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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Commercial Development
300-320 Moodie Drive
Ottawa, Ontario

Prepared For

Colonnade Bridgeport

June 23, 2017

Report PG4148-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade Bridgeport to conduct a geotechnical investigation for the proposed commercial development to be located at 300-320 Moodie Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the investigation were to:

- ❑ Determine the subsurface soil and groundwater conditions by means of boreholes.

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

Based on preliminary design details, it is understood that the proposed development consists of two commercial buildings (a 6 storey hotel and one storey commercial building - both of slab-on-grade construction) with associated parking areas, access lanes and landscaped areas.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on June 7 and 8, 2017. The investigation consisted of drilling 6 boreholes extending to a maximum depth of 29.6 m below ground surface. The test hole locations were selected in a manner to provide general coverage of the proposed development. The test hole locations are shown on Drawing PG4148-1-Test Hole Location Plan included in Appendix 2.

The boreholes were put down with a truck-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 from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected location, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory. The depths at which the split spoon samples were recovered from the boreholes are shown as SS on the Soil Profile and Test Data sheets in Appendix 1.

In conjunction with the recovery of the split spoon samples, the Standard Penetration Test (SPT) was conducted. 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 completed at regular intervals in cohesive soils.

Overburden thickness was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 1, BH 3 and BH 6. 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.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

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

3.2 Field Survey

The borehole locations were laid out in the field and surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM) consisting of the top of spindle of the fire hydrant located south of the subject site. A geodetic elevation of 90.48 m was provided for the TBM by Annis, O'Sullivan Vollebekk Ltd. The location and ground surface elevations at the borehole locations are presented on Drawing PG4148-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in section 6.7 and shown in appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a commercial development consisting of two low-rise commercial buildings and an associated asphalt covered parking lot. It is anticipated the two existing commercial buildings will be demolished as part of the subject development.

Associated access lanes and grass covered areas are also located around the perimeter of the site. The ground surface is relatively flat and approximately at grade with Moodie Drive and Fitzgerald Road. The subject site is bordered to the north by a tree line followed by a grass covered field, to the west by a commercial building, and to the south and east by Fitzgerald Road and Moodie Drive, respectively.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of topsoil or an asphaltic pavement structure overlying a deep, stiff to firm, silty clay deposit. Practical refusal to DCPT was encountered at depths ranging from 20.8 m to 29.6 m below ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the borehole locations.

Based on geological mapping, the bedrock consists of sandstone of the Nepean Formation. Based on available geological mapping, the overburden thickness is expected to range from 25 to 50 m.

4.3 Groundwater

The groundwater level (GWL) readings were recorded at the borehole locations on June 16, 2017 and are presented in Table 1 below and in the Soil Profile and Test Data sheets. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole due to the seasonal changes, which can lead to water perching inside the boreholes which results in higher water levels than noted during the investigation. The long-term groundwater level can also be estimated based on moisture levels and colour of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater level is expected at a 3.5 to 4.5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Level Readings				
Borehole Number	**Ground Elevation, m**	**Groundwater Levels, m**		**Recording Date**
		Depth	**Elevation**	
BH 1	89.81	2.83	86.98	June 16, 2017
BH 2	90.05	3.89	86.16	June 16, 2017
BH 3	89.76	2.10	87.66	June 16, 2017
BH 4	89.78	2.17	87.61	June 16, 2017
BH 5	89.55	2.12	87.43	June 16, 2017
BH 6	88.85	1.35	87.50	June 16, 2017
Note: The test hole locations were located in the field and surveyed by Paterson Group. The elevations are referenced to a geodetic datum provided by Annis O'Sullivan Vollebekk.				

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 buildings will be founded by conventional style shallow foundations or a raft foundation placed on an undisturbed stiff to very stiff silty clay bearing surface. End bearing piles are not expected to be a practical foundation option due to the excessive depth to bedrock.

Due to the presence of the silty clay layer, the proposed buildings will be subjected to a grade raise restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3. If higher than permissible grade raises are required, preloading with or without surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Preparation

Stripping Depth

Topsoil and fill, containing significant amounts of organic or deleterious materials, should be removed from within the proposed building footprints and other sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the perimeter of the proposed buildings. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading purposes beneath the proposed building, 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 in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and be compacted at minimum 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. Non-specified existing fill and site-excavated soils are not suitable for placement 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.

5.3 Foundation Design

It is anticipated that the one-storey commercial building can be constructed using conventional style shallow foundations. However, if bearing resistance values are insufficient for the 6 storey hotel building, consideration could be given to placing the proposed building on a raft foundation. Design recommendations for both foundation options have been provided below.

Conventional Footings

Strip footings, up to 3 m wide, and pad footings, up to 10 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings. The bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

If the above bearing resistance values provided for conventional style footings are insufficient for the proposed building, or if the founding elevation is below the stiff silty clay crust, consideration may be given to placing the proposed building on a raft foundation.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively. It is expected that the base of the slab is located at or below 4 m depth, the long term groundwater level will be at or below 4 m depth, the raft slab is impervious and the basement walls will be provided with a perimeter foundation drainage system.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **125 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **5 MPa/m** for a contact pressure of **125 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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. Above the groundwater level, adequate lateral support is provided to a stiff silty clay 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 or engineered fill.

Permissible Grade Raise

A permissible grade raise restriction has been determined for the subject site based on the undrained shear strength values completed within the silty clay deposit. Based on the testing results, a permissible grade raise restriction of **1.5 m** above existing ground surface is recommended for the subject site.

