Geotechnical Engineering

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PLANNING, REAL ESTATE & ECONOMIC DEVELOPMENT DEPARTMENT, CITY OF OTTAWA

APPROVED

By Allison Hamlin at 1:18 pm, May 05, 2022

Geotechnical Investigation

Proposed Commercial Development
Kanata West Business Park - Block 4
8800 Campeau Drive
Ottawa, Ontario

Prepared For

Maritime Ontario

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November 24, 2020

Report: PG5618-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Maritime Ontario to conduct a geotechnical investigation for the proposed warehouse/office to be located at 8800 Campeau Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

- ❏ determine the subsoil and groundwater conditions at this site by means of the existing borehole coverage from the original geotechnical investigation completed for the existing building.

- ❏ 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed commercial development will consist of a 2-storey office building with one underground level which is connected to an approximate 5,600 m^2, 1-storey warehouse of slab-on-grade construction. It is further understood that associated access roads, parking areas and landscaped areas will occupy the remainder of the site. The site is also anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

A previous investigation was complete at the subject site by this firm in January 2014. At that time, a total of 5 boreholes were advanced at the subject site to a maximum depth of 6.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The approximate locations of the boreholes are shown on Drawing PG5618-1 - Test Hole Location Plan included in Appendix 2.

All boreholes were advanced using a track-mounted auger drill rig, which was operated by a two person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths and at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted at each borehole 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 overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 1. 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 presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

The observed groundwater levels were recorded in the field. Ground observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson and laid out in the field by Stantec Geomatics to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. It is understood that the test hole locations and ground surface elevation at each test hole location are referenced to a geodetic datum. The location of the test holes and ground surface elevations at each test hole location are presented on Drawing PG5618-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soils 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.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant and has been stripped of topsoil and trees. A gravel surfaced access road was previously located within the central portion of the site, running southwest to northeast. Fill piles were observed in the northeast corner of the property. The subject site is bordered to the north by agricultural lands to the east by future Upper Canada Street and vacant properties, to the south by Campeau Drive and to the west by commercial aggregate quarrying operations. The existing ground surface across the subject site is relatively flat with an approximate geodetic elevation of 105 to 106 m.

4.2 Subsurface Profile

At the time of the original field investigation, the subsurface profile at the test hole location was generally observed to consist of a thin topsoil underlain by a compact to very loose brown silty sand deposit. A glacial till deposit, consisting of grey silty sand, with gravel, cobbles and boulders, was encountered underlying all boreholes with the exception of BH 18. Practical refusal to DCPT was encountered at a depth of 8.6 m in borehole BH 1. It is expected that the topsoil layer was removed since the time of the original investigation.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Based on available geological mapping, the bedrock at the subject site consists of interbedded limestone and shale of the Verulam formation as well as interbedded limestone and dolomite of the Gull River formation with an overburden thickness of 5 to 15 m.

4.3 Groundwater

The groundwater level was observed at an approximate depth of 1.5 to 2.0 m in the boreholes. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed warehouse and office building will be founded on conventional shallow footings bearing on an undisturbed silty sand and/or glacial till bearing surface.

Where the footing subgrade consists of silty sand which is observed to be in a loose state of compactness, the material should be proof compacted using suitable vibratory equipment making several passes under dry conditions and above freezing temperatures and which is approved by Paterson at the time of construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Fill Placement

Fill used for grading beneath the proposed building footprint, 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. 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 and paved areas should be compacted to at least 98% of the material's 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 at least 95% of their respective SPMDD.

5.3 Foundation Design

Conventional Spread Footings

Footings placed directly on an undisturbed, compact silty sand 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 **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footings placed on an undisturbed glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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 placed on a soil bearing surface and 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 silty clay or glacial till bearing medium 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 of the same or higher capacity as the soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. If a higher seismic site class is required (Class C), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building addition, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill, silty sand, or glacial till subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade or basement slab construction. Where the subgrade consists of existing fill or silty sand, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

For slab-on-grade areas, it is recommended that the upper 200 mm of sub-slab fill consist OPSS Granular A crushed stone. For basement slabs, it is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should also be provided under the basement slab areas. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Car only parking areas, access lanes and heavy truck parking/loading areas are anticipated at this site. The proposed pavement structures are shown in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking/loading 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.

