Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

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Geotechnical Investigation

Proposed Multi-Storey Building
2487 Innes Road
Ottawa, Ontario

Prepared For

Ottawa Carleton Construction Group Ltd.

June 8, 2020

Report: PG5171-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa Carleton Construction Group Ltd. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 2487 Innes Road, in the City of Ottawa, Ontario (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 test holes.

❑ provide geotechnical recommendations for the design of the proposed development based on the results of the test holes and other soil information available.

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

2.0 Proposed Development

Based on available conceptual plans, the proposed development will consist of a three-storey apartment building with one basement level. Associated at-grade parking areas, access lanes and landscaped areas are also anticipated. It is understood that existing buildings on the site will be demolished as part of the proposed development. It is anticipated that the subject site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on November 27, 2019. At that time, four (4) boreholes were advanced to a maximum depth of 6.7 m below existing ground surface. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG5171-1 - Test Hole Location Plan in Appendix 2.

The boreholes were drilled with a track-mounted drill rig 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 from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling and testing the overburden.

Sampling and In Situ Testing

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

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

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

The overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 4. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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

Flexible polyethylene standpipes were installed within all boreholes with the exception of BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All soil 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 in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration existing site features. Ground surface elevations were referenced to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located along Innes road across from the subject site. A geodetic elevation of 75.739 m was determined for the TBM. The location of the test holes and TBM are presented on Drawing PG5171-1 - 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.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the sulphate and chloride concentration, 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 is currently occupied by an existing two-storey residential dwelling with an associated garage structure, asphalt parking lane and a grass-covered side yard. The front and side yards are mainly grass covered with some trees noted along Innes Road. The subject site is bordered by two-storey residential dwellings to the north and west, Innes Road followed by three-storey townhouses to the south and two-storey townhouses to the east. The ground surface across the subject site is generally flat and at grade with Innes Road and the surrounding properties with the exception of a slight grade raise of approximately 0.5 m around the existing two-storey residential dwelling.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of a silty sand fill layer overlying a firm to stiff, grey silty clay. Practical refusal to DCPT was encountered at 29.6 m depth in BH 4. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of shale of the Billings formation with an anticipated overburden thickness of 25 to 50 m.

4.3 Groundwater

Groundwater levels were measured in the piezometers installed at the borehole locations. The groundwater level observations are presented on the Soil Profile and Test Data sheets in Appendix 1 and in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	74.14	3.85	70.29	December 11, 2019
BH 2	74.22	3.00	71.22	December 11, 2019
BH 4	74.35	3.22	71.13	December 11, 2019

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface. It should also be noted that groundwater levels are subject to seasonal fluctuations, and therefore the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered satisfactory for the proposed development from a geotechnical perspective. It is anticipated that the proposed building can be constructed using conventional shallow footings placed on an undisturbed, stiff silty clay and/or approved compact silty sand fill bearing surface.

Due to the presence of a silty clay deposit, a permissible grade raise restriction will be required for the subject site.

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, paved areas, pipe bedding and other settlement sensitive structures. It is anticipated that the existing fill, free of significant amounts of deleterious material and organics, can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the slab subgrade surface be proof-rolled **under dry conditions and above freezing temperatures** by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson at the time of construction.

In poor performing areas, the existing fill should be removed and replaced with an approved engineered fill. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. 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 beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building 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 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. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff silty clay and/or an approved compact silty sand fill bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **75 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa** incorporating a geotechnical resistance factor of 0.5 at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of one 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.

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

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 silty sand or stiff silty clay above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise

Based on the undrained shear strength testing results and experience with the local silty clay deposit, a permissible grade raise restriction for the subject site of **1.5 m** can be used for design purposes.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations considered at this site. The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil at the existing ground surface and deleterious fill, such as those containing significant amounts of organic materials within the footprint of the proposed building, the existing fill approved by the geotechnical consultant at the time of construction will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is recommended that the existing fill be proof-rolled with a suitably sized roller making several passes under dry condition prior to fill placement and approved by the geotechnical consultant. Any poor performing areas should be removed and replaced with OPSS Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the material's SPMDD. It is recommended that the upper 200 mm of sub-lab fill consist of 19 mm clear crushed stoned for the basement slab.

