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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Commercial Buildings
Buildings No. 14 and 15
Tanger Outlet Mall
Campeau Drive - Ottawa

Prepared For

RioCan REIT

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February 23, 2018

Report: PG4420-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Riocan REIT (Riocan) to conduct a geotechnical investigation for the proposed commercial buildings to be located along Campeau Drive at Palladium Drive, 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 by means of boreholes,

❑ to provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Project

It is understood that the proposed buildings will consist of several slab-on-grade buildings located within the southwestern portion of the existing Tanger Outlet Mall. It is further understood that associated loading docks, access lanes, parking and landscaped areas will be part of the project.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted on February 9, 2018. At that time a total of five (5) boreholes were completed by Paterson to provide general coverage of the subject site taking into account site features and underground utilities. The locations of the test holes are shown on Drawing PG4420-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted auger drill rig operated by a two person crew and test pits completed during our previous investigations were excavated using a hydraulic shovel. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights or a 50 mm diameter split-spoon sampler. The soil from the auger flights and split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the auger flight and split-spoon were recovered from the boreholes are depicted as AU and 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.

Undrained shear strength testing, using a vane apparatus, was conducted at regular intervals of depth in cohesive soils.

The thickness of the overburden was evaluated during our previous investigation by dynamic cone penetration testing (DCPT) at BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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.

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.

Groundwater

Flexible PVC standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Open hole groundwater infiltration levels were noted within the test pit locations completed during the previous investigations.

Sample Storage

All samples recovered during 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 locations of the test holes were selected by Paterson and located and surveyed in the field by Stantec Geomatics. The ground surface elevations at the test hole locations are understood to be referenced to a geodetic datum.

The locations and ground surface elevations of the test holes are presented on Drawing PG4420-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

At the time of our current investigation, the ground surface across the subject site was snow covered. It is understood that the proposed building footprint areas were currently being used as a snow storage area for snow clearing of the existing parking lot.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of a topsoil layer overlying a hard to very stiff silty clay deposit. Silt was noted below the silty clay layer. Practical refusal to DCPT was encountered at 8.9 m in BH 3. 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.

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam formation. Also, the bedrock surface is expected at depths ranging from 5 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the standpipes on February 23, 2018. The results of the groundwater monitoring are presented in Table 1. It should be noted that surface water can become trapped within the backfilled borehole that could indicate higher than normal readings. The long term groundwater level can also be estimated based on the recovered soil sample's moisture level and consistency. Based on these observations, the long term groundwater table is anticipated to be at a 3 to 4 m below existing ground surface. It should be further noted that the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Water Level		Date
		Depth (m)	Elevation (m)	
BH 1	102.73	2.23	100.50	February 23, 2018
BH 2	102.73	2.10	100.63	February 23, 2018
BH 3	102.50	2.18	100.32	February 23, 2018
BH 4	102.79	2.09	100.70	February 23, 2018
BH 5	102.44	2.11	100.33	February 23, 2018

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed commercial buildings. It is expected that the proposed commercial buildings will be founded by conventional shallow footings placed on an undisturbed, stiff silty clay bearing surface.

Due to the presence of the silty clay layer, the proposed development will be subjected to permissible grade raise restrictions. If the grade raise restriction is exceeded, several options are available, such as a preload/surcharge program or the placement of lightweight fill below the proposed buildings.

The above and other considerations are further 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 areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II or an approved alternative. 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 and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, 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, the material should be compacted in thin lifts to a minimum density of 95% of the respective 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

Based on the results of the geotechnical investigation, lightly loaded structures, such as the buildings anticipated, could be founded on shallow footings bearing on an engineered fill or stiff silty clay crust.

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

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.

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 stiff silty clay and/or engineered fill 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.

Permissible Grade Raise Recommendations

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill.

Based on the silty clay layer depth and stiffness of the deposit, the following permissible grade raises are recommended for the proposed development:

☐ A **permissible grade raise restriction of 1.5 m** is recommended for the **proposed buildings** across the subject section of the site.

☐ A **permissible grade raise restriction of 2 m** is recommended for **parking areas and access roadways**.

A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations. To reduce potential long term liabilities, consideration should be given to accounting for a larger groundwater lowering and to providing means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the buildings, etc). It should be noted that building on silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking as compared to unreinforced foundations.

5.4 Design for Earthquakes

As part of our previous geotechnical investigation for the overall development, seismic shear wave velocity testing was completed to accurately determine the applicable seismic site classification for foundation design for buildings constructed within the subject site from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. Two (2) of the shear wave profiles from our on-site testing are presented in Appendix 2.

Field Program

The shear wave testing was completed within the southwest portion of the subject site on October 30, 2012. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3, 4.5 and 20 m away from the first and last geophone.

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).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m immediately below the proposed buildings' foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

Based on our analysis of the shear wave velocity profiles, the average shear wave velocity through the overburden soil is **141 m/s**. The average shear wave velocity for the bedrock is **2,853 m/s**.

Based on our findings at borehole locations, inferred bedrock was encountered at BH 3 at a 9 m depth, which is considered to be a worst case scenario for seismic site classification. The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012.

