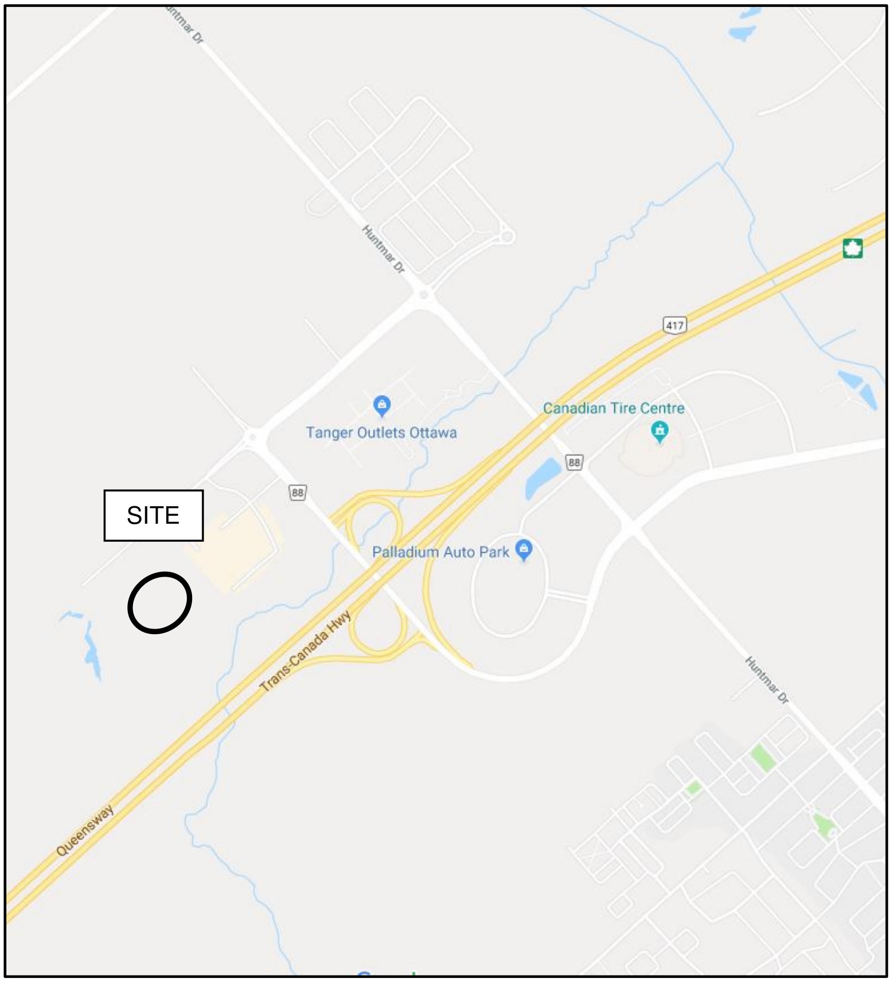


FIGURE 1

KEY PLAN

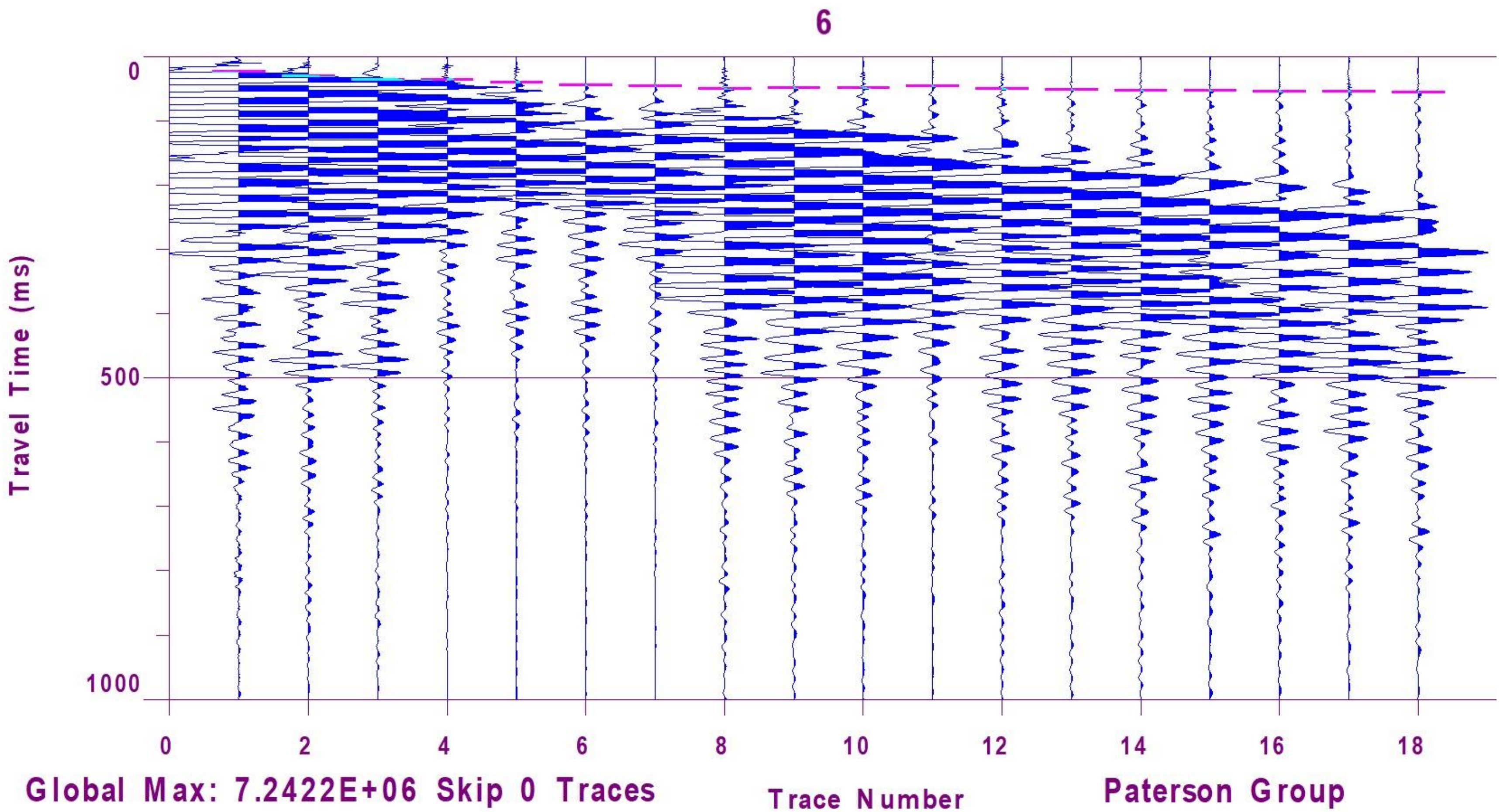


Figure 2 – Shear Wave Velocity Profile at Shot Location -4.5 m

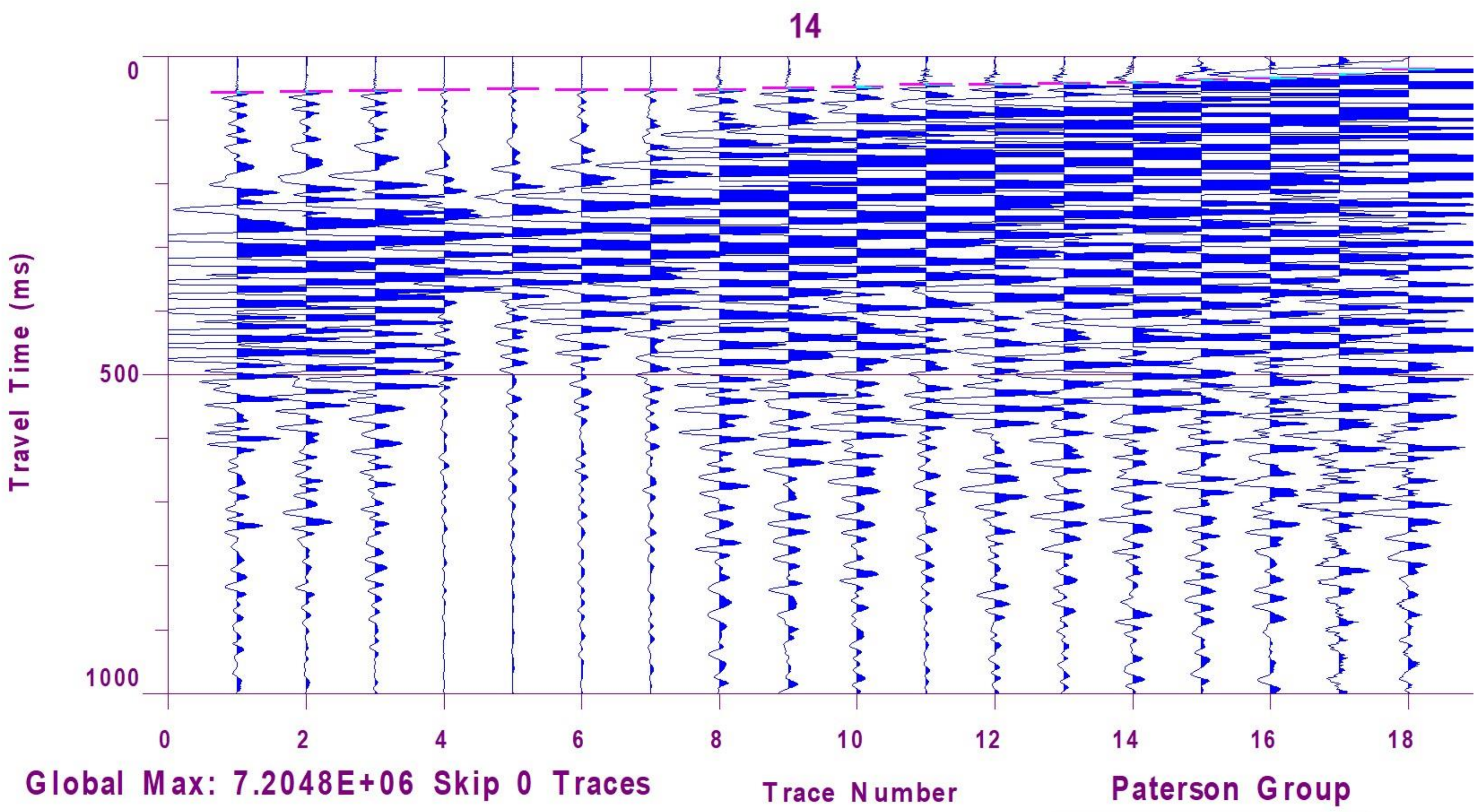
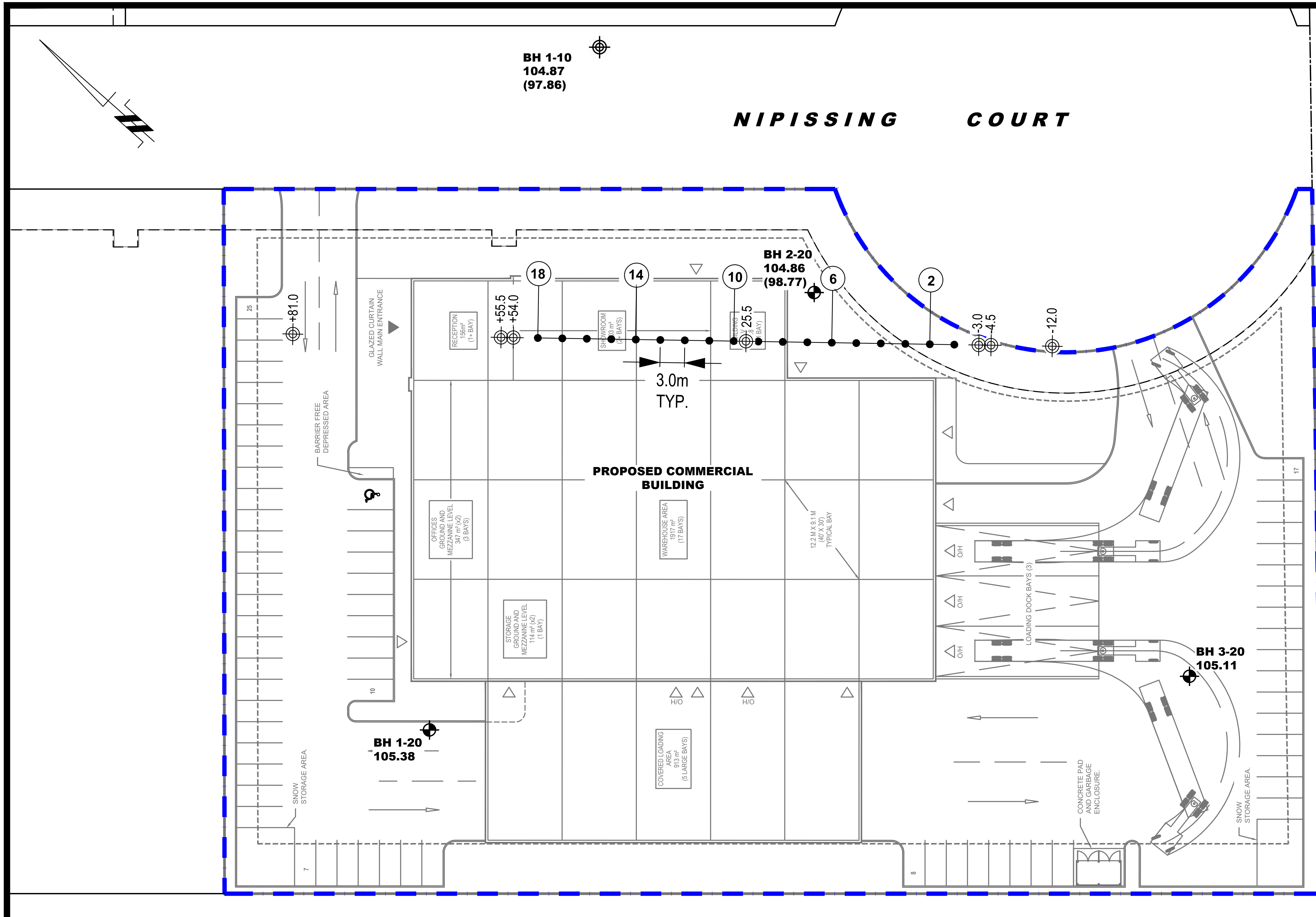


Figure 3 – Shear Wave Velocity Profile at Shot Location +54 m



LEGEND:

	BOREHOLE LOCATION (CURRENT INVESTIGATION)
	BORHOLE LOCATION (PATERSON GROUP REPORT No. PG0912)
	GEOPHONE LOCATIONS
10	GEOPHONE NUMBER
-3.0	SHOT LOCATION
104.86	GROUND SURFACE ELEVATION (m)
(98.77)	PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC ELEVATION (NAD83 ZONE 18T)

SCALE: 1:500

0 5 10 15 20 25 30m

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Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

1497328 ONTARIO INC. c/o TAGGART REALTY MANAGEMENT

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING - BLOCK 2 - NIPISSING COURT

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 02/2020
Drawn by: NFRV	Report No.: PG5235-1
Checked by: DP	Dwg. No.: PG5235-1
Approved by: FA	Revision No.:

p:\autocad drawings\geotechnical\pg52xx\pg5235\pg5235-1 test hole location plan.dwg