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, in our opinion, 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 dry unit weight of 20 kN/m^3.

The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by 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)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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. The vertical seismic coefficient is assumed to be zero.

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

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

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and access lanes.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ silty clay 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 100% 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.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. 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 structure. 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 and placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or equivalent. 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 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 footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of heated structures 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 excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

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 to stiff 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 99% 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 loose lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) grey 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. It is anticipated that groundwater infiltration into the excavations should be low and controllable using conventional open sumps.

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

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

Impacts on Neighbouring Structures

It is understood that one basement level is planned for the proposed building. Based on the existing groundwater level and low permeability of the adjacent soils, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

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 into 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 analytical testing results 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 moderately aggressive corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable, a materials testing and observation services program is required to be completed. The following aspects should be performed by the geotechnical consultant:

- ❑ A review of the site grading plan(s) from a geotechnical perspective, once available.
- ❑ A review of architectural and structural drawings to ensure adequate frost protection is provided to the subsoil.
- ❑ 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 the construction has been conducted in general accordance with the 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 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 recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

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 Ottawa Carleton Construction Group Ltd. 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
June 9-2020
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PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng. David J. Gilbert, P.Eng.

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
Proposed Multi-Storey Building - 2487 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

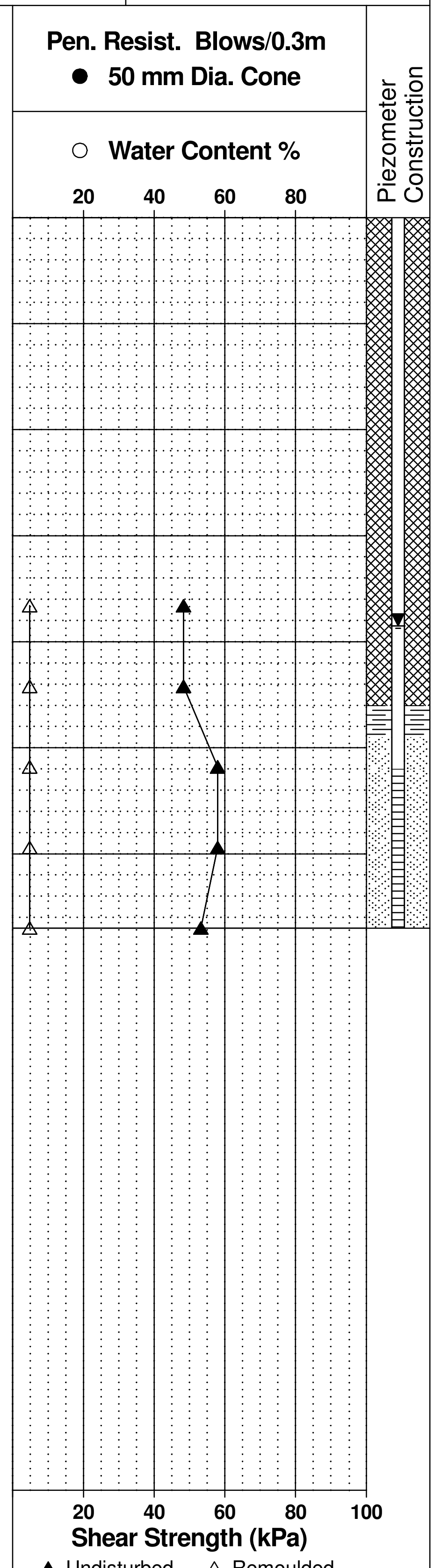
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 2019 November 27

FILE NO. PG5171

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.14
FILL: Brown silty sand, some organics		AU	1				
		SS	2	58	12	1	73.14
		SS	3	54	13	2	72.14
2.29							
Firm to stiff, grey **SILTY CLAY**		SS	4	96	2	3	71.14
						4	70.14
						5	69.14
						6	68.14
6.70							
End of Borehole							
(GWL @ 3.85m - Dec. 11, 2019)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 2487 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 27

