Geotechnical Engineering

Environmental Engineering

Hydrogeology

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Geotechnical Investigation

Proposed Warehouse Development
210 and 220 Maple Creek Court
Ottawa, Ontario

Prepared For

Wall Sound (2434894 Ontario Inc.)
c/o BBS Construction

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December 16, 2016

Report: PG3905-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by BBS Construction on behalf of Wall Sound (2434894 Ontario Inc.) to conduct a geotechnical investigation for the proposed warehouse development to be located at 210 and 220 Maple Creek 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 boreholes.
- ☐ 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.

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

2.0 Proposed Development

Based on the current conceptual site plan, it is our understanding that the proposed development consists of a series of warehouse structures along with associated access lanes, car parking and landscaped areas.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on November 14, 2016. At that time, a total of six (6) boreholes (BH 1 to BH 6) were distributed in a manner to provide general coverage of the proposed development taking consideration of existing site features and underground utilities. The locations of the test holes are shown on Drawing PG3905-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a low clearance track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedures consisted of augering or excavating 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. Grab samples were collected from the test pits at selected intervals. 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.

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

51 mm diameter PVC groundwater monitoring wells was installed in BH 6 and flexible PVC standpipe was install in the remainder of the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

☐ 3 m long slotted 51 mm diameter PVC screen.
☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
☐ No.3 silica sand backfill within annular space around screen.
☐ Bentonite hole plug directly above PVC slotted screen to approximately 300 mm below the ground surface.
☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Sample Storage

All samples 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 were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by McIntosh Perry. It is understood that the elevations are referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG3905-1 - 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 field logs. The results are presented on the Soil Profile and Test Data sheets in Appendix 1.

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 we are otherwise directed.

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

The subject site consists of two lot parcels located to the southeast of the cul-de-sac at the end of Maple Creek Court. The north parcel of the subject site (220 Maple Creek Court) has recently been cleared of trees to produce several piles of wood and mulch throughout the north portion of the site. The south portion of the site identified with a civic address of 210 Maple Creek Court was previously cleared and is current grass covered at the time of our field investigation. . A row of boulders were noted along the front property boundary and along the central portion of the site dividing the north and south parcel.

The subject site is bordered to the north by several cover-all structures with associated sediment control ponds, to the east by undeveloped land, to the south by undeveloped commercial occupied by several shipping containers and to the west by a commercial warehouse structure.

The site is relatively flat and approximately at grade with the adjacent roadway and neighboring properties.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consist of a thin layer of topsoil and/or mulch overlying a compact to dense glacial till. The upper portion of the glacial till was weathered into a compact redish-brown silty sand with gravel, trace cobbles overlying a brown compact to dense silty sand with gravel, cobbles and boulders. Practical DCPT refusal was encountered at BH 3 and BH 5 varying between 5 and 5.3 m, respectively, below existing ground surface.

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 subject site is located in an area of interbedded limestone and shale of the Verulam Formation with overburden drift thickness between 5 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured in the boreholes on November 24, 2016. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1. Long-term groundwater level can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected between 3 to 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface Elevation (m)	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 1	113.85	1.11	112.74	November 24, 2016
BH 2	115.54	1.88	113.66	November 24, 2016
BH 3	113.96	0.92	113.04	November 24, 2016
BH 4	114.93	2.64	112.29	November 24, 2016
BH 5	114.27	1.07	113.20	November 24, 2016
* BH 6	114.96	1.25	113.71	November 24, 2016

Notes: * denotes groundwater level reading taken in monitoring well installed at borehole location.

4.4 Preliminary Percolation Rates

It is our understanding that a preliminary percolation rates are required for the undisturbed, compact to dense glacial till as part of the storm water management calculations for the subject site.

The percolation rate was interpreted from the hydraulic conductivity which was estimated based on the available soils information within the subject site as part of the future development. Based on these values, the average percolation rate (T-Time) was estimated to be within the ranges in Table 2 below.

Table 2 - Preliminary Percolation Rates

Material	Hydraulic Conductivity - k (m/sec)	Percolation (T-time) (mins/cm)
Glacial Till	1×10^{-7} to 1×10^{-8}	20 to 50

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed warehouse development. It is expected that the proposed buildings will constructed with conventional shallow foundations.

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 organics, 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 due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. 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 paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Shallow Footings

conventional footings placed on an undisturbed, compact glacial till 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**.

A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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 glacial till 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

The proposed site can be taken as seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious materials, within the footprint of the proposed buildings, the native soil or engineered fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 150 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD.

Any soft 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

Car only parking areas, access lanes and heavy truck parking areas are anticipated at this site. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - 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 4 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - 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 lifts and compacted to a minimum of 98% of the material's 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 its load carrying capacity.