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

$$V_{s30} = \frac{30m}{\left(\frac{9m}{141m/s} + \frac{21m}{2{,}853m/s}\right)}$$

$$V_{s30} = 421m/s$$

Based on the results of the seismic testing, the average shear wave velocity of the upper 30 m profile below the proposed building's underside of foundation, Vs_{30}, was calculated to be **421 m/s** for the subject site. Therefore, a **Site Class C is applicable for design of the foundation for the proposed buildings** as per Table 4.1.8.4.A of the OBC 2012.

5.5 Slab on Grade Construction

With the removal of topsoil and fill, containing deleterious or organic materials, the existing stiff, silty claymay be considered an acceptable subgrade surface on which to commence backfilling for slab on grade 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. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

5.6 Pavement Structure

For design purposes, the pavement structures presented in the following tables could be used for the design of car only parking areas, heavy truck 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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil	

Table 3 - Recommended Pavement Structure
Heavy Truck Parking Areas and Access Lanes

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing the load bearing capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. Along local streets, the drains should be placed along the edges of the pavement. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

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 structures. It is understood that the proposed buildings will be of slab-on-grade construction and it should be noted that the perimeter foundation drainage system provides an outlet for perched water below the proposed sidewalks anticipated to be surrounding the buildings. Perched water below the sidewalks can lead to heaved sidewalks due to freeze/thaw cycles. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The 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. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 composite drainage blanket, such as Miradrain G100N or Delta Drain 6000.

Concrete Sidewalks Adjacent to Building

To avoid differential settlements within the proposed sidewalks 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 free draining, non frost susceptible material such as, Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to 95% 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 could 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.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either 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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A crushed stone. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. 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 the SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

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.

Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps. The following general information regarding permits for pumping water should be noted:

If the anticipated pumping volumes exceed 400,000 L/day of ground and/or surface water, a temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) would be required for this project 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.

If typical ground or surface water volumes being pumped during the construction phase are anticipated 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.

An EASR building permit may be required for the proposed building excavation program depending on the period of work. It is not expected that a PTTW will be required. The requirement for an EASR permit should ascertained at the time of excavation.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

❑ Review grading plan from a geotechnical perspective, once available.

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

❑ Sampling and testing of the concrete and granular 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. We request permission to review our recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification 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 Riocan REIT or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

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David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Tanger Outlet Bldgs. 14 & 15
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

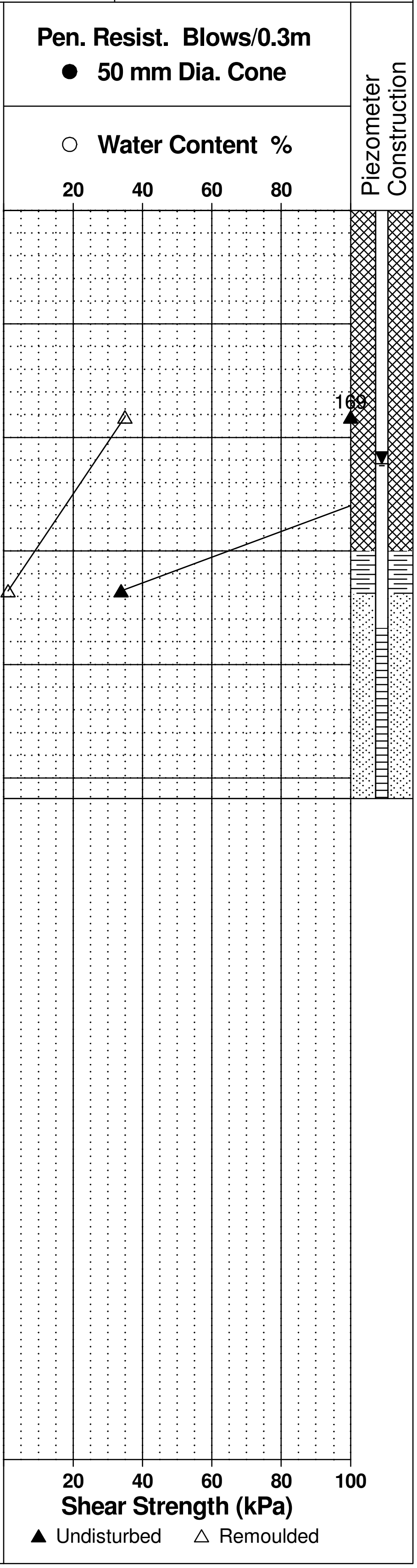
BORINGS BY CME 55 Power Auger

DATE February 9, 2018

FILE NO. PG4420

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.73
TOPSOIL 0.28		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	67	5	1	101.73
		SS	3	54	P	2	100.73
- firm and grey by 3.0m depth						3	99.73
3.96						4	98.73
Grey **SILT,** trace clay		SS	4	25	P	5	97.73
5.18							
End of Borehole (GWL @ 3.0m depth based on field observations)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Tanger Outlet Bldgs. 14 & 15
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