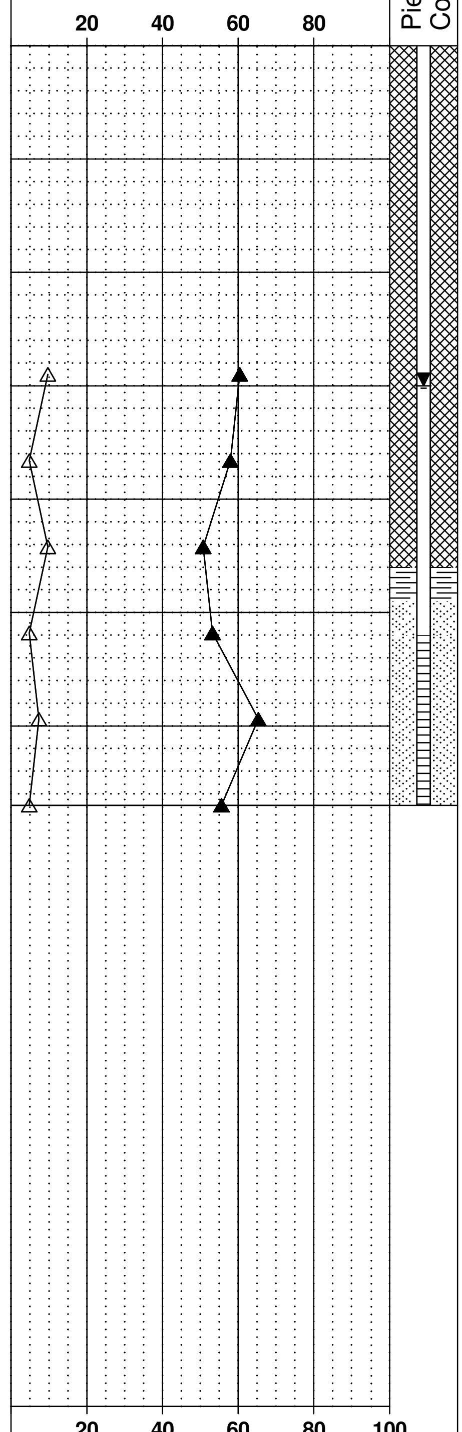
FILE NO. PG5171

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.22
FILL: Brown silty sand, some organics	AU	1				
	SS	2	79	13	1	73.22
	SS	3	50	11		
2.13					2	72.22
Stiff, grey **SILTY CLAY**					3	71.22
					4	70.22
					5	69.22
					6	68.22
6.70						
End of Borehole						
(GWL @ 3.00m - Dec. 11, 2019)						

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
Proposed Multi-Storey Building - 2487 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 27

FILE NO. PG5171

HOLE NO. BH 3

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 % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	74.46		
FILL: Brown silty sand, trace topsoil 0.08		AU	1						
FILL: Brown silty sand		SS	2	58	10	1	73.46		
2.13		SS	3	50	11	2	72.46		
End of Borehole									
(BH dry upon completion)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 2487 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