It is recommended that subdrains be installed 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. 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. 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, such as clean sand or OPSS Granular B Type I granular material. 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. A drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system is recommended.

Concrete Sidewalks Adjacent to Building

To avoid differential settlements within the proposed sidewalks adjacent to the proposed buildings, 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.

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.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and 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 at least 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 compacted to a minimum of 95% of the material’s SPMDD.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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

Groundwater Control for Building Construction

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.

Permit to Take Water

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

For typical ground or surface water volumes, being pumped during the construction phase, 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.

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.

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 non-aggressive to slightly 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. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Wall Sound (2434894 Ontario Inc.), BBS Construction 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.

Richard Groniger, C. Tech

LICENSED PROFESSIONAL ENGINEER
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Report Distribution:

- ☐ BBS Construction (3 copies)
- ☐ Paterson Group (1 copy)

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
Prop. Warehouse Dev. - 210 & 220 Maple Creek Court
Ottawa, Ontario

DATUM Ground surface elevations provided by McIntosh Perry.

REMARKS

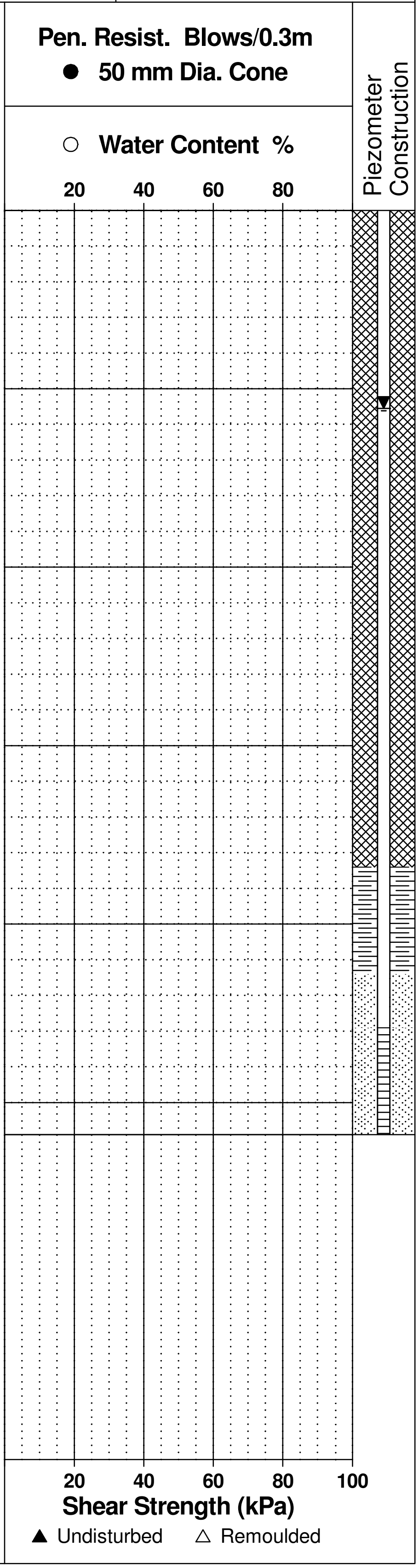
BORINGS BY CME 55 Power Auger

DATE November 14, 2016

FILE NO. PG3905

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	113.85
TOPSOIL and ORGANICS 0.33		AU	1				
		SS	2	83	24		
						1	112.85
		SS	3	58	42		
						2	111.85
GLACIAL TILL: Compact to very dense, brown silty fine sand with gravel, cobbles and boulders, trace clay		SS	4	67	56		
						3	110.85
- grey by 3.2m depth		SS	5	73	50+		
						4	109.85
		SS	6	58	39		
		SS	7	100	84		
5.18						5	108.85
End of Borehole (GWL @ 1.11m-Nov. 24, 2016)							



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DATUM Ground surface elevations provided by McIntosh Perry.

