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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Supplemental Geotechnical Investigation

Proposed Multi-Storey Building
24-30 Pretoria Avenue
Ottawa, Ontario

Prepared For

JB Holdings c/o RLA Architecture

August 2, 2019

Report: PG4798-2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by JB Holdings to complete a supplemental geotechnical investigation for the proposed development to be located at 24-30 Pretoria Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the geotechnical investigation were to:

☐ determine subsurface soil and groundwater conditions by means of boreholes.

☐ provide geotechnical recommendations for 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 for the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

It is understood that the proposed development will consist of a six-storey residential building with 1 level of underground parking level. Associated access lanes, surface parking and landscaped areas are also anticipated as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The initial field program for the geotechnical investigation was carried out on March 7, 2019. At that time, two (2) boreholes were advanced to a maximum depth of 9.5 m below the existing grade. Following the addition of the adjacent property to the proposed development, a supplemental geotechnical investigation was completed on July 11, 2019. At that time, an one (1) additional borehole was advanced to a depth of 6 m below the existing grade.

The borehole locations were distributed in a manner to provide general coverage of the proposed development taking into consideration existing site features and underground utilities. The approximate locations of the boreholes are shown on Drawing PG4798-1 - Test Hole Location Plan included in Appendix 2.

The boreholes for the initial field investigation were completed using a track-mounted auger drill rig operated by a two-person crew. The borehole for the supplemental field investigation was completed using a portable drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson field personnel under the direction of a senior engineer. The drilling procedure consisted of augering/sampling to the required depths at the selected locations, and 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. The portable equipment used for this project uses a 21.17 kg hammer dropped from a height of 760 mm. The N values presented on the Soil Profile and Test Data sheets attached have been corrected for the weight of the hammer.

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. 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 test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located to the east of 24 Pretoria Avenue. A geodetic elevation of 67.19 m was provided for the TBM by Farley, Smith and Dennis Surveying Ltd.

The locations of the boreholes, TBM, and the ground surface elevation at the boreholes, are presented on Drawing PG4798-1 - 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 soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attack against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The site is currently occupied by three low-rise residential buildings, along with associated driveways and landscaped areas. Several mature trees are also located on the subject property. The site is relatively flat and generally at grade with the surrounding properties and Pretoria Avenue.

The site is bordered to the south and west by residential properties with low-rise buildings, to the east by a low-rise commercial building and to the north by Pretoria Avenue.

4.2 Subsurface Profile

Generally, the soil profile encountered at the borehole locations consisted of either a topsoil layer followed by a brown silty sand to a depth of 0.6 m, or an asphalt or brick pavement structure overlying a layer of fill, consisting of silty sand to silty clay, to a depth of 1.5 m. The fill or sand layer was underlain by a deep silty clay deposit of a stiff to very stiff consistency to a depth of 9.5 m below existing grade.

One (1) DCPT was completed at BH1 to a depth of 30 m below existing grade with no practical refusal indicating the depth to bedrock is greater than 30 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, the bedrock in this area consists of shale of the Billings Formation with an overburden thickness ranging between 25 to 50 m.

4.3 Groundwater

The groundwater level (GWL) readings were recorded at the borehole locations on March 14, 2019 and on July 24, 2019 and are presented 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 resulting in higher water levels than noted during the investigation. The long-term groundwater level can also be estimated based on moisture levels and color 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.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It is expected that the proposed building 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.

5.2 Site Grading and Preparation

Stripping Depth

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

Existing foundation walls and other construction debris should be entirely removed from within the perimeter of the proposed building. 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 placed for grading 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. The fill should be placed in a maximum of 300 mm thick loose lifts and compacted with suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to a minimum 98% of the standard proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is a minor concern. These materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to raise the subgrade level for areas to be paved, the material should be compacted in maximum 300 mm thick loose lifts and compacted to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Protective Mud Slab

If a raft slab is required to support the proposed building, it is recommended that a concrete mudslab be used to protect the undisturbed subgrade surface. The excavation bottom will be founded on silty clay which should be protected from disturbance due to worker traffic. A 75 to 100 mm thick lean concrete mud slab is recommended to be poured onto the undisturbed silty clay surface once exposed. The lean concrete should consist of a minimum 15 MPa compressive strength concrete.

5.3 Foundation Design

Spread Footing Foundation

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

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.