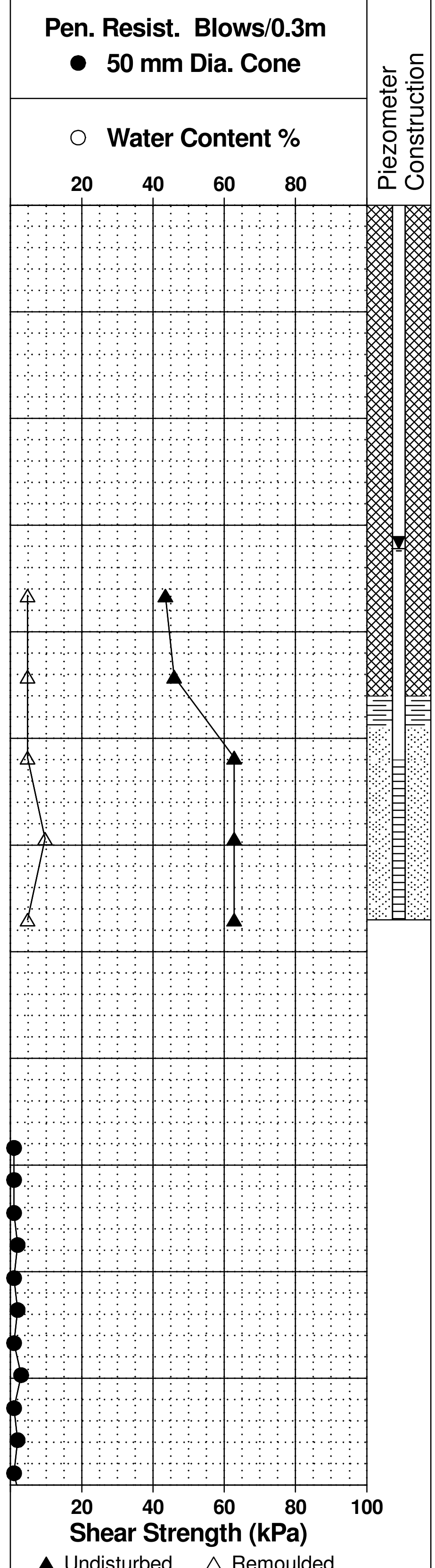
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 2019 November 27

FILE NO. PG5171

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.35
FILL: Brown silty sand	AU	1				
	SS	2	75	11	1	73.35
	SS	3	62	11	2	72.35
2.44	SS	4	50	3	3	71.35
Firm to stiff, grey **SILTY CLAY**					4	70.35
					5	69.35
					6	68.35
6.70						
Dynamic Cone Penetration Test commenced at 6.70m depth. Cone pushed to 9.4m depth.					7	67.35
					8	66.35
					9	65.35
					10	64.35
					11	63.35
					12	62.35



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 2487 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 27

FILE NO. PG5171

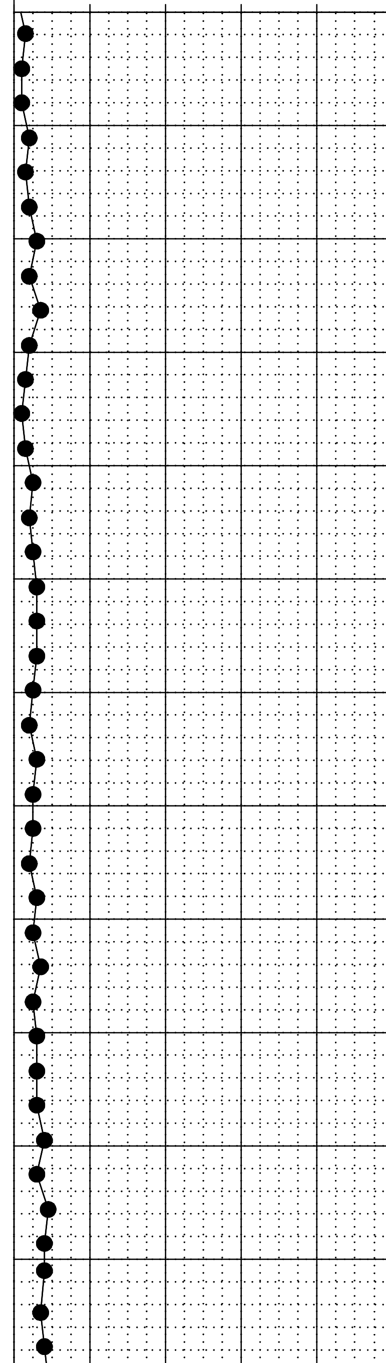
HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						12	62.35
						13	61.35
						14	60.35
						15	59.35
						16	58.35
						17	57.35
						18	56.35
						19	55.35
						20	54.35
						21	53.35
						22	52.35
						23	51.35
						24	50.35