REMARKS

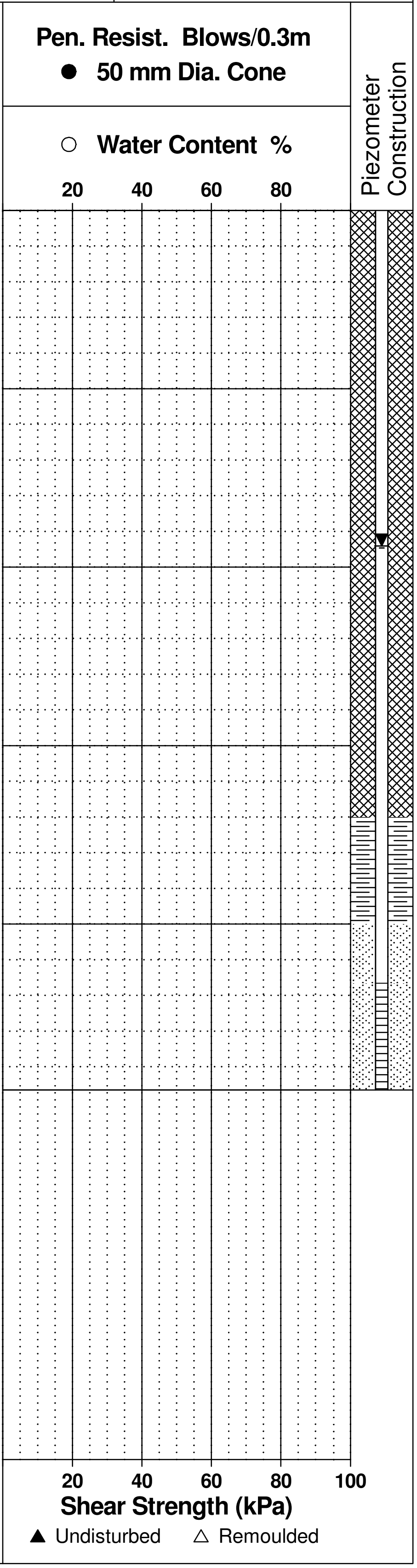
BORINGS BY CME 55 Power Auger

DATE November 14, 2016

FILE NO. PG3905

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	115.54
TOPSOIL and ORGANICS 0.28	AU	1				
GLACIAL TILL: Compact, red-brown silty fine sand, some gravel, cobbles, trace boulders 0.71						
GLACIAL TILL: Compact to very dense, brown silty fine sand with gravel, cobbles and boulders, trace clay	SS	2	75	22	1	114.54
	SS	3	33	50+		
					2	113.54
	SS	4	81	63		
	SS	5	80	50+	3	112.54
- grey by 3.7m depth	SS	6	78	50+	4	111.54
4.93	SS	7	71	50+		
End of Borehole (GWL @ 1.88m-Nov. 24, 2016)						



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SOIL PROFILE AND TEST DATA

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DATE November 14, 2016

FILE NO. PG3905

HOLE NO. BH 3

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL and ORGANICS	0.23
GLACIAL TILL: Red-brown sandy silt with gravel, cobbles, trace clay and boulders	0.69
GLACIAL TILL: Brown silty fine sand with clay, trace gravel, cobbles and boulders	1.68
GLACIAL TILL: Very dense, grey silty fine sand with gravel, cobbles, boulders, trace clay	4.82
Dynamic Cone Penetration Test commenced at 4.82m depth.	4.98

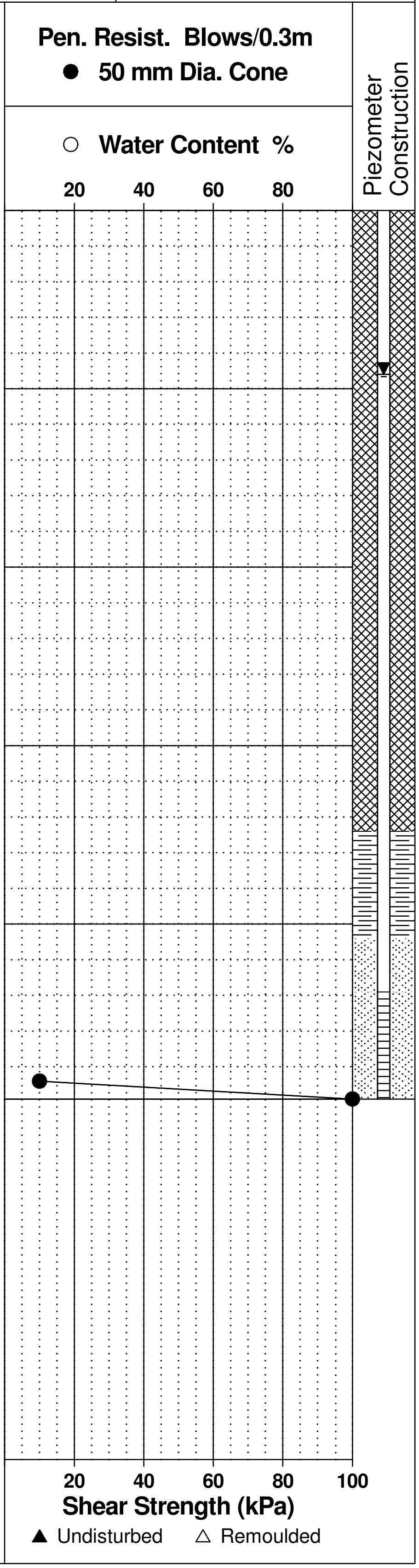
End of Borehole

Practical DCPT refusal at 4.98m depth

(GWL @ 0.92m-Nov. 24, 2016)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	67	9
SS	3	8	41
SS	4	100	62
SS	5	58	49
SS	6	80	50+

DEPTH (m)	ELEV. (m)
0	113.96
1	112.96
2	111.96
3	110.96
4	109.96



SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Ground surface elevations provided by McIntosh Perry.