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 stores, etc). It should be noted that building over 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

The site class for seismic site response is a **Class D** for the foundations considered. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4 A) for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and/or fill, containing significant amounts of organic or deleterious materials, within the footprint of the proposed buildings, the native soil will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of a Granular A crushed stone for slab on grade construction. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compacted to, at least, 98% of the material's 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 and access lanes.

Table 1 - Recommended Flexible 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 2 - Recommended Flexible Pavement Structure - 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
400	**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 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 dy condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fin subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying 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. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter drainage system is recommended for the proposed buildings. 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 structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A drainage geocomposite, such as Delta Drain 6000 or equivalent, connected to the perimeter foundation drainage system with a positive outlet to the site storm sewer is also recommended.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, 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 temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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

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

A trench box is recommended to be installed at all times to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain exposed 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. 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 its 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 material's 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 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 material's 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.

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.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

The 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 pH of the sample indicates that it is not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas chloride content and the resistivity are indicative of an aggressive corrosive environment.

7.0 Recommendations

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

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

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

8.0 Statement of Limitations

The recommendations provided 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 recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Colonnade Bridgeport or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Colin Belcourt

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Report Distribution:

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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
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

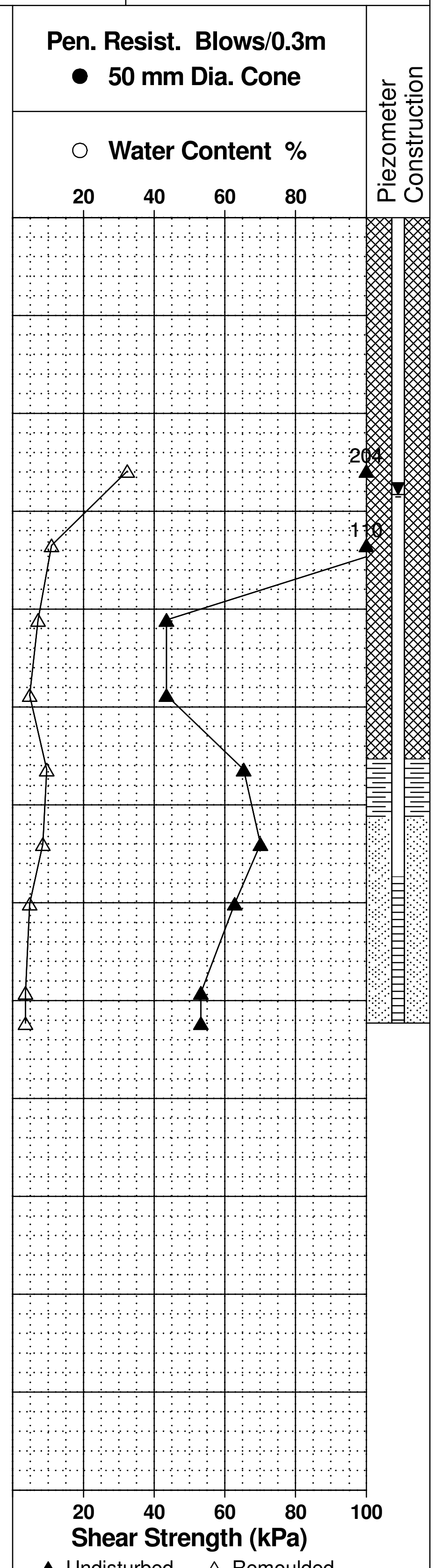
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.81
TOPSOIL 0.15		SS	1	29	4		
FILL: Brown silty sand, trace gravel and glass 0.66		SS	2	100	9	1	88.81
		SS	3	100	8	2	87.81
Hard to very stiff, brown **SILTY CLAY,** trace sand seams						3	86.81
- firm to stiff and grey by 3.7m depth						4	85.81
						5	84.81
						6	83.81
						7	82.81
8.23						8	81.81
Dynamic Cone Penetration Test commenced at 8.23m depth. Cone pushed to 20.7m depth.						9	80.81
						10	79.81
Inferred **SILTY CLAY**						11	78.81
						12	77.81
						13	76.81



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						13	76.81		
						14	75.81		
						15	74.81		
						16	73.81		
Inferred **SILTY CLAY**						17	72.81		
						18	71.81		
						19	70.81		
20.70						20	69.81		
Inferred **GLACIAL TILL** 20.75									
End of Borehole Practical DCPT refusal at 20.75m depth (GWL @ 2.83m - June 16, 2017)									

Pen. Resist. / Water Content axis: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