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

○ Water Content %

20 40 60 80



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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

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DATUM TBM - Top spindle of fire hydrant located across from subject site, along Innes Road. Geodetic elevation = 75.739m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 27

FILE NO. PG5171

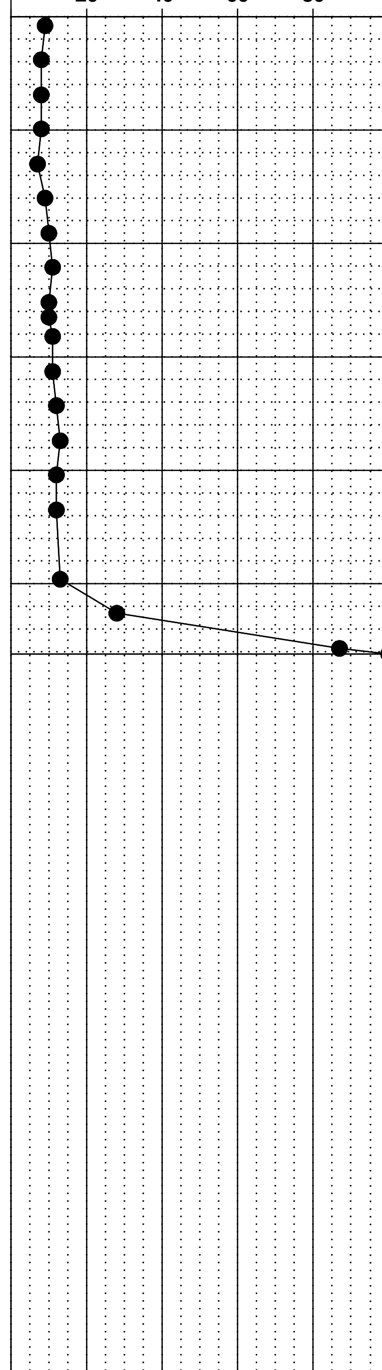
HOLE NO. BH 4

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									
						24	50.35		
						25	49.35		
						26	48.35		
						27	47.35		
						28	46.35		
						29	45.35		
29.62									

End of Borehole

Practical DCPT refusal at 29.62m depth.

(GWL @ 3.22m - Dec. 11, 2019)



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 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

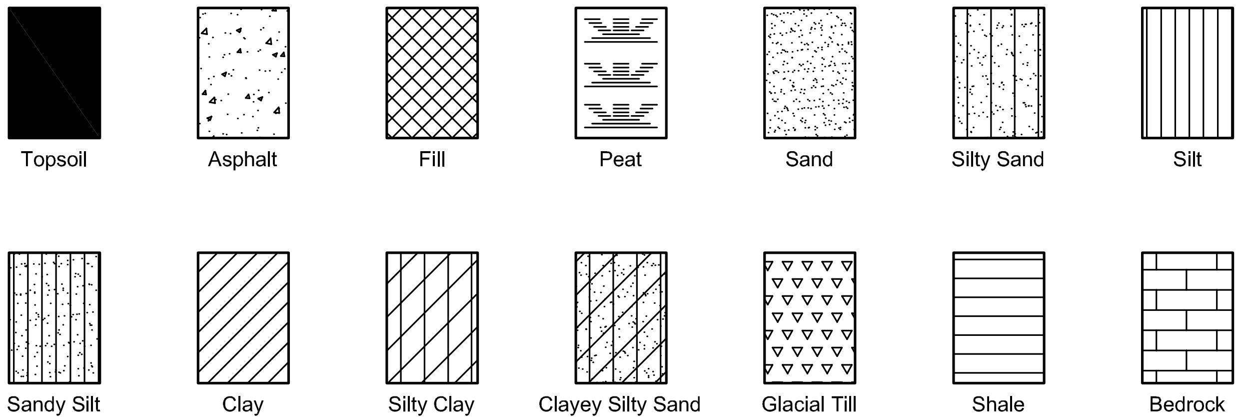
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