REMARKS

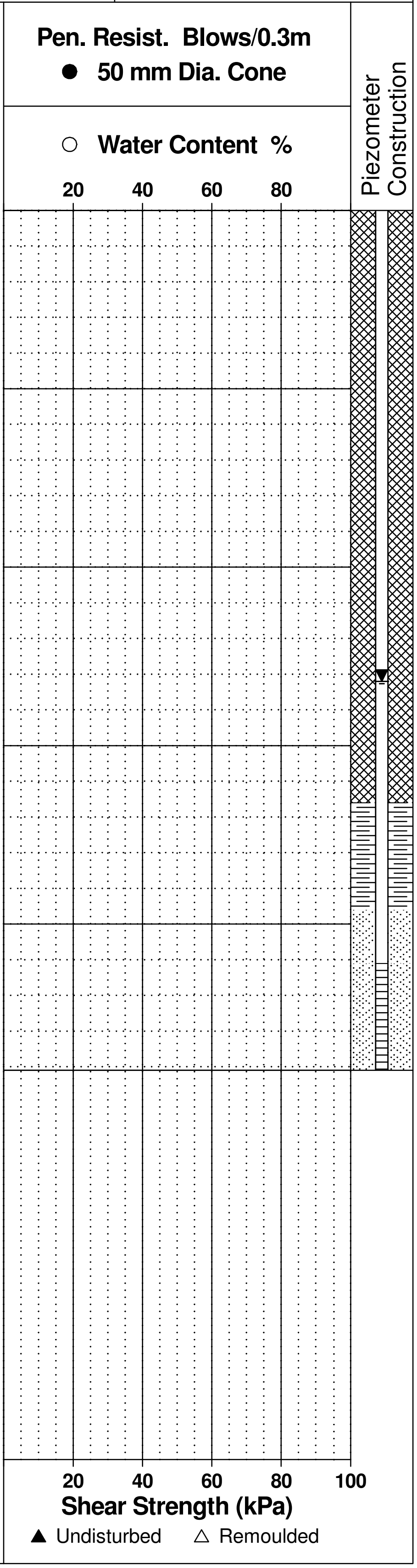
BORINGS BY CME 55 Power Auger

DATE November 14, 2016

FILE NO. PG3905

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
TOPSOIL and ORGANICS					0	114.93
0.30						
GLACIAL TILL: Red-brown silty fine sand, some gravel, cobbles and boulders	AU	1				
0.69						
	SS	2	100	25	1	113.93
	SS	3	95	67		
					2	112.93
GLACIAL TILL: Very dense, brown silty fine sand with gravel, cobbles, boulders, trace clay	SS	4	73	50+		
					3	111.93
- grey by 3.5m depth	SS	5	73	50+		
					4	110.93
4.82	SS	6		50+		
End of Borehole (GWL @ 2.64m-Nov. 24, 2016)						



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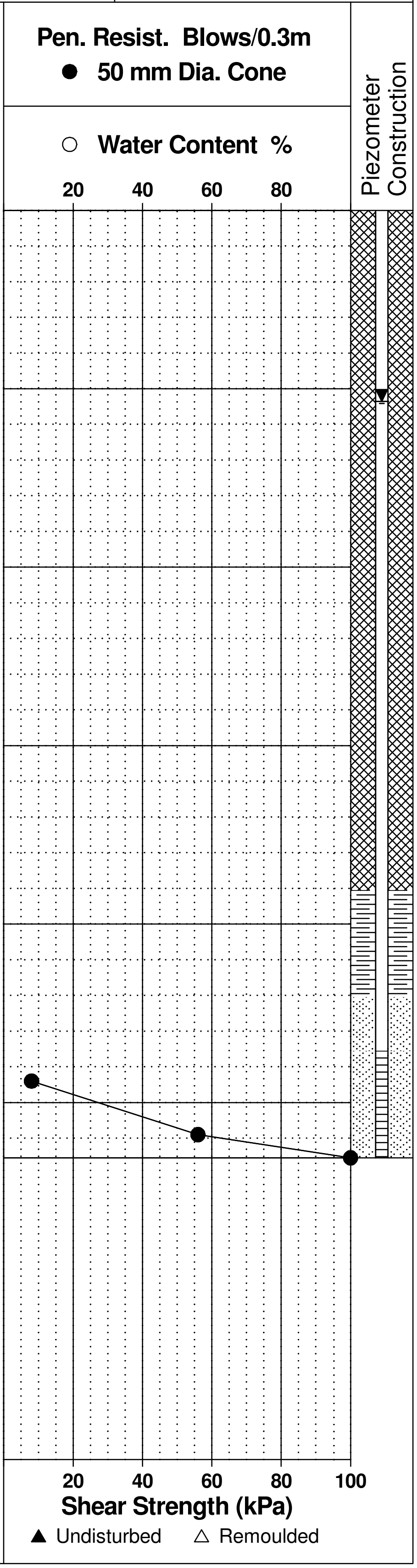
BORINGS BY CME 55 Power Auger