The bearing medium under footing-supported structures is required to provide adequate lateral support with respect to the excavations and different founding levels. Adequate lateral support is provided to a silty clay bearing medium above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

The bearing resistance value at SLS for conventional style footings 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, consideration may be given to placing the proposed building on a raft foundation.

For design purposes, the raft foundation base is assumed to be located at approximately a 4 m depth down to an undisturbed native soil surface. The bearing medium will consist of a sensitive brown to grey silty clay which is susceptible to disturbance under construction traffic. The bearing surface should be protected to prevent disturbance as described above.

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 **200 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 **350 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 **200 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed with the above parameters and a potential total and differential settlement of 25 and 20 mm, respectively.

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.

5.4 Design for Earthquakes

The site class for seismic site response is a **Class E** for the foundations considered at this site. Reference should be made to the latest revision of the 2012 Ontario Buildings Code for a full discussion of the earthquake design requirements. The soil underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab and 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 building, 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 - Surface 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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter drainage system is recommended for the proposed building. 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 unless used in conjunction with a composite drainage blanket. 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 storm sewer is also recommended.

6.2 Protection Against Frost Action

Footings, pile caps and grade beams 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.

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. If sufficient room is unavailable due to existing structures or property boundaries, a temporary shoring system may be required.

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.

Support of Adjacent Structures

Founding conditions of adjacent structures bordering the site should be assessed and support requirements should be evaluated prior to the start of excavation. The requirement for foundation support of adjacent structures will depend on the proposed depth of excavation with respect to the lateral separation and depth of the adjacent footings.

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 grey silty clay or below the long term groundwater level, 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.

6.5 Groundwater Control

Permit to Take Water

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the excavations. 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.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) Category 3 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

Based on our observations, the long term groundwater level is expected at a depth ranging from 3.5 to 4.5 m below the existing grade and within the silty clay deposit. Based on this information, the excavation for the proposed building is not anticipated to extend significantly, or at all, below the long-term groundwater level. Therefore, any groundwater lowering is anticipated to be minimal and no impacts to adjacent structures are anticipated due to groundwater lowering.

It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site. If required, the installation of a temporary shoring system will disturb the soil immediately behind the shoring system which may cause some movement of adjacent structures. To lessen these effects, consideration should be given to using a secant wall or sheet piling as the shoring system in specific areas that could support the adjacent structures. Furthermore, until the waterproofing is completed, temporary dewatering will also cause typical minor differential settlements to unsupported structures.

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.

The foundation of adjacent buildings should be protected. The proposed excavation could reduce the existing earth frost cover to an unacceptable thickness. For situations where this could occur, such as shoring, is recommended to reduce frost penetration and potential unexpected settlement.

6.7 Analytical Testing Results

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 results of the chloride content, pH and resistivity indicate the presence of an non aggressive to slightly aggressive environment for exposed ferrous metals at this site.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

☐ Review of the proposed structure(s) and adjacent structures from a geotechnical perspective.

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

☐ Sampling and testing of the concrete and fill materials placed.

☐ 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 construction has been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations 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 JB Holdings or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Colin Belcourt

Colin Belcourt, P. Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Aug. 2, 2019
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 24-30 Pretoria Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located east of 24 Pretoria Avenue. Geodetic elevation of 67.19m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