5.8 Percolation Rates

Infiltration galleries are anticipated to be located beneath the asphaltic parking areas within the subject site. Paterson completed a detailed hydrogeological investigation of the lands southwest of the subject site as part of previous phases of the Kanata West Business Park in order to establish hydraulic conductivity and percolation time of in-situ materials.

It is anticipated that a silty sand will be encountered at the base of the infiltration galleries during the installation and will affect the rate of stormwater infiltration into the underlying material. The percolation rate was interpreted from the hydraulic conductivity which was estimated based on previous investigations within the area and on experience. Based on these values, the average percolation rate (T-Time) was estimated to be within the ranges in Table 4.

Table 4 - Estimated Percolation Rates

Material	Hydraulic Conductivity - k (m/sec)	Percolation (T-time) - (mins/cm)
Silty Fine Sand [1]	**1×10^{-7} to 1×10^{-8}**	**20 to 50**

[1] - Values are based upon site specific testing carried out at a nearby phase of the development

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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 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 catch basins storm sewer.

Sub-slab drainage is also recommended in order to control groundwater infiltration for basement slab areas. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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

6.2 Protection 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 thick soil cover should be provided for adequate frost protection of heated structured, or an equivalent combination of soil cover and foundation insulation.

Exterior unheated footings, such as those for isolated exterior piers and loading docks, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the 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 expected that sufficient room will be available for the greater part of the excavation to be undertake by open-cut methods (i.e. unsupported excavations).

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 shallower. The shallower slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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 pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 98% of the material’s SPMDD.

It should generally be possible to re-use the site excavated materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface ares are considered above the trench backfill, the trench backfill material within the frost zone, (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick lifts and compacted to 95% of the materials SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. 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.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

The subsoil conditions at this site 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 use of straw, propane heaters, 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.

7.0 Recommendations

It is a requirement for the foundation data provided herein to be applicble that the following material testing and observation program be performed by the geotechnical consultant.

- ❑ A review of the site grading plan(s) from a geotechnical perspective, once available.
- ❑ 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 placing backfilling materials.
- ❑ 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 Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation 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 permission to review our recommendations when the drawings and specifications are completed.

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 Maritime Ontario or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 24, 2020
D. J. GILBERT
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PROVINCE OF ONTARIO

Kevin A Pickard, EIT

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Maritime Ontario
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. PG3115