DATE November 14, 2016

FILE NO. PG3905

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							
TOPSOIL and ORGANICS 0.08						0	114.10
GLACIAL TILL: Red-brown silty fine sand, trace gravel, cobbles and boulders 0.69		AU	1				
		SS	2	33	40	1	113.10
		SS	3	75	23	2	112.10
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	4	100	29		
- grey by 3.4m depth						3	111.10
		SS	5	71	48		
		SS	6	75	61	4	110.10
4.82		SS	7	80	50+		
Dynamic Cone Penetration Test commenced at 4.82m depth 5.31						5	109.10
End of Borehole							
Practical DCPT refusal at 5.31m depth							
(GWL @ 1.07m-Nov. 24, 2016)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Dev. - 210 & 220 Maple Creek Court
Ottawa, Ontario

DATUM Ground surface elevations provided by McIntosh Perry.

REMARKS

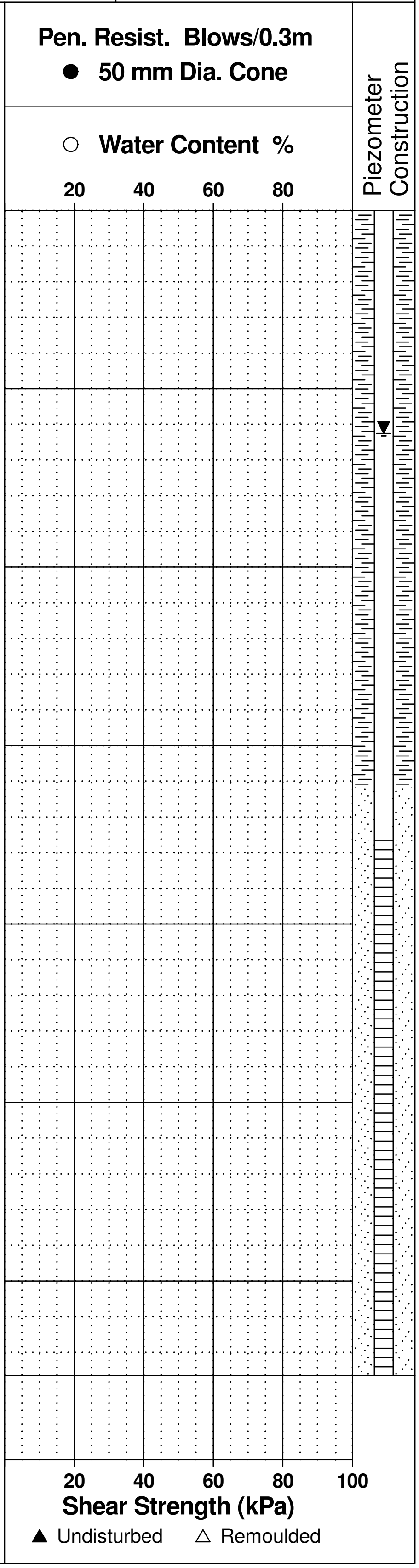
BORINGS BY CME 55 Power Auger

DATE November 14, 2016

FILE NO. PG3905

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	114.96
TOPSOIL and ORGANICS 0.23							
GLACIAL TILL: Red-brown silty sand, some gravel, cobbles, trace boulders 0.60		AU	1				
						1	113.96
		SS	2	83	26		
		SS	3	67	50+		
						2	112.96
GLACIAL TILL: Very dense, brown silty fine sand with gravel, cobbles and boulders, trace clay		SS	4	70	50+		
						3	111.96
- grey by 3.4m depth		SS	5	73	50+		
						4	110.96
		SS	6	70	50+		
						5	109.96
						6	108.96
6.53		SS	7	100	50+		
End of Borehole (GWL @ 1.25m-Nov. 24, 2016)							