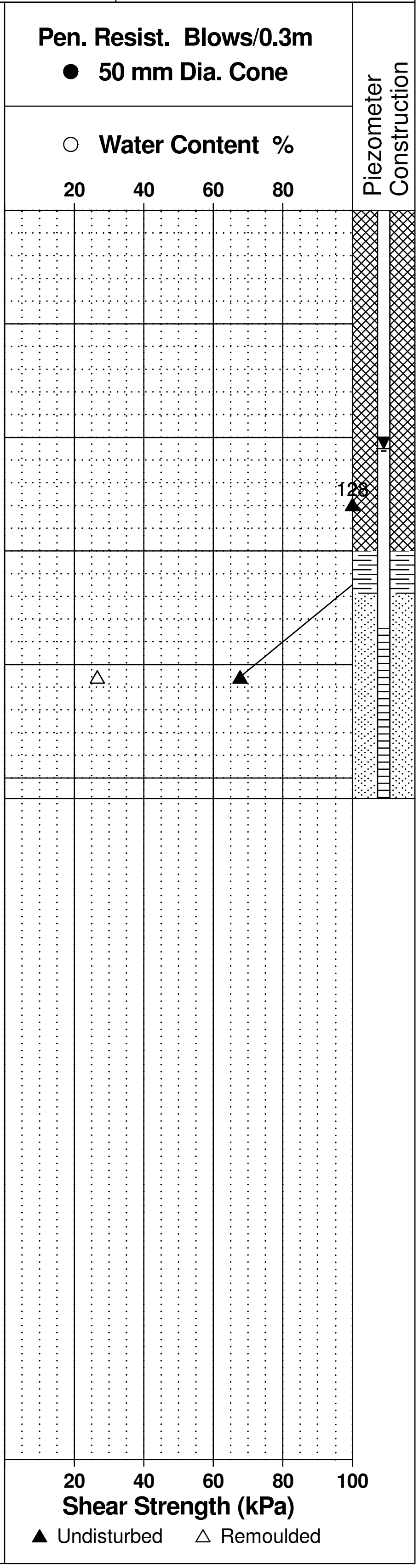
DATE February 9, 2018

FILE NO. PG4420

HOLE NO. BH 2

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.33	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	50	7
	4.34	SS	3	83	P
Grey **SILT,** trace clay	5.18	SS	4	33	P
End of Borehole (GWL @2.10m - Feb. 23, 2018)					

DEPTH (m)	ELEV. (m)
0	102.73
1	101.73
2	100.73
3	99.73
4	98.73
5	97.73



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Tanger Outlet Bldgs. 14 & 15
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

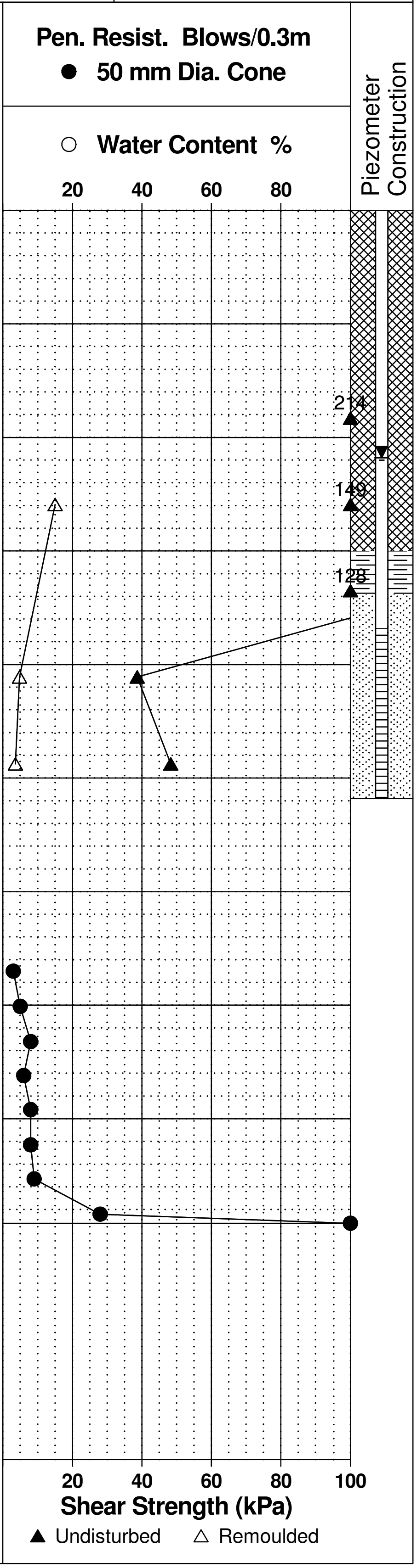
BORINGS BY CME 55 Power Auger

DATE February 9, 2018

FILE NO. PG4420

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
TOPSOIL 0.28		AU	1			0	102.50
		SS	2	50	7	1	101.50
Hard to very stiff, brown **SILTY CLAY,** trace sand						2	100.50
						3	99.50
		SS	3	100	P		
- firm and grey by 3.8m depth						4	98.50
5.18		SS	4	100		5	97.50
Dynamic Cone Penetration Test commenced at 5.18m depth. Cone pushed to 6.4m depth.						6	96.50
						7	95.50
						8	94.50
8.92 End of Borehole Practical DCPT refusal at 8.92m depth (GWL @2.18m - Feb. 23, 2018)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Tanger Outlet Bldgs. 14 & 15
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