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger

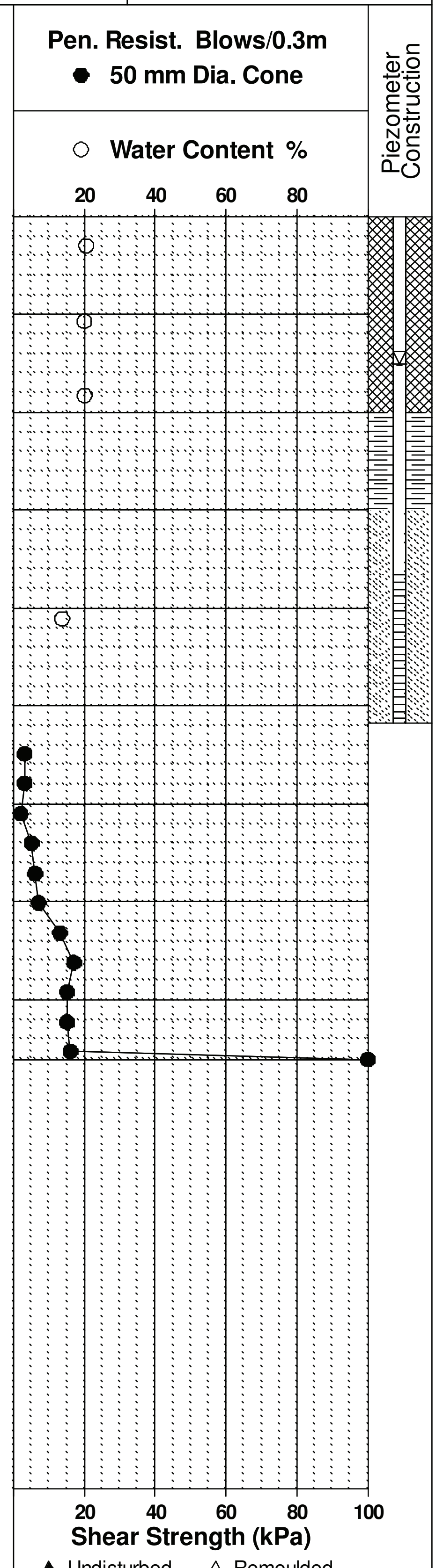
DATE January 15, 2014

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.99
TOPSOIL	0.28						
Compact, brown **SILTY FINE SAND** - grey by 1.5m depth		AU	1				
		SS	2	71	11	1	104.99
		SS	3	54	11	2	103.99
	2.97	SS	4	25	13		
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	5	58	3	3	102.99
		SS	6	67	17	4	101.99
	5.18	SS	7	50	3	5	100.99
Dynamic Cone Penetration Test commenced at 5.18m depth. Inferred **GLACIAL TILL**						6	99.99
						7	98.99
	8.61					8	97.99

End of Borehole

Practical DCPT refusal at 8.61m depth

(GWL @ 1.5m depth based on field observations)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. PG3115

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.99
TOPSOIL 0.36		AU	1				
Brown **SILTY CLAY,** trace sand 0.69		AU	2				
Compact, brown **SILTY FINE SAND**		SS	3	100	14	1	103.99
		SS	4	83	12	2	102.99
		SS	5	71	8		
- grey by 3.0m depth		SS	6	58	9	3	101.99
		SS	7	54	6	4	100.99
		SS	8	58	10	5	99.99
6.02		SS	9	58	12	6	98.99
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, trace clay 6.70		SS	10	67	24		
End of Borehole							
(GWL @ 1.8m depth based on field observations)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