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)

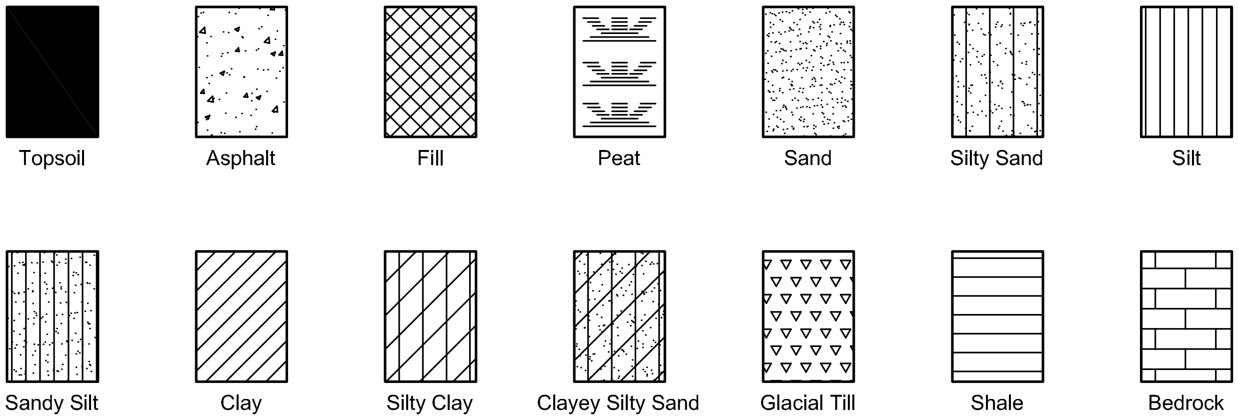
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

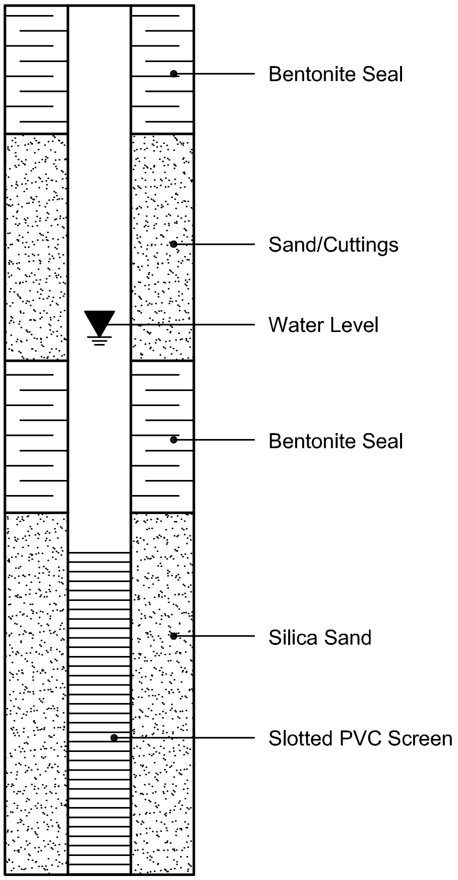
SYMBOLS AND TERMS (continued)