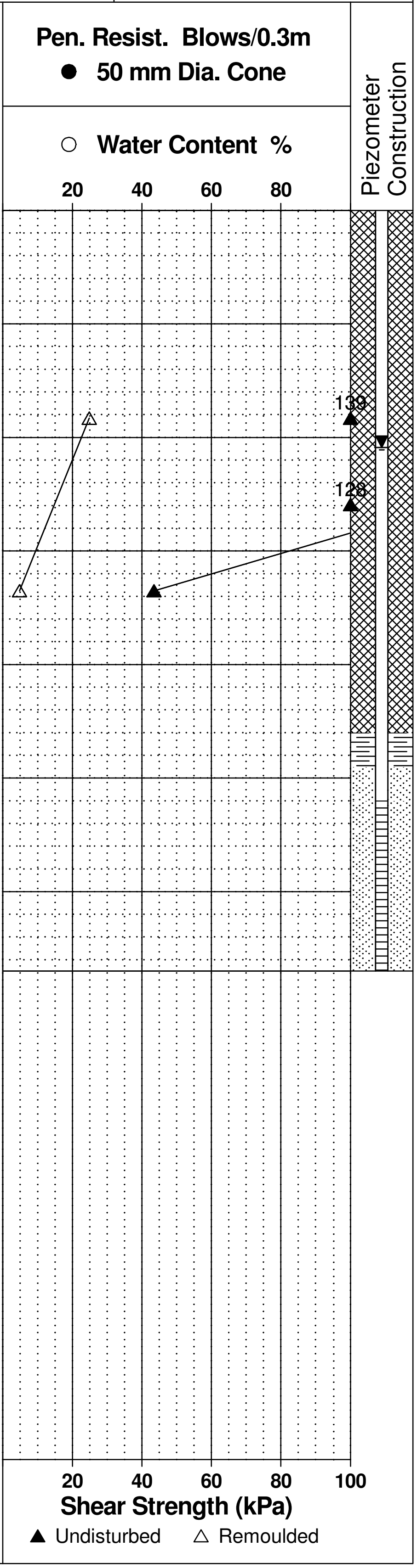
BORINGS BY CME 55 Power Auger

DATE February 9, 2018

FILE NO. PG4420

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.79
TOPSOIL 0.33	AU	1				
Very stiff, brown **SILTY CLAY,** trace sand	SS	2	58	3	1	101.79
					2	100.79
	SS	3	92	P		
- firm and grey by 3.0m depth 3.58					3	99.79
					4	98.79
Compact, grey **SILT,** trace clay					5	97.79
					6	96.79
6.70	SS	4	50	P		
End of Borehole (GWL @2.09m - Feb. 23, 2018)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Tanger Outlet Bldgs. 14 & 15
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

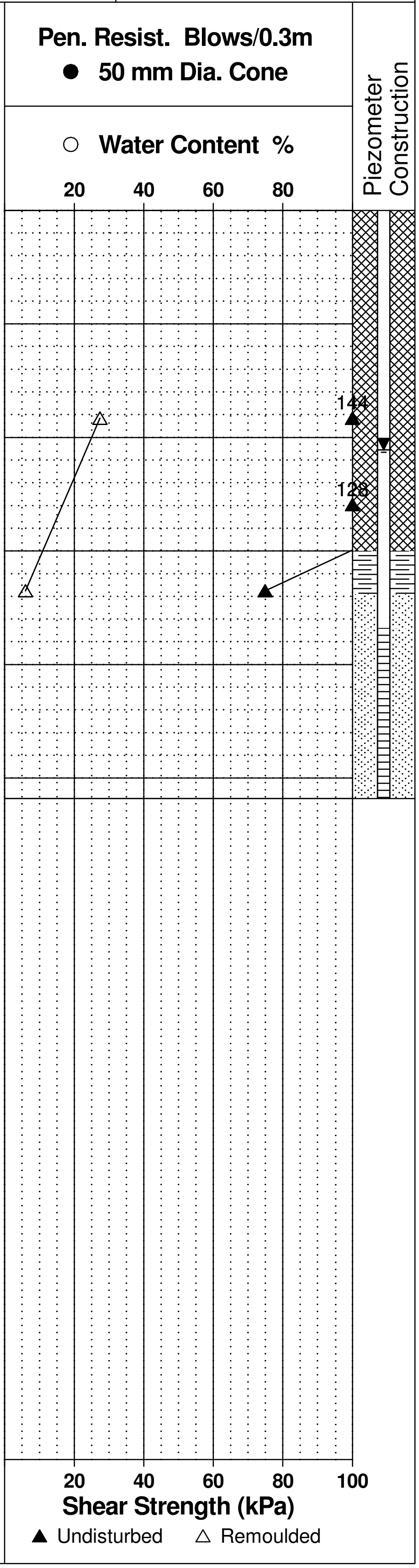
BORINGS BY CME 55 Power Auger

DATE February 9, 2018

FILE NO. PG4420

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.44
TOPSOIL 0.28		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	67	7	1	101.44
						2	100.44
						3	99.44
- grey by 3.2m depth 3.58							
						4	98.44
Compact, grey **SILT,** trace clay							
5.18		SS	3	33	P	5	97.44
End of Borehole							
(GWL @2.11m - Feb. 23, 2018)							