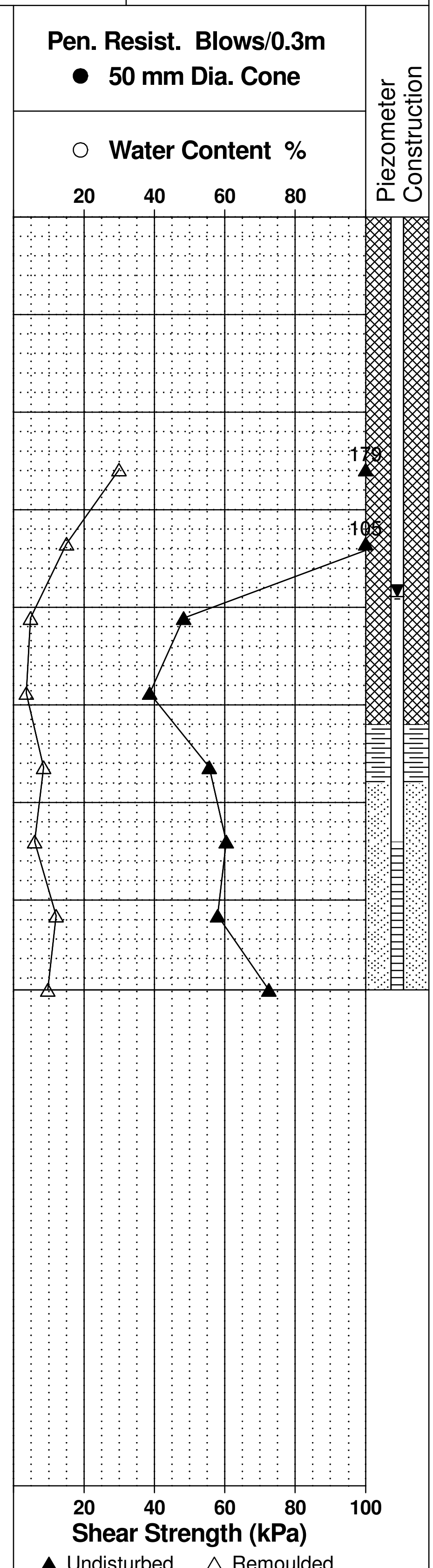
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.05
TOPSOIL 0.15		SS	1	58	9		
FILL: Brown silty sand, some gravel, cobbles and clay 0.94		SS	2	79	11	1	89.05
Very stiff to stiff, brown **SILTY CLAY,** trace sand seams - firm to stiff and grey by 3.7m depth		SS	3	100	9	2	88.05
						3	87.05
						4	86.05
						5	85.05
						6	84.05
7.92						7	83.05
End of Borehole (GWL @ 3.89m - June 16, 2017)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

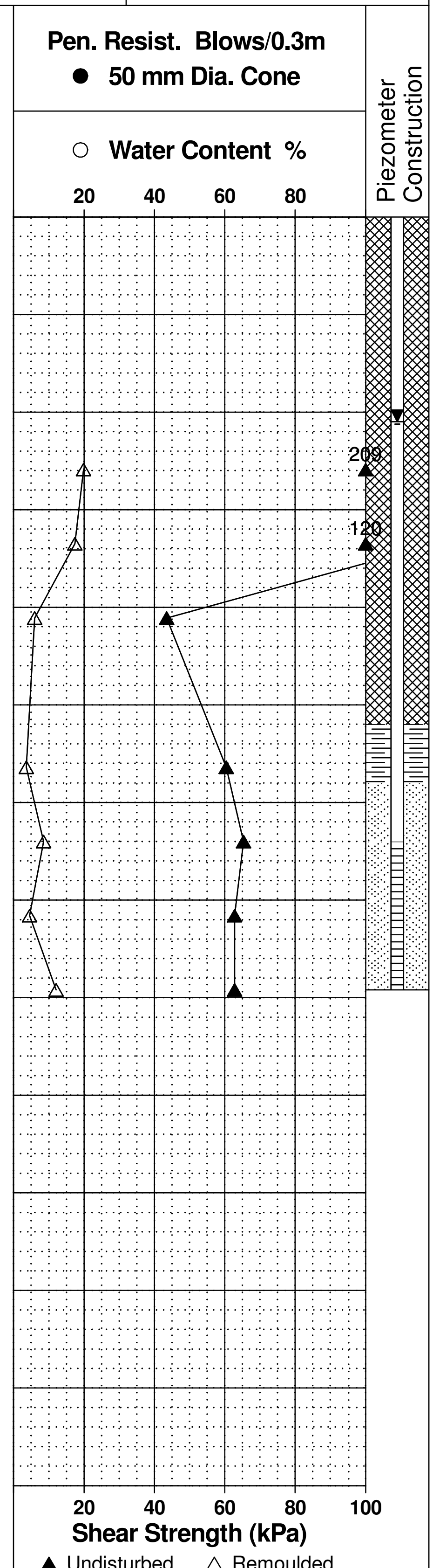
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.76
Asphaltic concrete 0.05							
FILL: Brown silty sand with crushed stone, some clay 0.41		SS	1	67	33		
0.66							
FILL: Crushed stone		SS	2	50	8	1	88.76
		SS	3	100	6	2	87.76
Hard to stiff, brown **SILTY CLAY,** trace sand seams						3	86.76
- firm to stiff and grey by 3.7m depth						4	85.76
		TW	4	100		5	84.76
						6	83.76
						7	82.76
7.92							
Dynamic Cone Penetration Test commenced at 7.92m depth. Cone pushed to 25.2m depth						8	81.76
						9	80.76
						10	79.76
Inferred **SILTY CLAY**						11	78.76
						12	77.76
						13	76.76