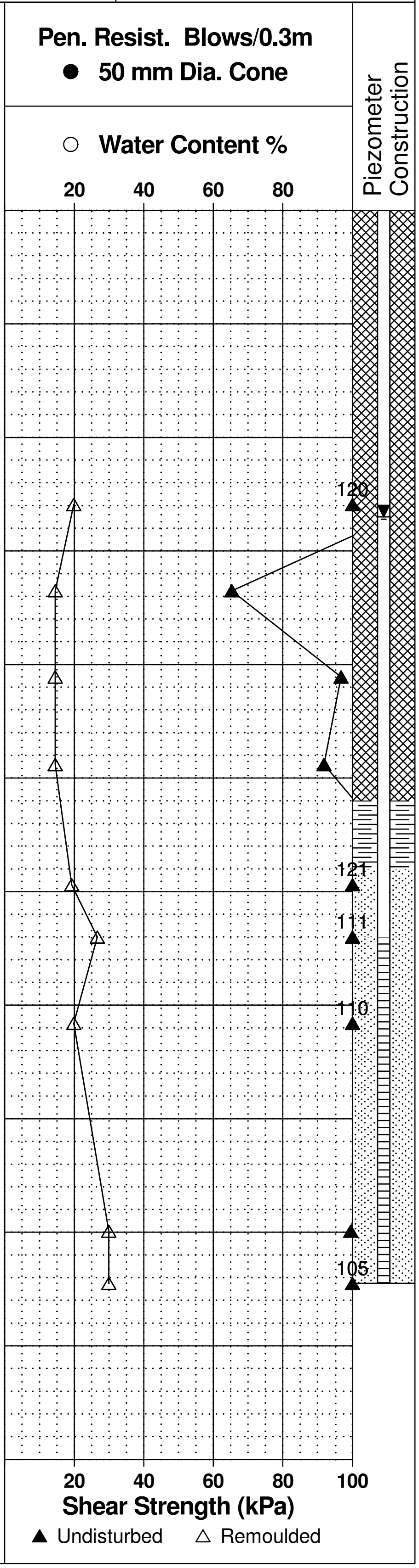
BORINGS BY CME 55 Power Auger

DATE 2019 March 7

FILE NO. PG4798

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.06
FILL: Brown silty sand to silty clay with gravel, trace organics	AU	1				
	SS	2	75	26	1	66.06
1.37						
Very stiff to stiff, brown **SILTY CLAY**	SS	3	96	7	2	65.06
					3	64.06
- stiff to very stiff and grey by 4.3m depth					4	63.06
					5	62.06
					6	61.06
	SS	4	100	P		
					7	60.06
					8	59.06
					9	58.06
9.45						
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 18.0m depth.					10	57.06
					11	56.06



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 24-30 Pretoria Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located east of 24 Pretoria Avenue. Geodetic elevation of 67.19m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 7

FILE NO. PG4798

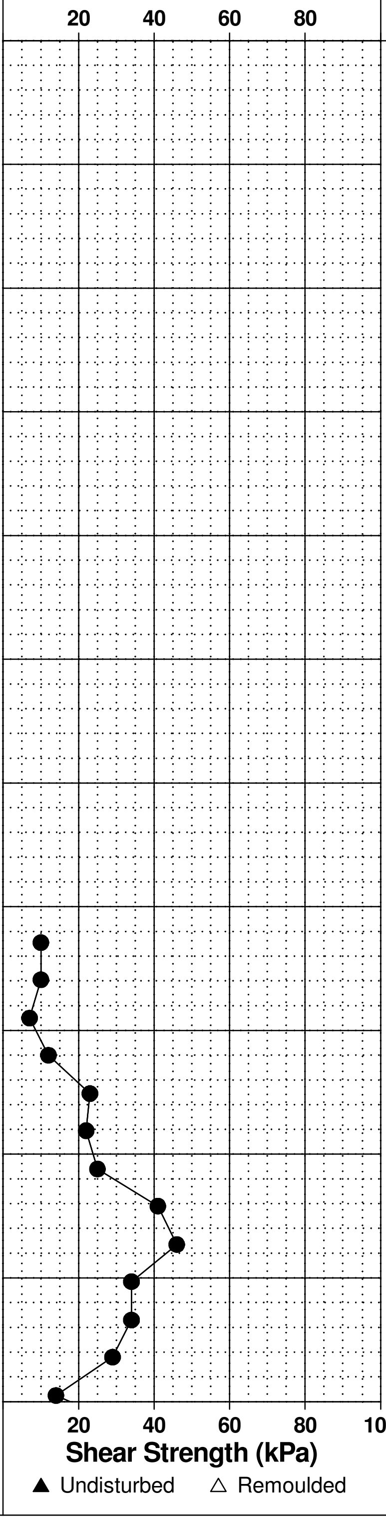
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE							
						11	56.06
						12	55.06
						13	54.06
						14	53.06
						15	52.06
						16	51.06
						17	50.06
						18	49.06
						19	48.06
						20	47.06
						21	46.06
						22	45.06

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

○ Water Content %

Piezometer Construction



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 24-30 Pretoria Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located east of 24 Pretoria Avenue. Geodetic elevation of 67.19m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