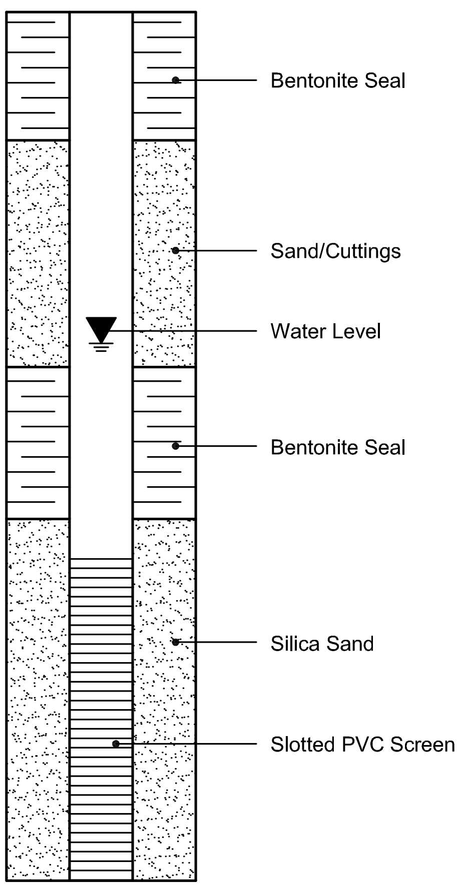
SYMBOLS AND TERMS (continued)

STRATA PLOT

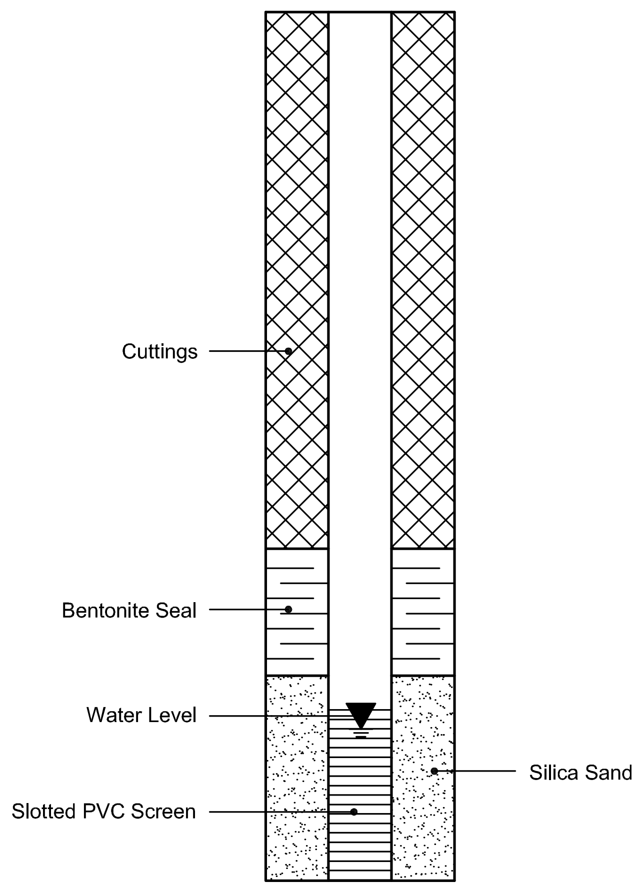


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



	Client ID:	BH4-SS4	-	-	-
	Sample Date:	27-Nov-19 13:00	-	-	-
	Sample ID:	1949469-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.26	-	-	-
Resistivity	0.10 Ohm.m	44.7	-	-	-
Anions					
Chloride	5 ug/g dry	26	-	-	-
Sulphate	5 ug/g dry	57	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5171-1 - TEST HOLE LOCATION PLAN