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

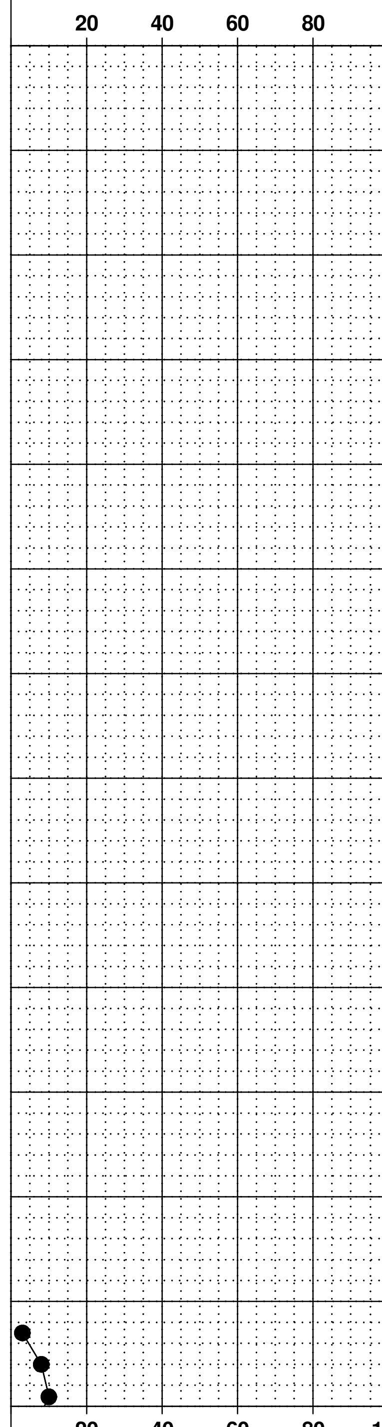
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						13	76.76		
						14	75.76		
						15	74.76		
						16	73.76		
						17	72.76		
						18	71.76		
Inferred **SILTY CLAY**						19	70.76		
						20	69.76		
						21	68.76		
						22	67.76		
						23	66.76		
						24	65.76		
25.30						25	64.76		
						26	63.76		



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
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DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

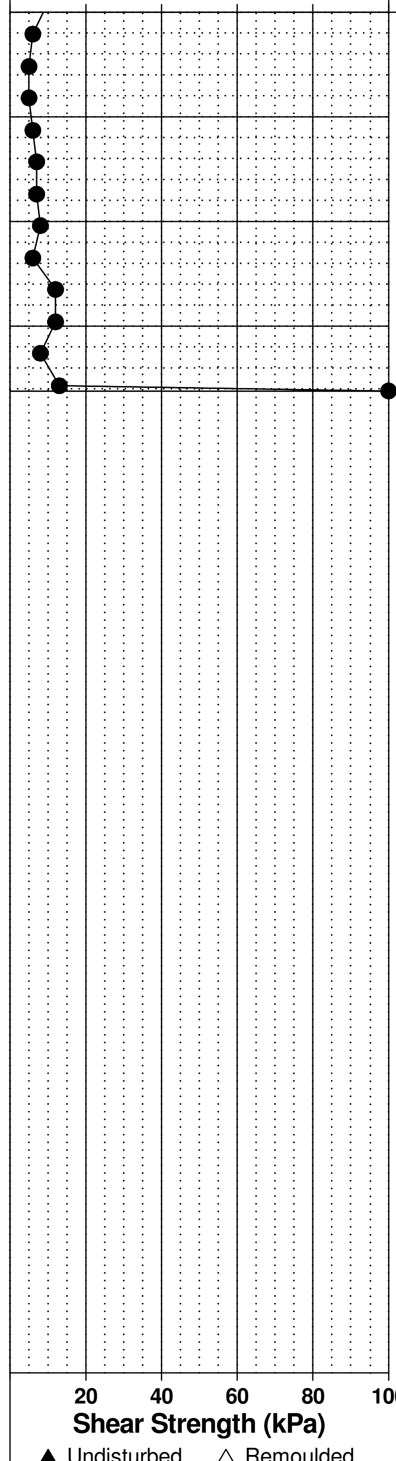
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						26	63.76		
Inferred **GLACIAL TILL**						27	62.76		
						28	61.76		
						29	60.76		
29.62									
End of Borehole Practical DCPT refusal at 29.62m depth. (GWL @ 2.10m - June 16, 2017)									