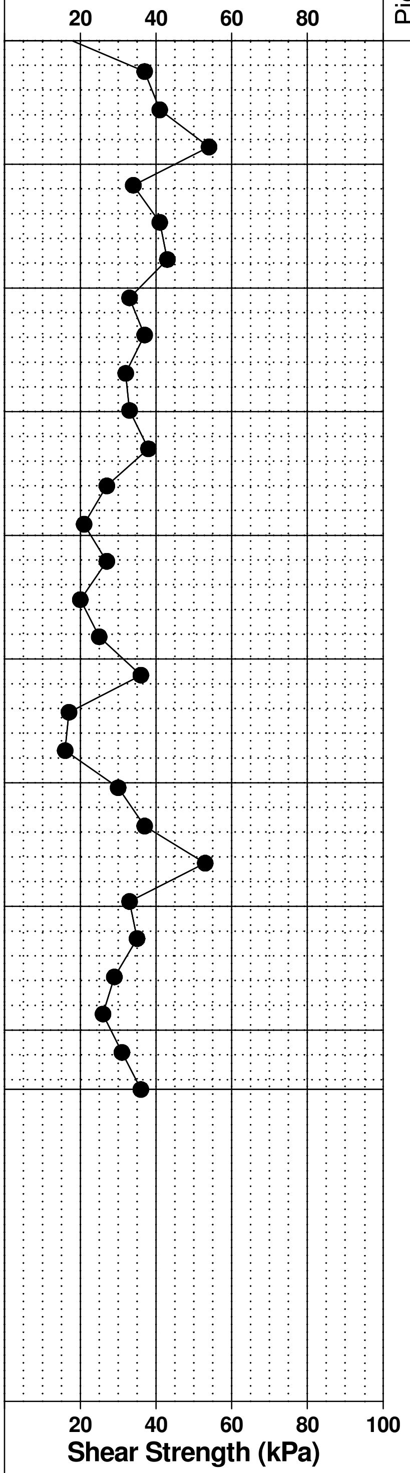
BORINGS BY CME 55 Power Auger

DATE 2019 March 7

FILE NO. PG4798

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						22	45.06		
						23	44.06		
						24	43.06		
						25	42.06		
						26	41.06		
						27	40.06		
						28	39.06		
						29	38.06		
						30	37.06		
30.48									
End of Borehole (GWL @ 2.70m - March 14, 2019)									



▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 24-30 Pretoria Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located east of 24 Pretoria Avenue. Geodetic elevation of 67.19m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 7

FILE NO. PG4798

HOLE NO. BH 2

SOIL DESCRIPTION

GROUND SURFACE

Asphaltic concrete 0.08

FILL: Brown silty sand to silty clay, trace sand and organics 1.52

Very stiff to stiff, brown **SILTY CLAY**

- grey by 3.8m depth

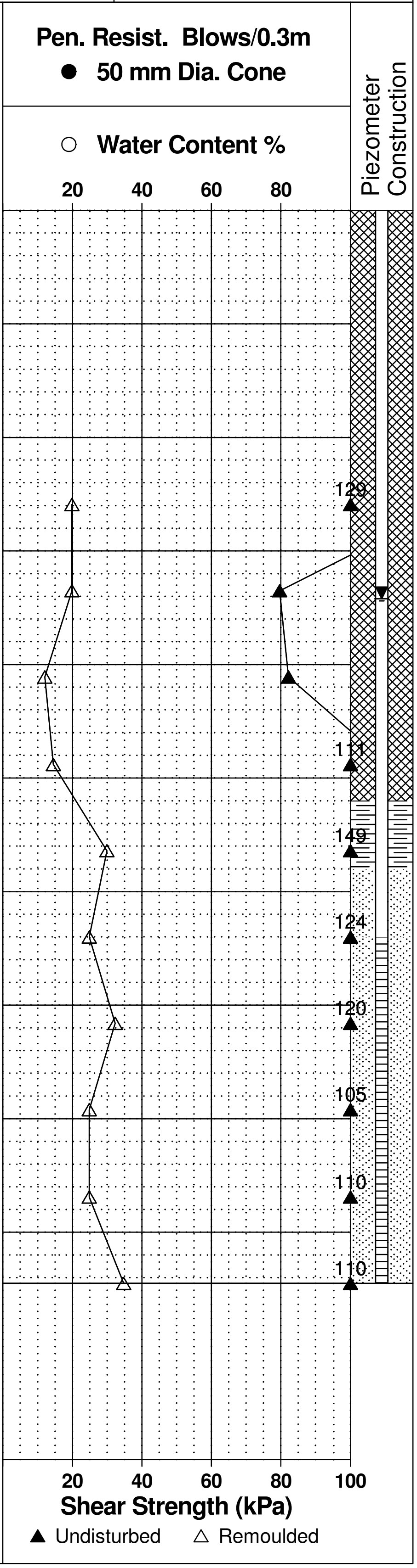
9.45

End of Borehole

(GWL @ 3.42m - March 14, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	62	29
SS	3	100	6