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)

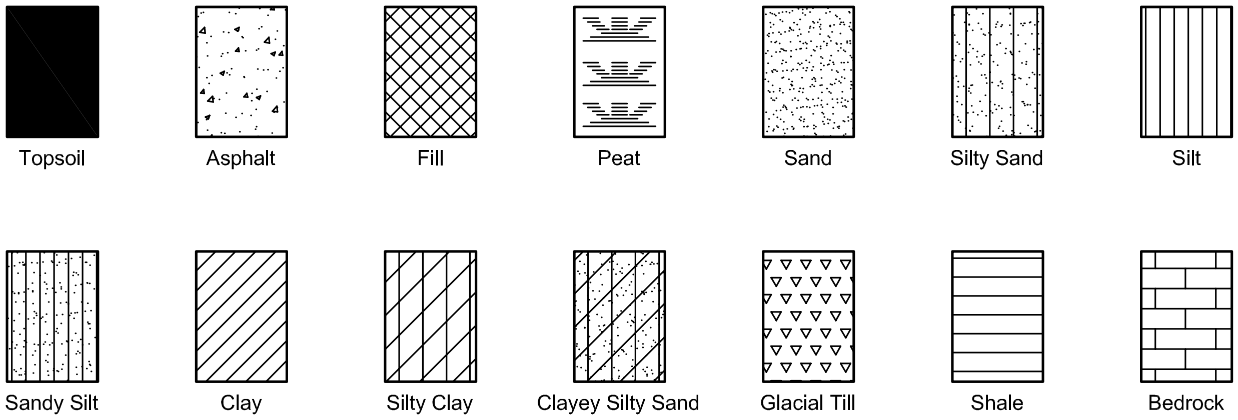
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

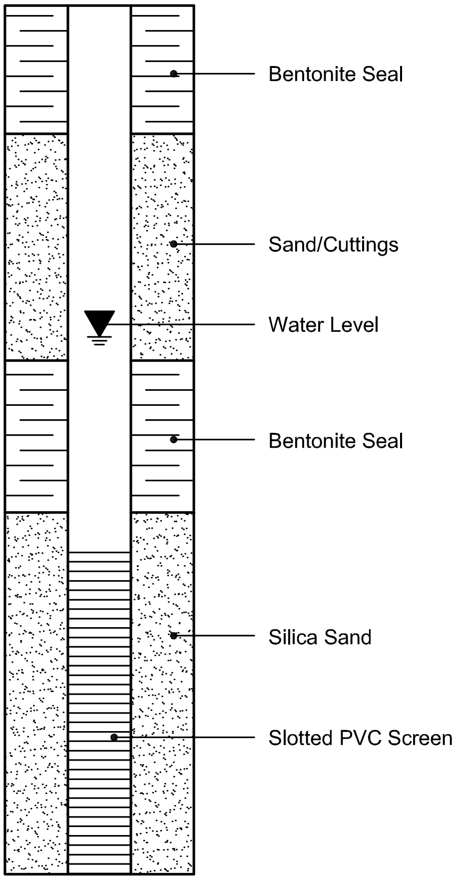
SYMBOLS AND TERMS (continued)

STRATA PLOT

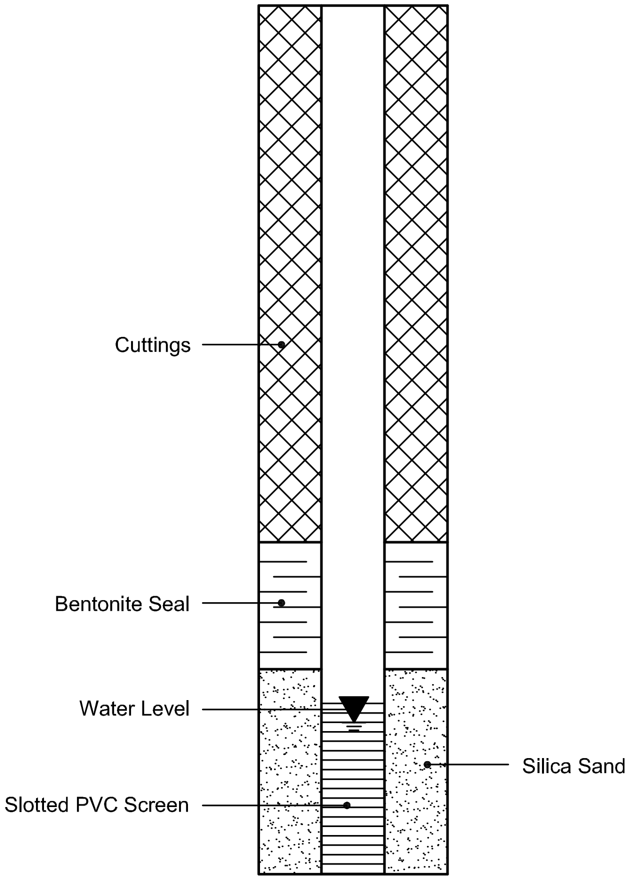


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 23202

Report Date: 16-Feb-2018
Order Date: 12-Feb-2018
Project Description: PG4420

	Client ID:	BH1-SS3	-	-	-
	Sample Date:	09-Feb-18	-	-	-
	Sample ID:	1807080-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.64	-	-	-
Resistivity	0.10 Ohm.m	17.0	-	-	-
Anions					
Chloride	5 ug/g dry	201	-	-	-
Sulphate	5 ug/g dry	47	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4420-1 - TEST HOLE LOCATION PLAN