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

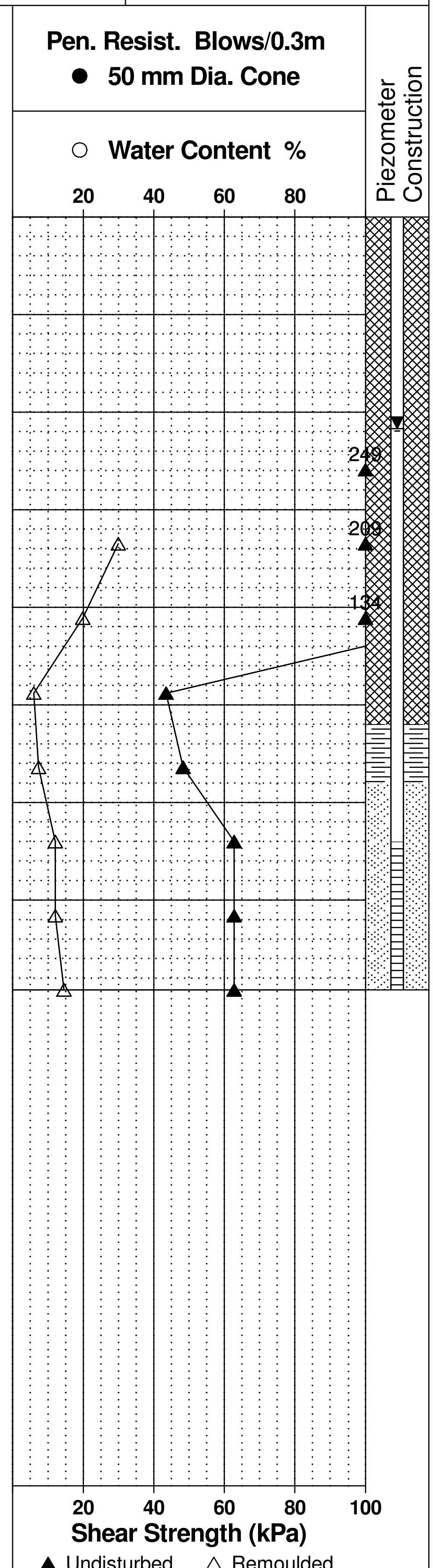
BORINGS BY CME 55 Power Auger

DATE June 7, 2017

FILE NO. PG4148

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.78
Asphaltic concrete 0.05							
FILL: Brown silty sand with crushed stone 0.99		SS	1	67	7		
		SS	2	62	6	1	88.78
		SS	3	100	9	2	87.78
Hard to stiff, brown **SILTY CLAY,** trace sand						3	86.78
- firm to stiff and grey by 4.3m depth						4	85.78
						5	84.78
						6	83.78
						7	82.78
7.92							
End of Borehole							
(GWL @ 2.17m - June 16, 2017)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

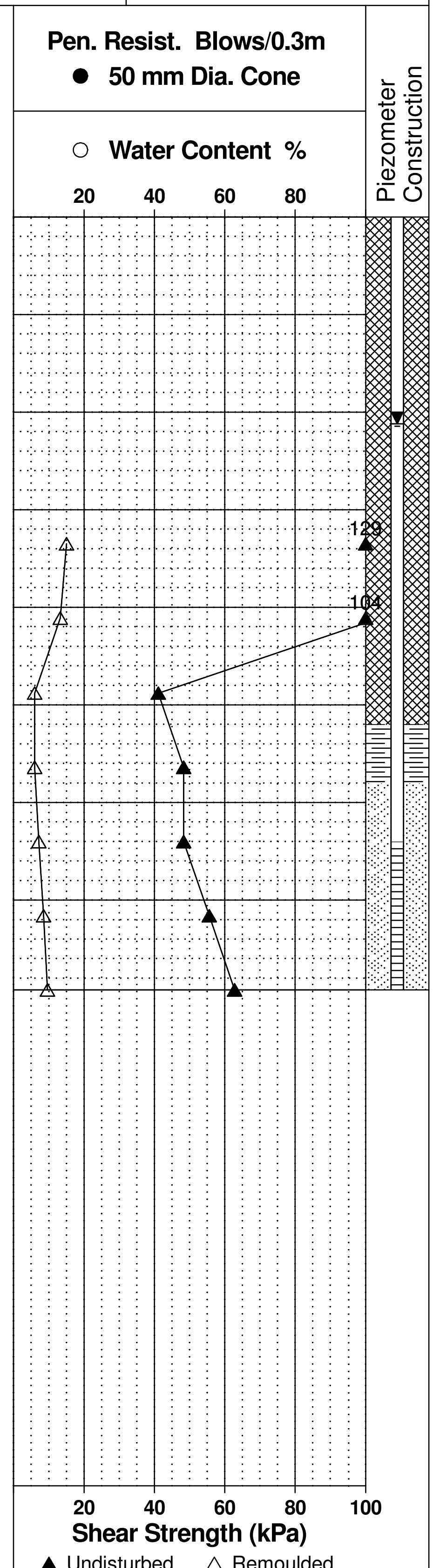
BORINGS BY CME 55 Power Auger

DATE June 8, 2017

FILE NO. PG4148

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	89.55
Asphaltic concrete 0.05		SS	1	70	38		
FILL: Brown silty sand with crushed stone 1.12		SS	2	58	11	1	88.55
		SS	3	100	12	2	87.55
		SS	4	100	10		
Very stiff to stiff, brown **SILTY CLAY,** trace sand seams						3	86.55
- firm to stiff and grey by 4.3m depth						4	85.55
						5	84.55
						6	83.55
7.92						7	82.55
End of Borehole							
(GWL @ 2.12m - June 16, 2017)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 8, 2017