DEPTH (m)	ELEV. (m)
0	66.73
1	65.73
2	64.73
3	63.73
4	62.73
5	61.73
6	60.73
7	59.73
8	58.73
9	57.73



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 24-30 Pretoria Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located east of 24 Pretoria Avenue. Geodetic elevation of 67.19m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

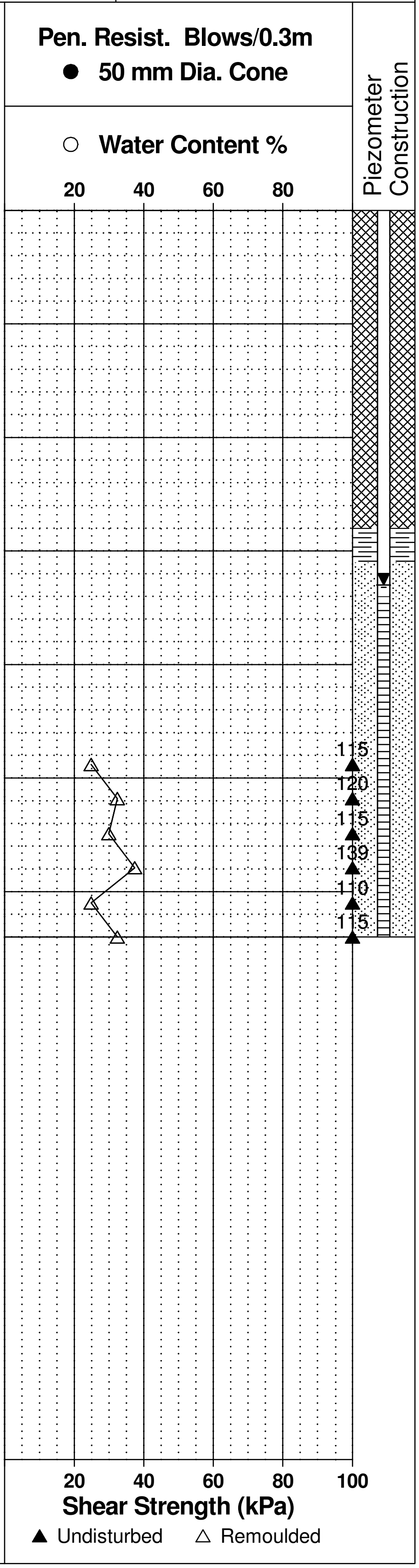
BORINGS BY Portable Drill

DATE 2019 July 11

FILE NO. PG4798

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL	0.18					0	67.53
Loose, brown **SILTY SAND**	0.60	AU	1		13		
		SS	2	100	18	1	66.53
		SS	3	100	17		
		SS	4	100	15	2	65.53
Very stiff, brown **SILTY CLAY** with sand		SS	5	100	8		
		SS	6	100	5	3	64.53
		SS	7	100	3	4	63.53
- grey by 4.3m depth							
						5	62.53
	6.40					6	61.53
End of Borehole (GWL @ 3.30m - July 24, 2019)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

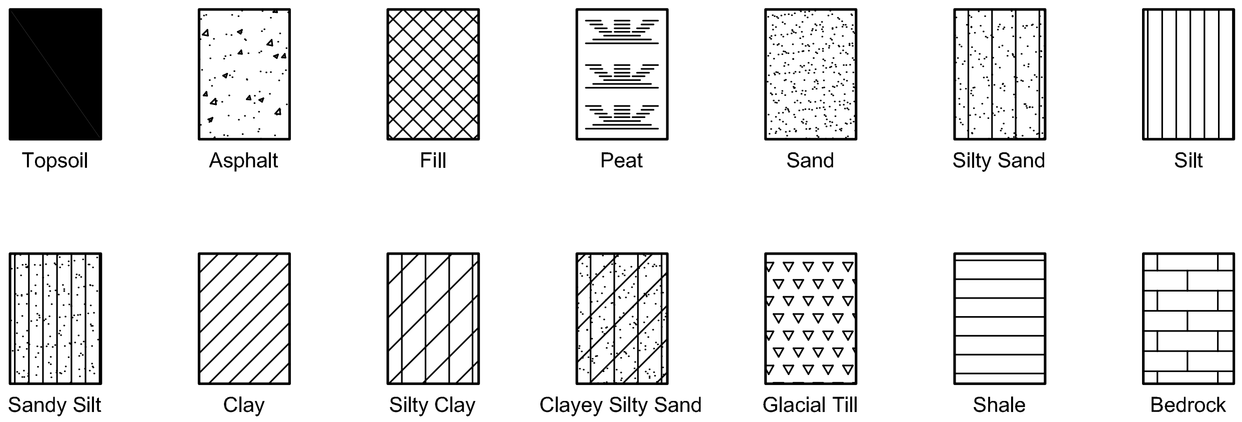
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