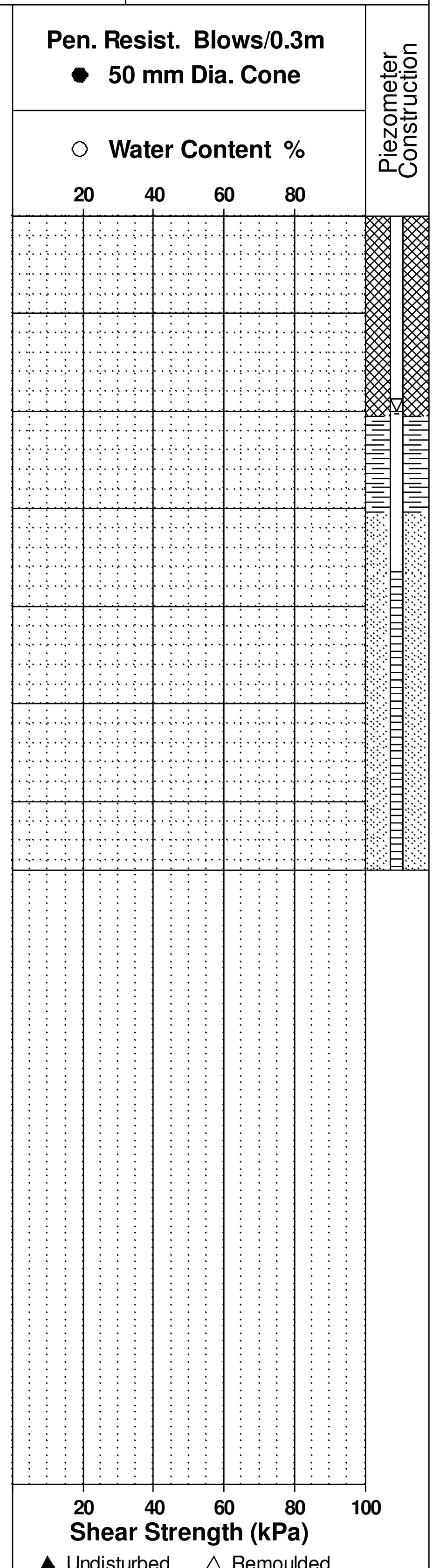
FILE NO. PG3115

REMARKS

HOLE NO. BH17

BORINGS BY CME 55 Power Auger **DATE** January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.68
TOPSOIL 0.30							
Loose, grey **SILTY FINE SAND**		SS	1	83	10	1	104.68
		SS	2	54	8	2	103.68
		SS	3	67	2		
		SS	4	62	3	3	102.68
		SS	5	67	8	4	101.68
		SS	6	67	5	5	100.68
6.00		SS	7	54	2	6	99.68
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders 6.70		SS	8	50	3		
End of Borehole (GWL @ 2.0m depth based on field observations)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
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DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

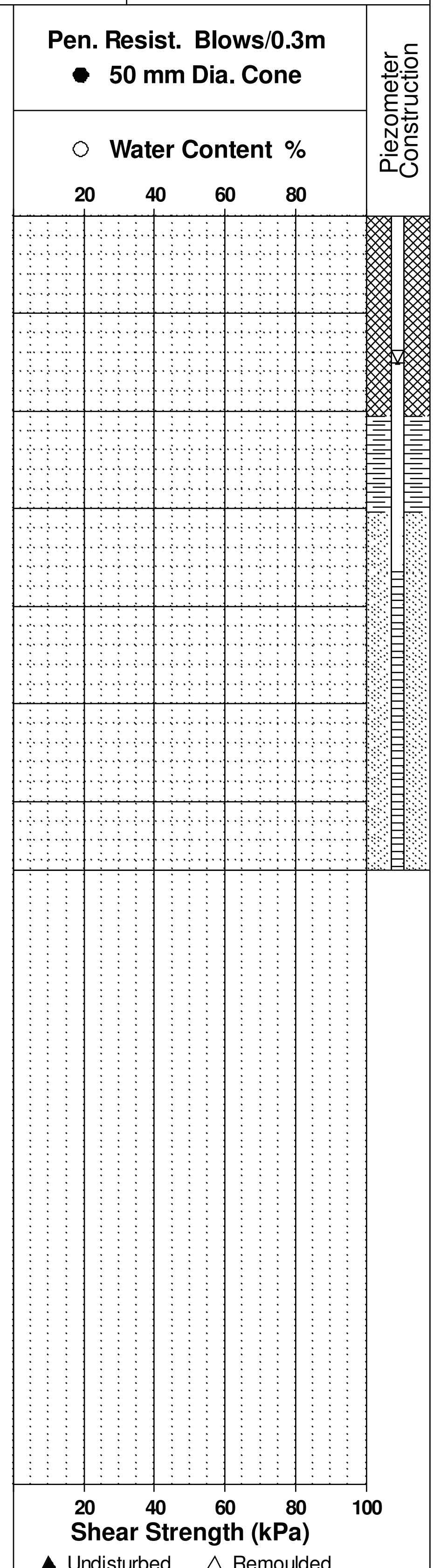
FILE NO. PG3115

REMARKS

HOLE NO. BH18

BORINGS BY CME 55 Power Auger **DATE** January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.53
TOPSOIL 0.20		AU	1				
Compact to loose, brown **SILTY FINE SAND** - grey by 3.0m depth		SS	2	83	11	1	104.53
		SS	3	62	13	2	103.53
		SS	4	67	9	3	102.53
		SS	5	58	7		
		SS	6	58	10	4	101.53
		SS	7	71	7	5	100.53
		SS	8	67	6	6	99.53
6.70		SS	9	50	3		
End of Borehole (GWL @ 1.5m depth based on field observations)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. **PG3115**