FILE NO. PG4148

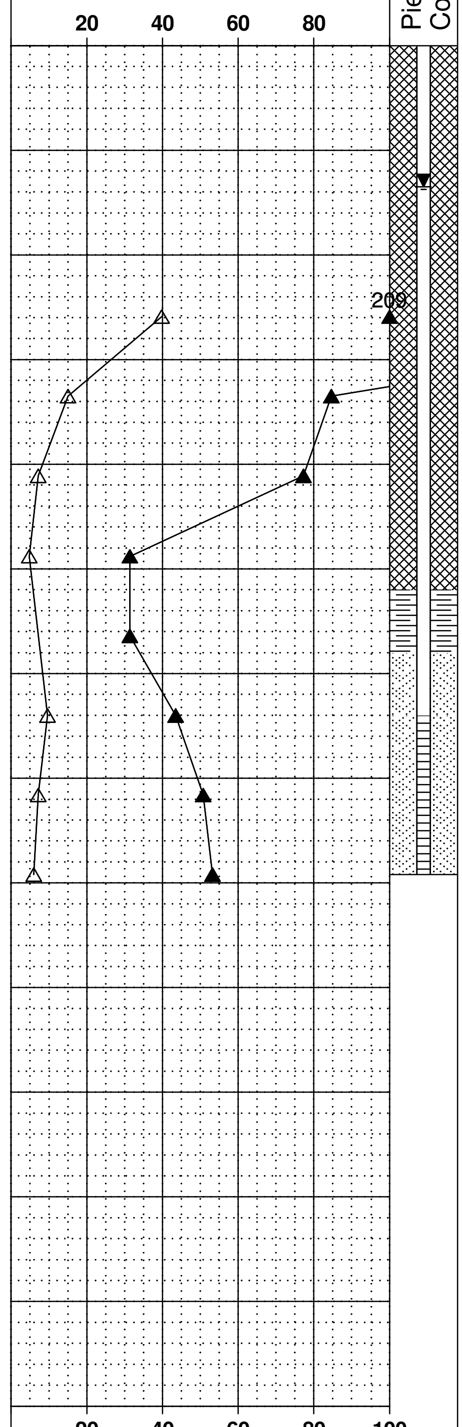
HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.85
Asphaltic concrete — 0.05							
FILL: Brown silty sand with crushed stone — 0.30		SS	1	42	7		
FILL: Black sandy silt with organic matter, some crushed stone — 1.04		SS	2	67	5	1	87.85
Hard to stiff, brown **SILTY CLAY**		SS	3	100	10	2	86.85
						3	85.85
- stiff to firm and grey by 3.7m depth						4	84.85
						5	83.85
						6	82.85
- stiff by 6.7m depth — 7.92						7	81.85
Dynamic Cone Penetration Test commenced at 7.92m depth. Cone pushed to 24.2m depth						8	80.85
						9	79.85
						10	78.85
Inferred **SILTY CLAY**						11	77.85
						12	76.85
						13	75.85

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 300-320 Moodie Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Fitzgerald Road, opposite the southeast corner of site. Geodetic elevation = 90.48m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** June 8, 2017

FILE NO. PG4148

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
Inferred **SILTY CLAY**						13	75.85		
						14	74.85		
						15	73.85		
						16	72.85		
						17	71.85		
						18	70.85		
						19	69.85		
						20	68.85		
						21	67.85		
						22	66.85		
						23	65.85		
24.20						24	64.85		
Inferred **GLACIAL TILL** 24.28									
End of Borehole Practical DCPT refusal at 24.28m depth (GWL @ 1.35m - June 16, 2017)									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

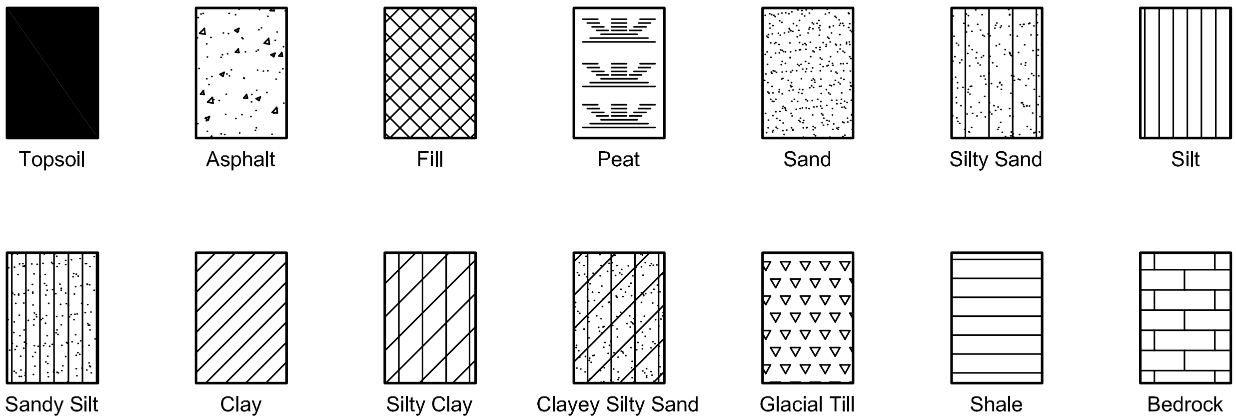
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

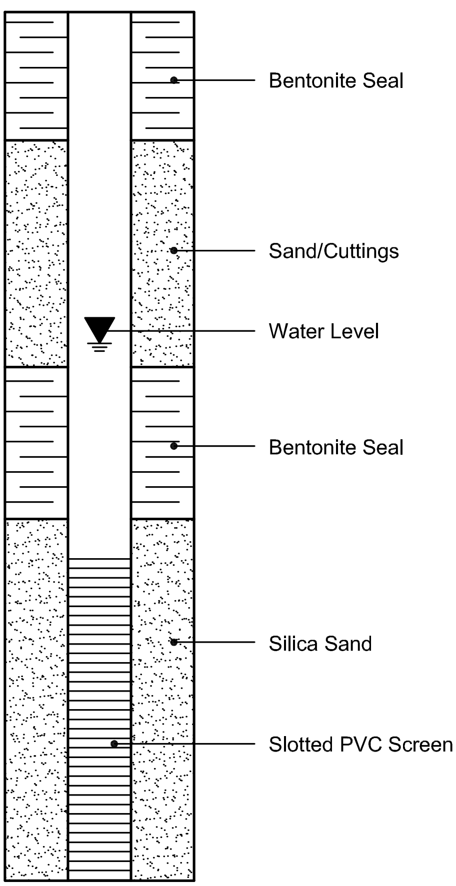
SYMBOLS AND TERMS (continued)