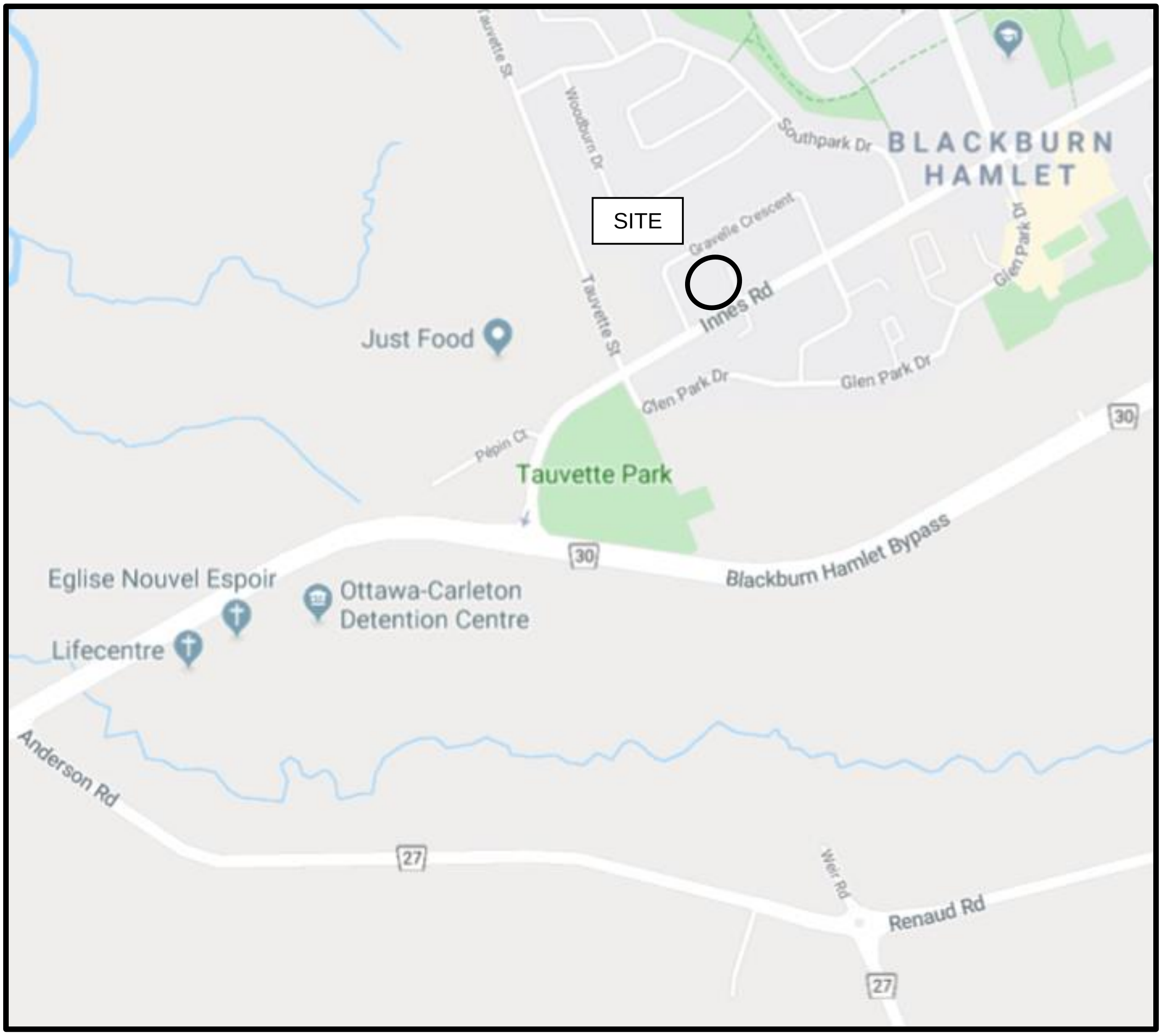


FIGURE 1

KEY PLAN

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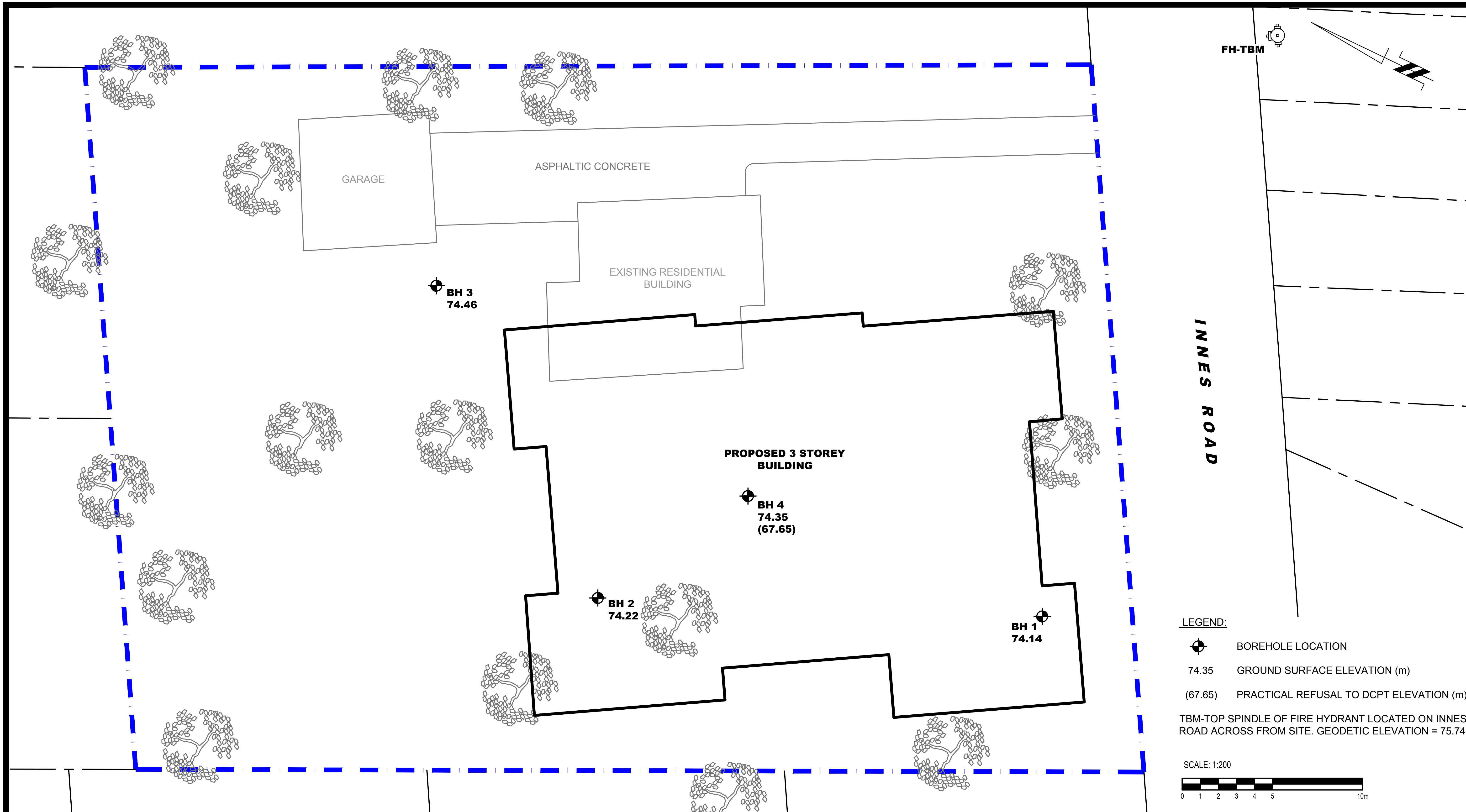

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NO.	REVISIONS	DATE	INITIAL

OTTAWA CARLETON CONSTRUCTION GROUP LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
2487 INNES ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 12/2019
Drawn by: NFRV	Report No.: PG5171-1
Checked by: DP	Dwg. No.: PG5171-1
Approved by: SB	Revision No.:

p:\autocad drawings\geotechnical\pg51xx\pg5171\pg5171-3 test hole location plan.dwg