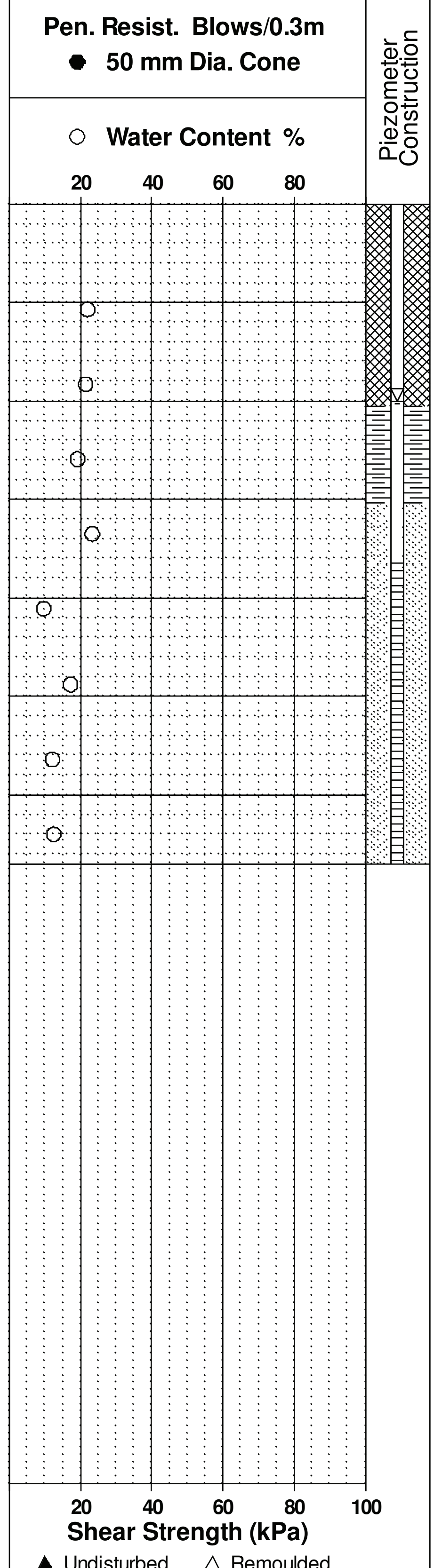
REMARKS

HOLE NO. **BH19**

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.08
TOPSOIL 0.23							
		SS	1	62	12	1	104.08
		SS	2	71	1	2	103.08
Compact o very loose, brown **SILTY FINE SAND**		SS	3	62	1		
- grey-brown by 2.2m depth						3	102.08
- trace gravel below 3.0m depth		SS	4	67	8		
- grey by 3.7m depth		SS	5	83	1	4	101.08
5.26		SS	6	25	3	5	100.08
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	7	100	14	6	99.08
6.70		SS	8	100	32		
End of Borehole							
(GWL @ 2.0m depth based on field observations)							



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

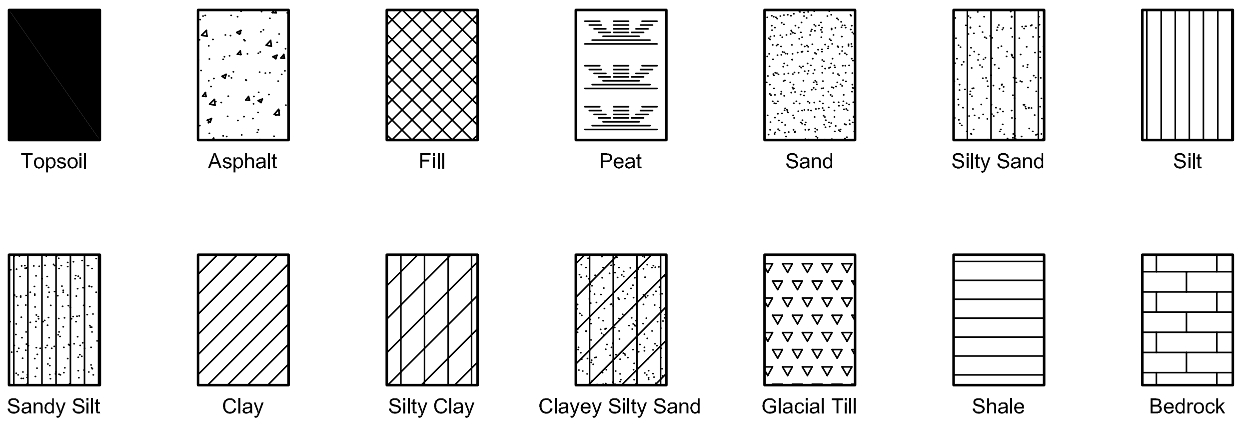
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