STRATA PLOT

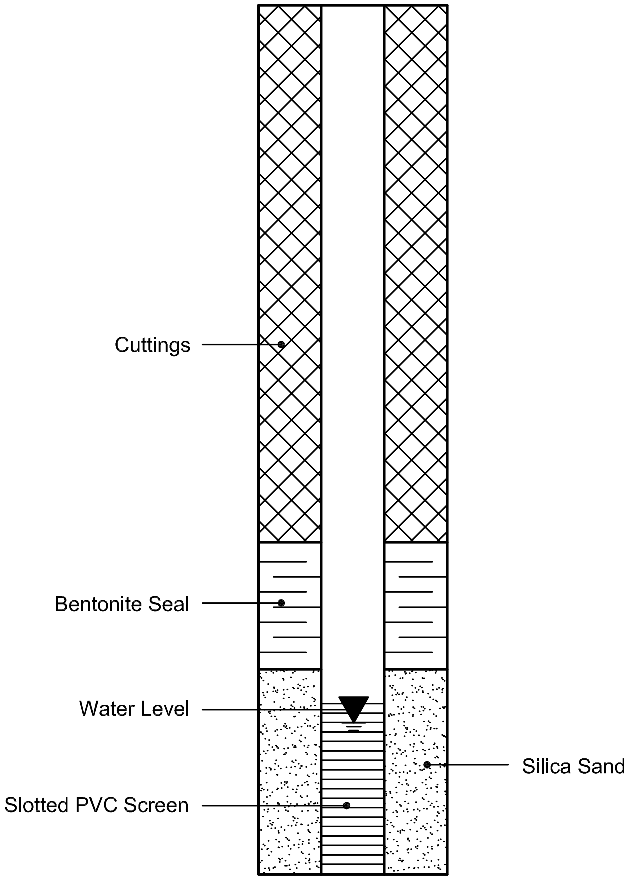


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21269

Report Date: 17-Nov-2016

Order Date: 15-Nov-2016

Project Description: PG3905

	Client ID:	BH1 SS2	-	-	-
	Sample Date:	14-Nov-16	-	-	-
	Sample ID:	1647159-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	86.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.69	-	-	-
Resistivity	0.10 Ohm.m	79.9	-	-	-
Anions					
Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	13	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3905-1 - TEST HOLE LOCATION PLAN