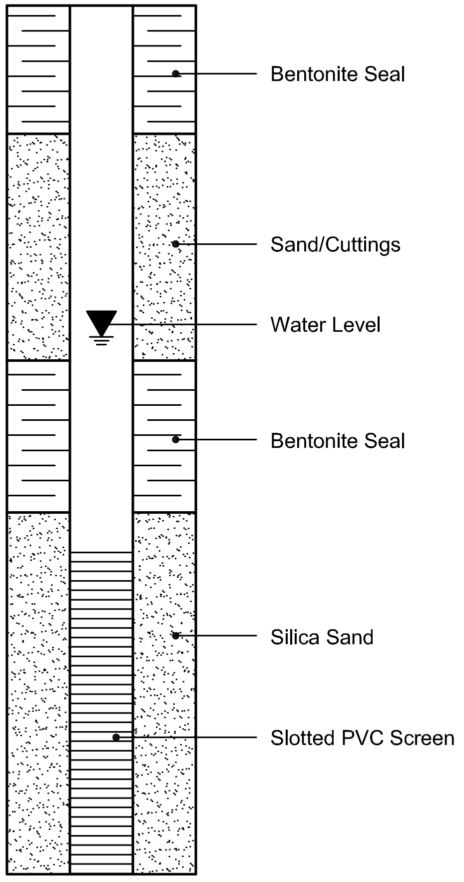
SYMBOLS AND TERMS (continued)

STRATA PLOT

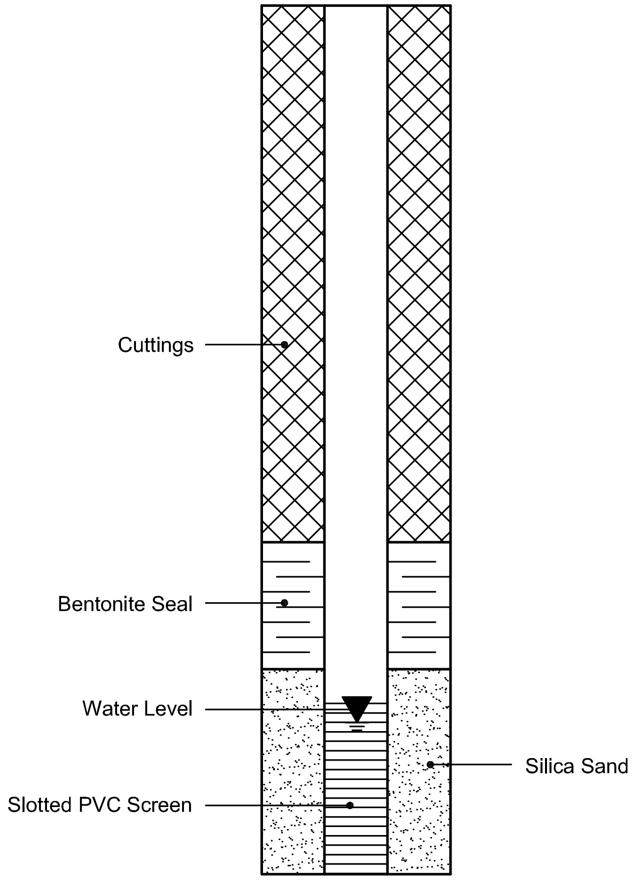


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 27232

Report Date: 18-Jul-2019

Order Date: 12-Jul-2019

Project Description: PG4798

	Client ID:	BH3-SS4	-	-	-
	Sample Date:	11-Jul-19 09:00	-	-	-
	Sample ID:	1928678-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.24	-	-	-
Resistivity	0.10 Ohm.m	109	-	-	-
Anions					
Chloride	5 ug/g dry	12	-	-	-
Sulphate	5 ug/g dry	15	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4798-1 - TEST HOLE LOCATION PLAN