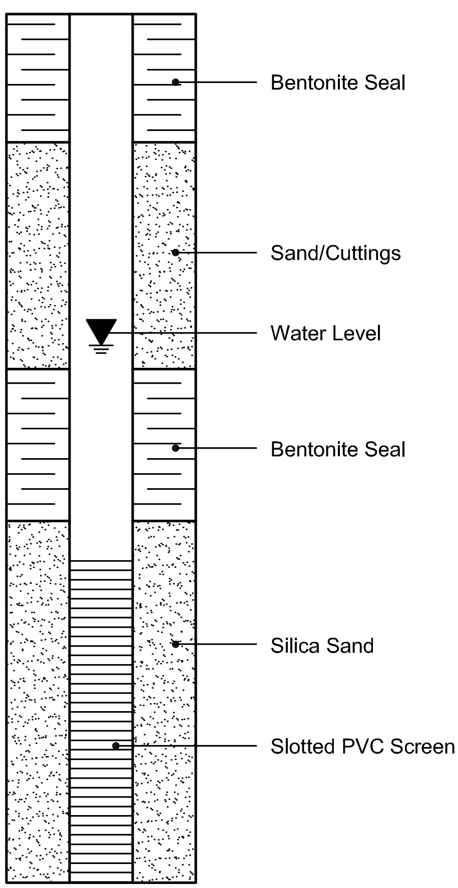
SYMBOLS AND TERMS (continued)

STRATA PLOT

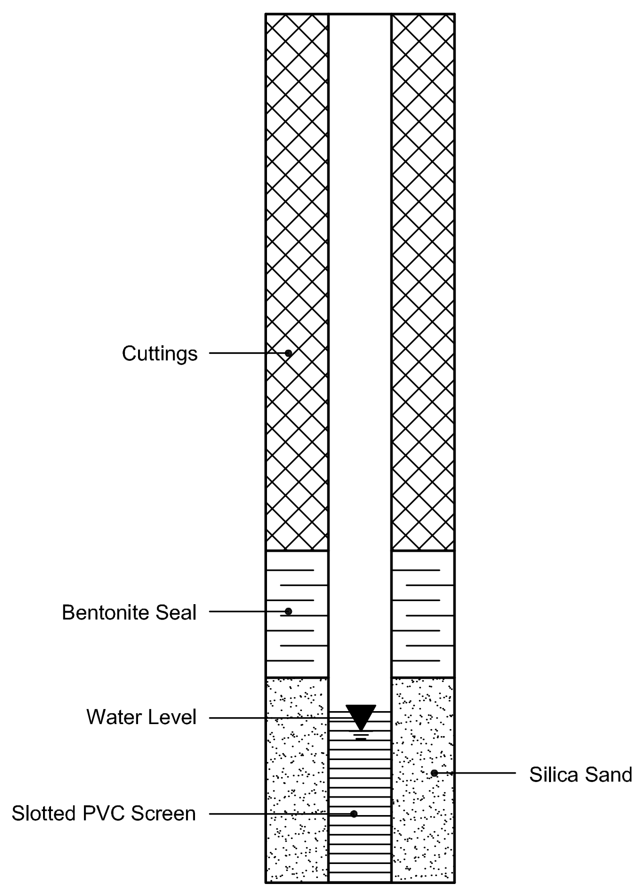


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5618-1 - TEST HOLE LOCATION PLAN