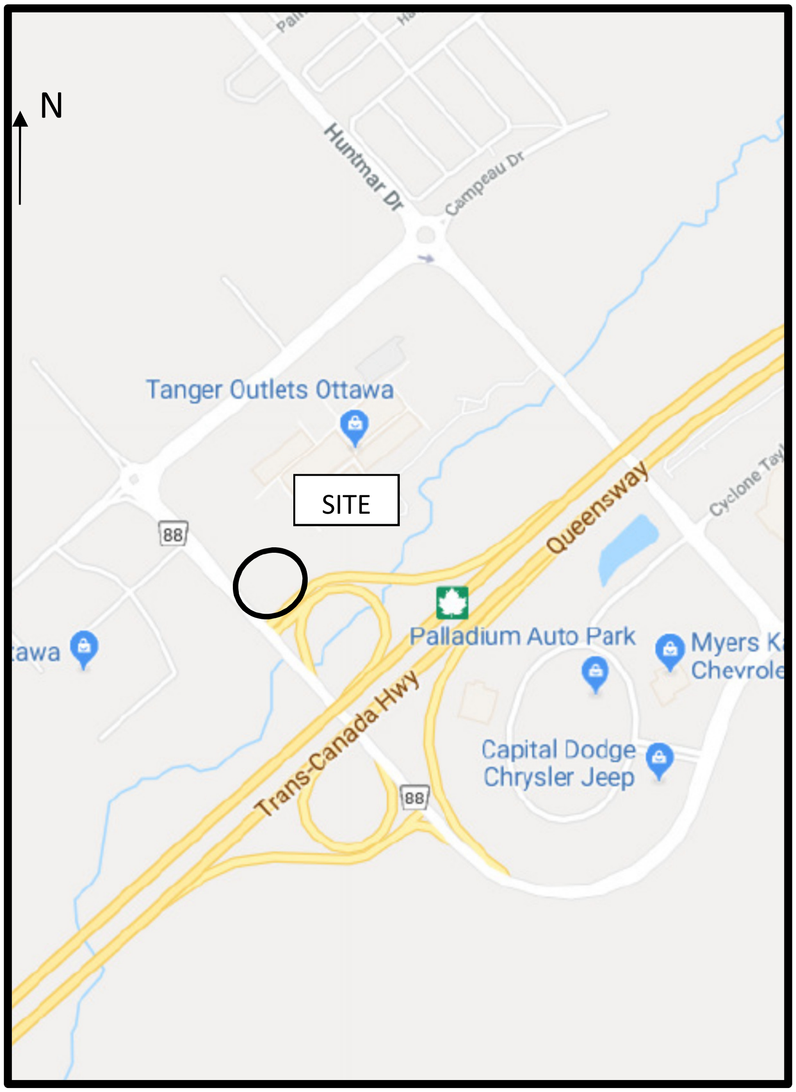


FIGURE 1

KEY PLAN

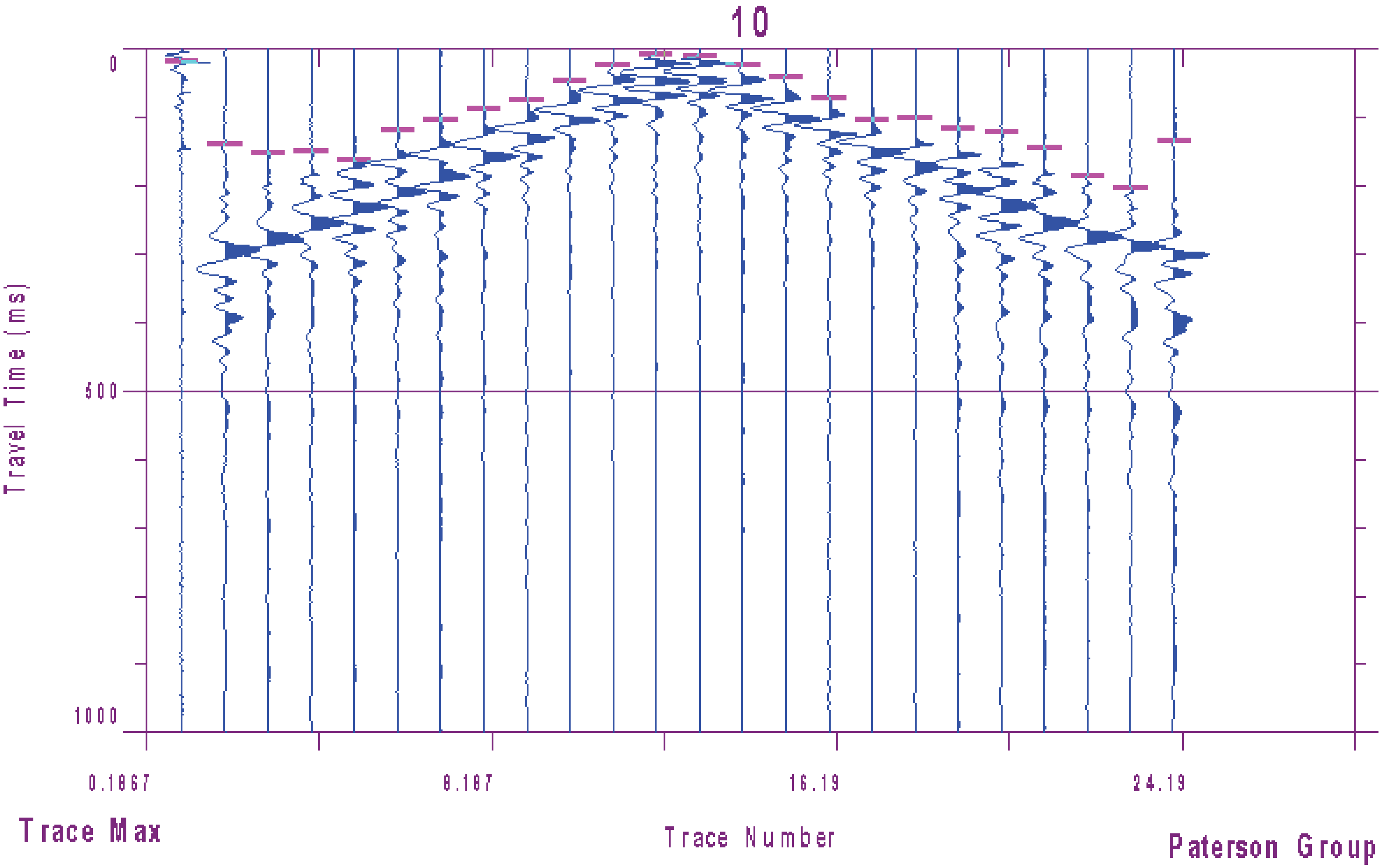


Figure 2 – Shear Wave Velocity Profile at Shot Location 34.5 m

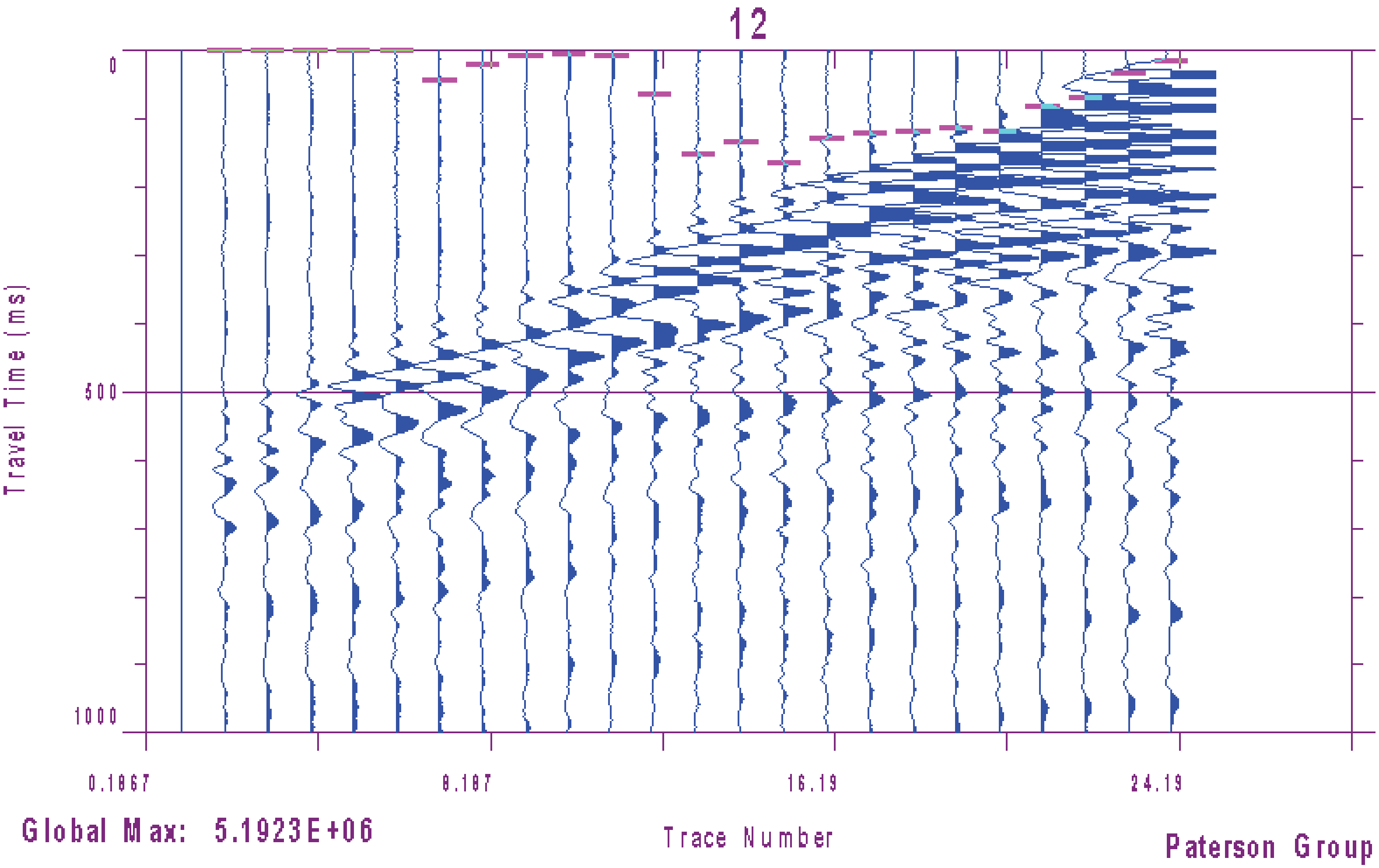
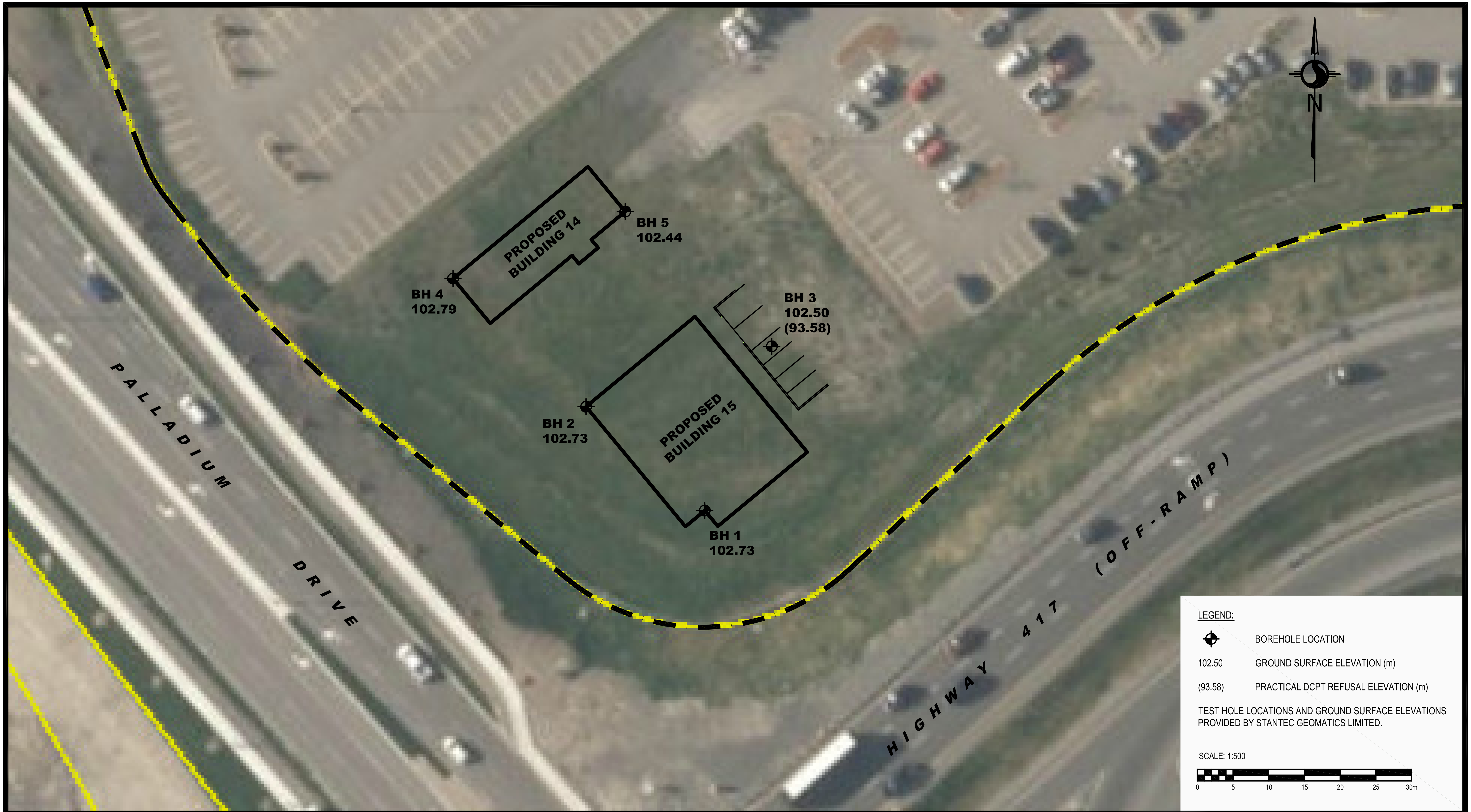


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m



LEGEND:

⌖	BOREHOLE LOCATION
102.50	GROUND SURFACE ELEVATION (m)
(93.58)	PRACTICAL DCPT REFUSAL ELEVATION (m)

TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY STANTEC GEOMATICS LIMITED.

SCALE: 1:500

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NO.	REVISIONS	DATE	INITIAL
0			

RIOCAN REIT
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL BUILDINGS - BUILDINGS 14 & 15 - TANGER OUTLET
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 02/2018
Drawn by: MPG	Report No.: PG4420-1
Checked by: SB	Dwg. No.: PG4420-1
Approved by: DJG	Revision No.: 0

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