STRATA PLOT

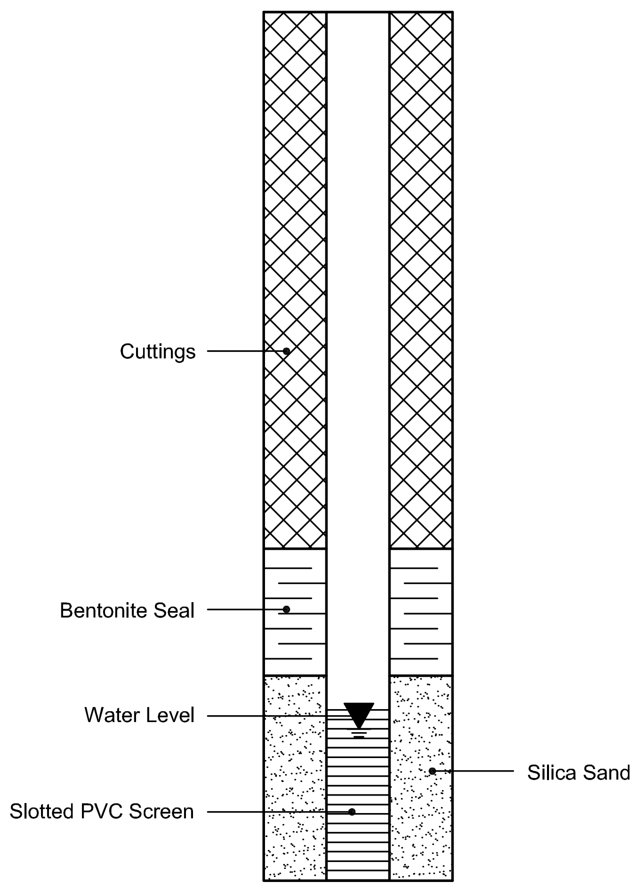


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1724312

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 20615

Report Date: 20-Jun-2017

Order Date: 14-Jun-2017

Project Description: PG4148

	Client ID:	BH4 SS3	-	-	-
	Sample Date:	07-Jun-17	-	-	-
	Sample ID:	1724312-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.13	-	-	-
Resistivity	0.10 Ohm.m	1.96	-	-	-
Anions					
Chloride	5 ug/g dry	3480	-	-	-
Sulphate	5 ug/g dry	136	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4148-1 - TEST HOLE LOCATION PLAN