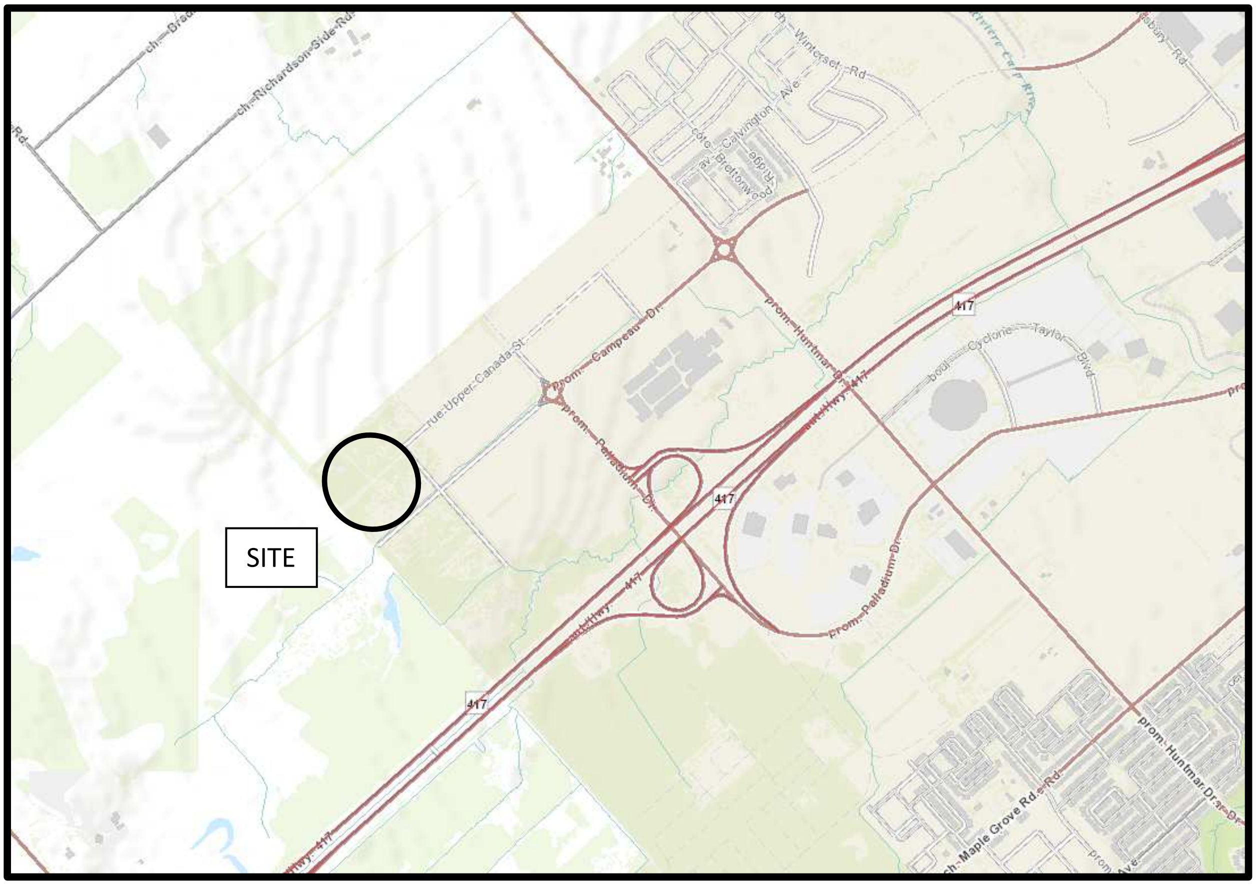
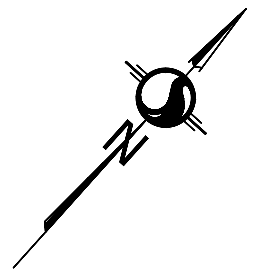
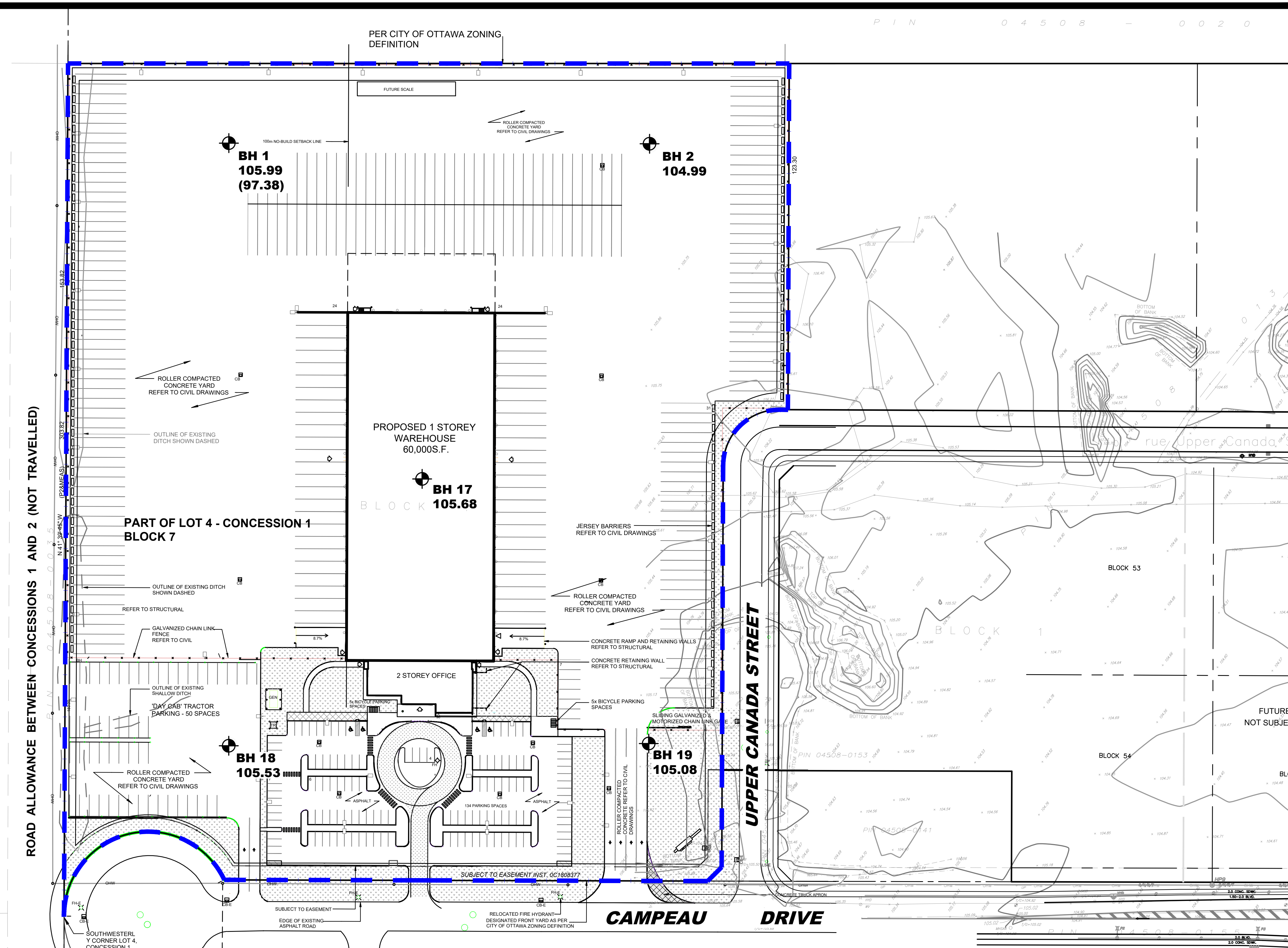


FIGURE 1

KEY PLAN

patersongroup





LEGEND:

⊕	BOREHOLE LOCATION (PREVIOUS INVESTIGATION , PG3115)
105.99	GROUND SURFACE ELEVATION (m)
(97.38)	PRACTICAL REFUSAL TO DCPT/AUGERING ELEVATION (m)

TBM-TOP OF STORM WATER MANHOLE NORTH WEST OF THE INTERSECTION OF PALLADIUM AND CAMPEAU DRIV. GEODETIC ELEVATION=104.53m PROVIDED BY STANTEC GEOMATICS LTD.

SCALE: 1:1500

0 25 50 75 100m

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NO.	REVISIONS	DATE	INITIAL

MARITIME ONTARIO

GEOTECHNICAL INVESTIGATION

PROPOSED WAREHOUSE AND OFFICE BUILDINGS - KANATA WEST BUSINESS PARK - BLOCK 4 - CAMPEAU DR.

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500	Date: 11/2020
Drawn by: RCG	Report No.: PG5618-1
Checked by: KP	Dwg. No.: PG5618-1
Approved by: DJG	Revision No.: 1

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