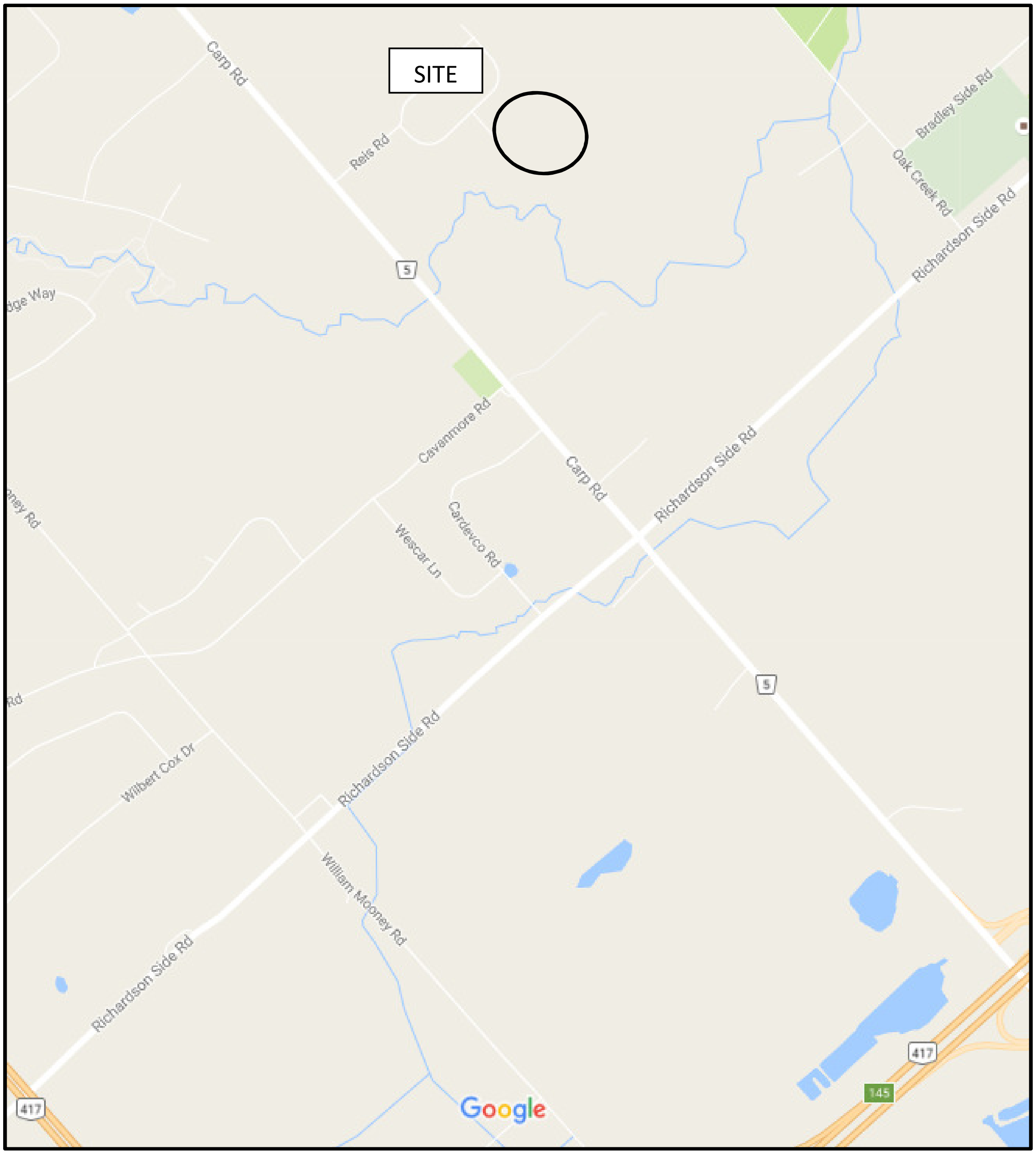
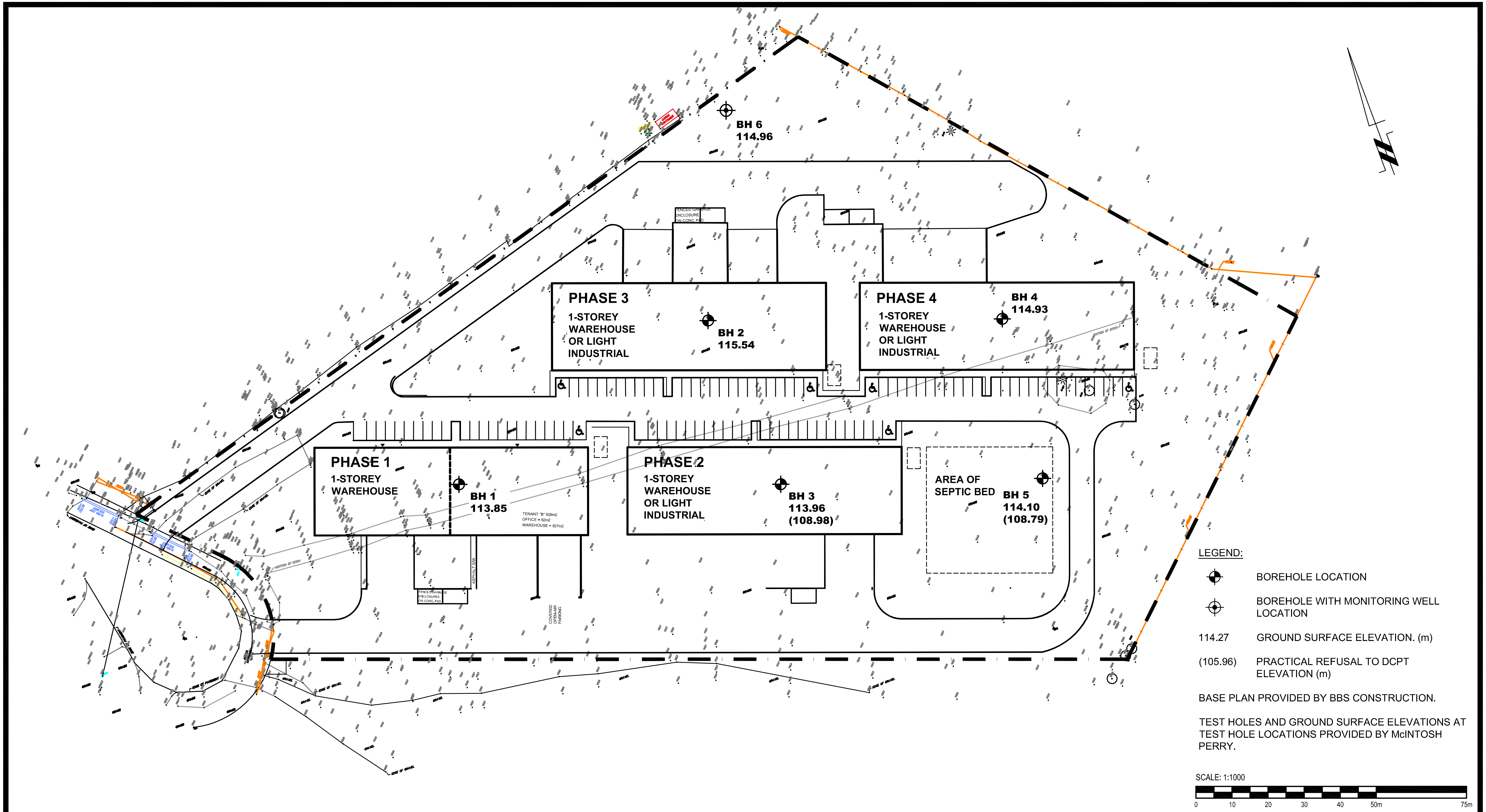


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
114.27	GROUND SURFACE ELEVATION. (m)
(105.96)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

BASE PLAN PROVIDED BY BBS CONSTRUCTION.

TEST HOLES AND GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS PROVIDED BY McINTOSH PERRY.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

2434894 ONTARIO INC. c/o BBS CONSTRUCTION

GEOTECHNICAL INVESTIGATION
PROP. WAREHOUSE DEVELOPMENT - 210 & 220 MAPLE CREEK COURT
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 11/2016
Drawn by: MPG	Report No.: PG3905-1
Checked by: RG	Dwg. No.: PG3905-1
Approved by: DJG	Revision No.: 0

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