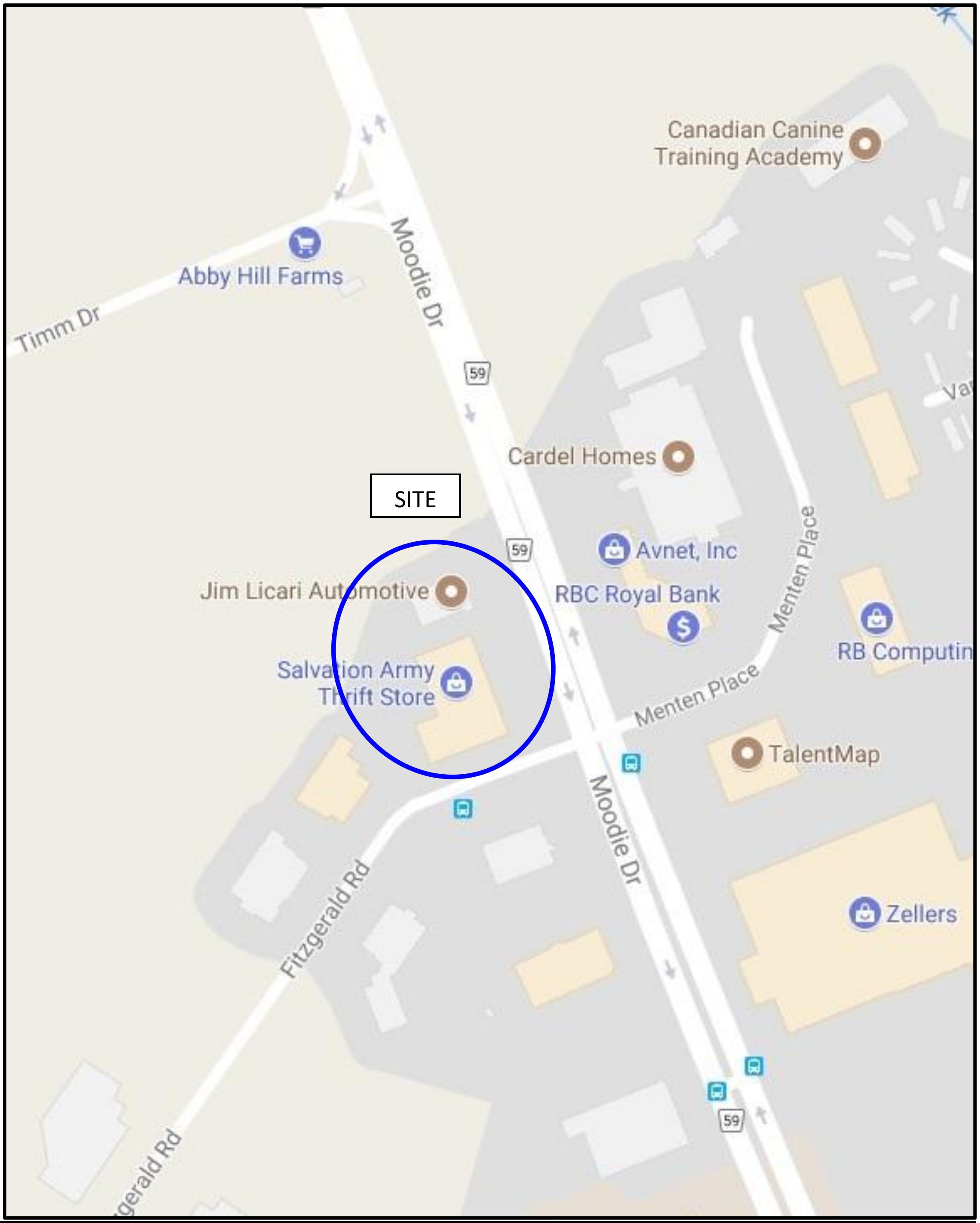
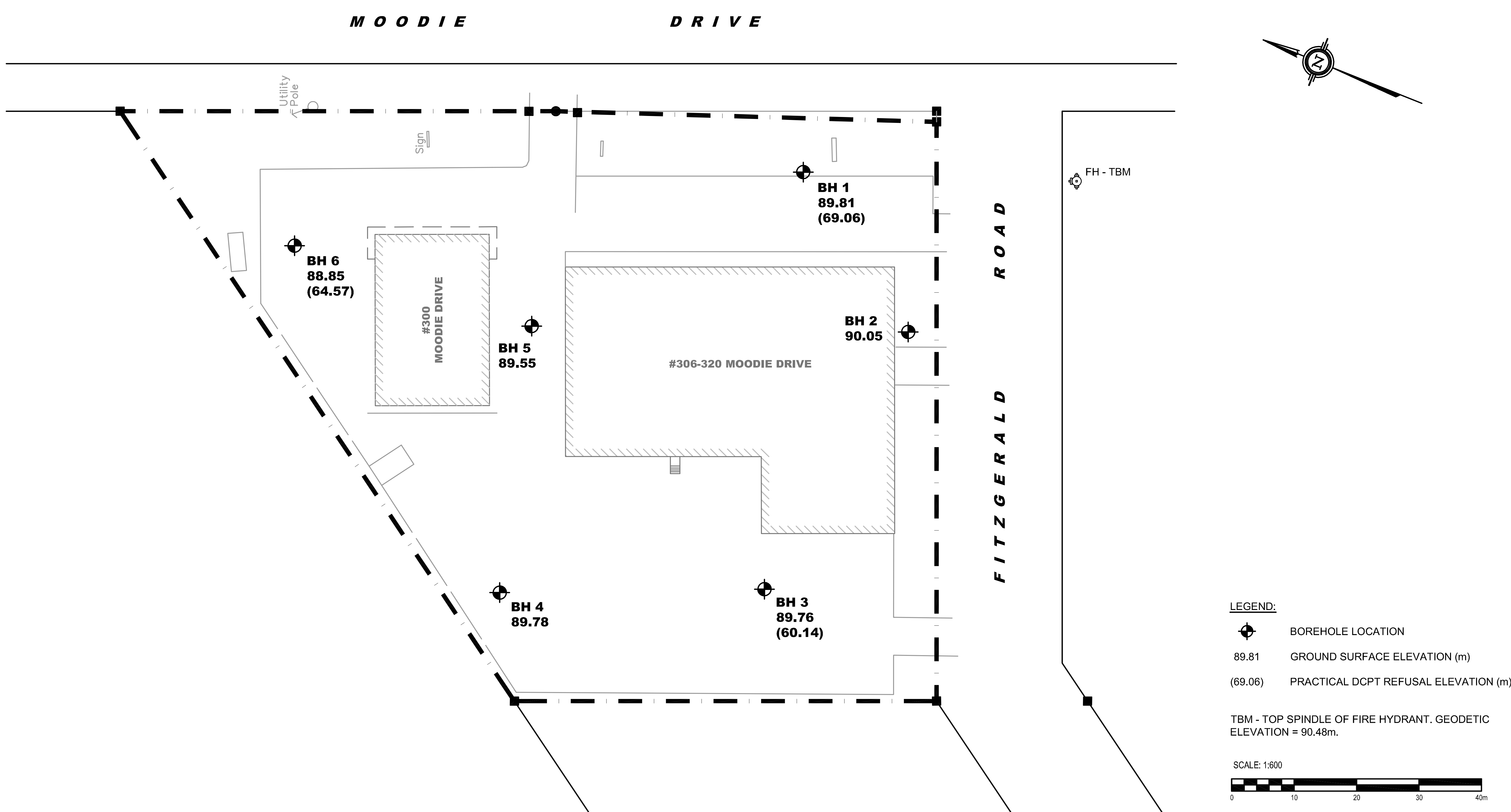


FIGURE 1

KEY PLAN

patersongroup



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NO.	REVISIONS	DATE	INITIAL
0			

COLONNADE BRIDGEPORT
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL DEVELOPMENT - 300-320 MOODIE DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 06/2017
Drawn by: MPG	Report No.: PG4148-1
Checked by: DJG	Dwg. No.: PG4148-1
Approved by: DJG	Revision No.: 0

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