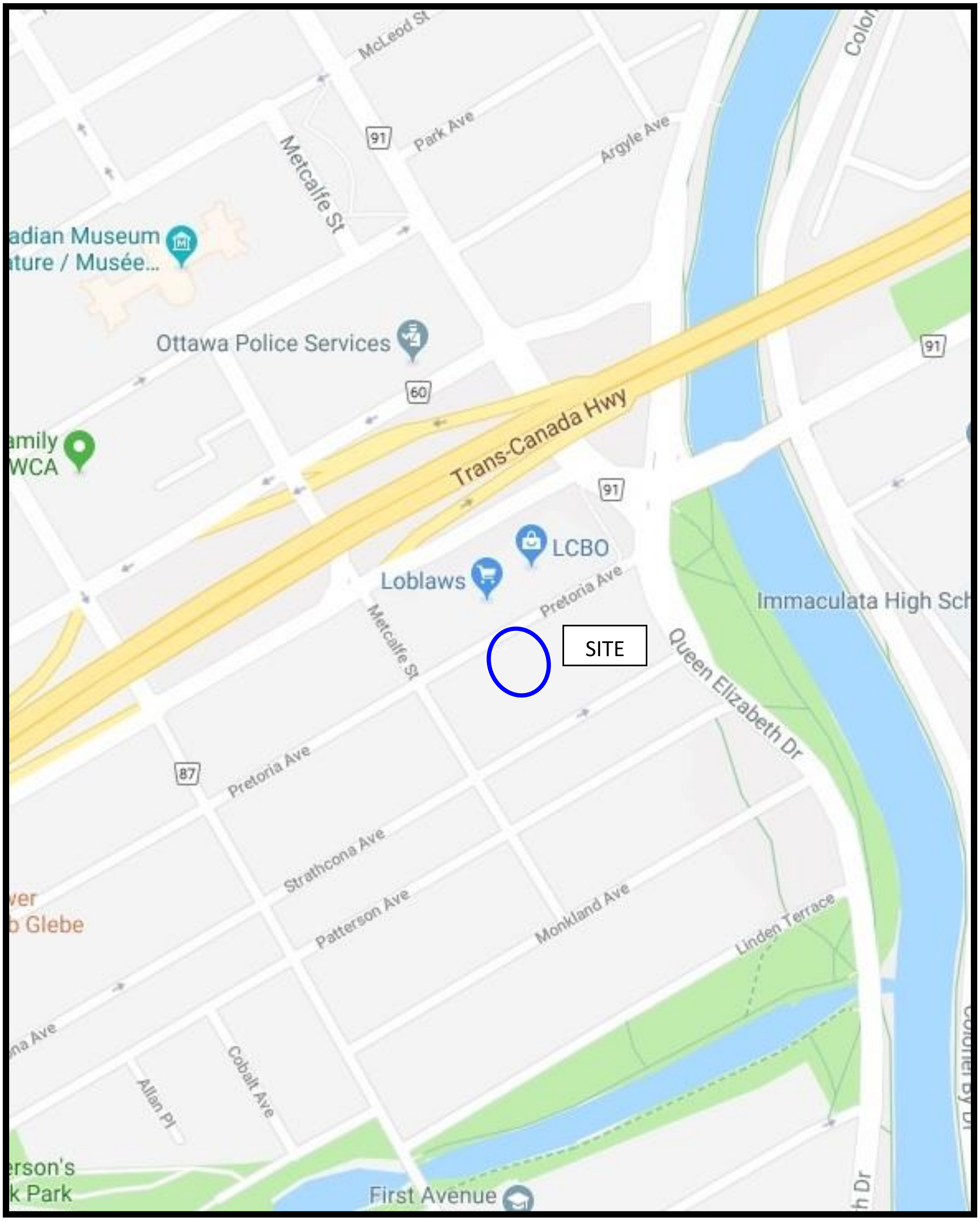


FIGURE 1

KEY PLAN

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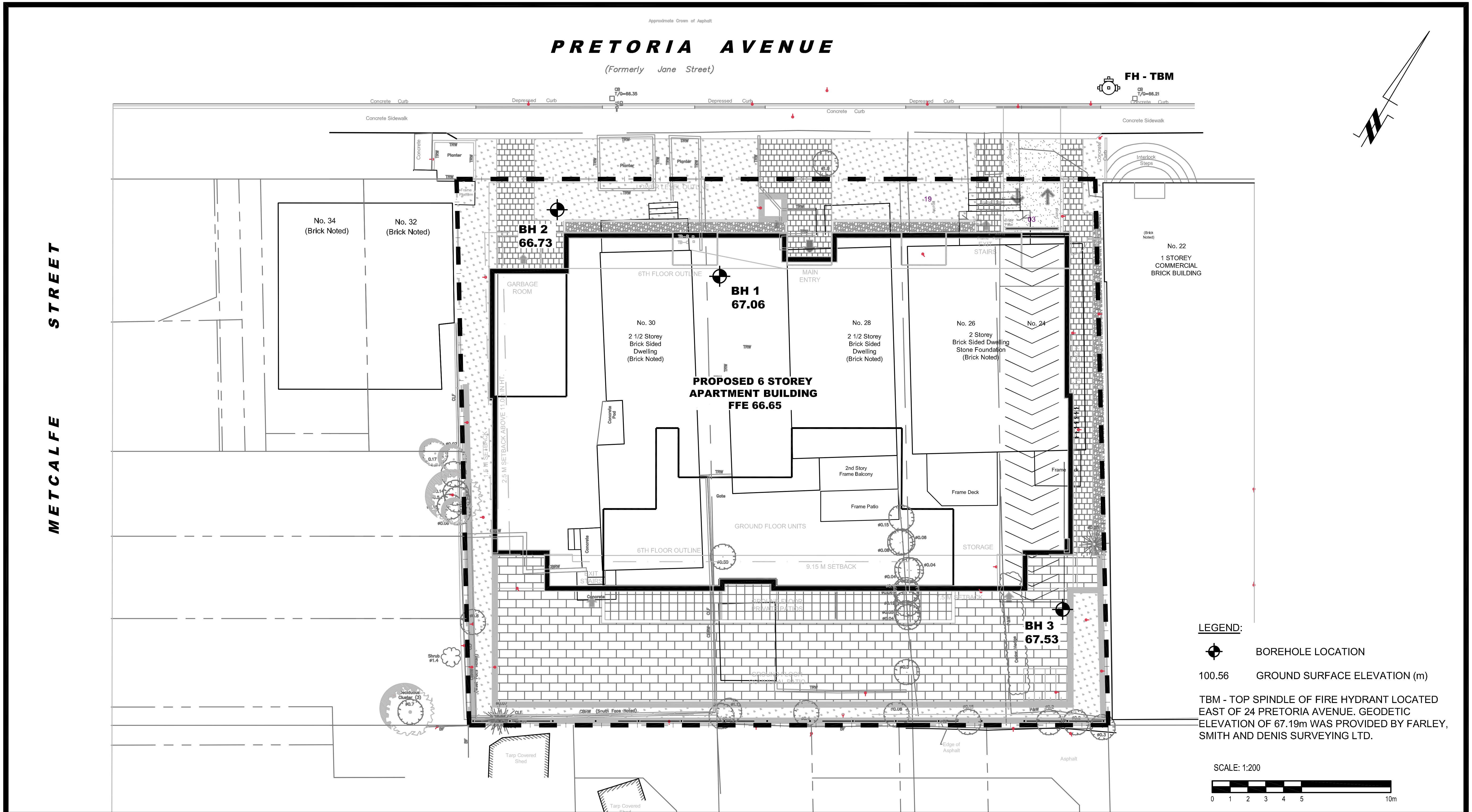

LEGEND:

⌖	BOREHOLE LOCATION
100.56	GROUND SURFACE ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED EAST OF 24 PRETORIA AVENUE. GEODETIC ELEVATION OF 67.19m WAS PROVIDED BY FARLEY, SMITH AND DENIS SURVEYING LTD.

SCALE: 1:200

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NO.	REVISIONS	DATE	INITIAL
1	BASE PLAN UPDATED	17/07/2019	NG

JB HOLDINGS c/o RLA ARCHITECTURE

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 24-30 PRETORIA AVENUE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200
Date: 03/2019
Drawn by: MPG
Report No.: PG4798-2
Checked by: CB
Approved by: DJG

PG4798-1

Revision No.: 1

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