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Geotechnical Investigation

Proposed Multi-Storey Building
16 Hamilton Avenue
Ottawa, Ontario

Prepared For

Surface Developments

October 22, 2018

Report: PG4664-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Surface Developments to conduct a geotechnical investigation for the proposed multi-storey residential building to be located at 16 Hamilton Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under a separate cover.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a 9 storey residential and commercial mixed use building with one partial level of basement. Associated access lanes and landscaped areas are also anticipated. It is expected that proposed site will be fully municipally serviced.

The building will have at grade parking at the rear of the building which is considered to be an open garage with the building above this space.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on June 13, 2018 and August 10, 2018. During that time, 7 boreholes were drilled to a maximum depth of 5.8 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG4664-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, 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, placed in sealed plastic bags, and transported to our laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Subsurface conditions observed in the boreholes 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

A 50 mm diameter groundwater monitoring well was installed at BH 1, BH 2, BH 3, BH 4, and BH 6 to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets presented in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of the report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations and ground surface elevations at the borehole locations were surveyed by Paterson field personnel. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located across the street from 20 Hamilton Avenue. A geodetic elevation of 64.86 m was provided from a previous survey plan of the general area. The location of the TBM, boreholes and the ground surface elevation of the borehole locations are presented on Drawing PG4664-1 - 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.

4.0 Observations

4.1 Surface Conditions

The site is currently occupied by a commercial building and an asphalt covered parking lot. The site is relatively flat and generally at grade with Hamilton Avenue. The site is bordered to the south by a commercial properties, to the east and north by a residential dwellings, and to the west by Hamilton Avenue.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a pavement structure overlying a fill layer consisting of brown silty sand with gravel, organics and traces of construction debris and ash. The fill layer is generally underlain by a layer of stiff, brown silty clay followed by glacial till, consisting of silty sand with clay, gravel, cobbles and boulders.

Practical auger refusal on inferred bedrock was encountered at BH 1 to BH 4 and BH 6 at depths varying between 5 to 5.8 m below existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone with shale of the Bobcaygeon formation and the overburden drift thickness is anticipated between a depth of 2 to 3 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells subsequent to the sampling program and the results are presented in Table 1. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	64.73	2.94	61.79	June 20, 2018
BH 2	65.05	3.35	61.70	June 20, 2018
BH 3	64.62	3.00	61.62	June 20, 2018
BH 4	63.83	3.13	60.70	August 15, 2018

Note: The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located across the street from 20 Hamilton Avenue. A geodetic elevation of 64.86 m was provided for the TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey building. The proposed building is expected to be founded on conventional footings placed on an undisturbed, compact to dense, glacial till bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

All topsoil and deleterious materials, such as those containing organic materials and/or construction debris, should be removed from within the footprint of the proposed building.

Existing foundation walls and other construction debris should be entirely removed from within the proposed 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 proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at SLS of **175 kPa** and a factored bearing resistance value at ULS of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Settlement

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 compact to dense glacial till, above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered granular fill, as described above.

5.4 Design for Earthquakes

The proposed building can be designed using a seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the glacial till 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 prior to placing any fill. The upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

Depending the depth of the basement with respect to the property boundaries and the depth to bedrock, the basement walls will likely retain exterior backfill material.

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.

The majority of the foundation walls are expected be above the long term groundwater level; therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective 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 for all subsurface units below the watertable when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

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. Note that 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 stay at least 0.3 m away from the walls with the compaction equipment.

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. Note that the vertical seismic coefficient is assumed to be zero.

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

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

The earth pressures calculated are unfactored. For the ULS case, the earth pressure 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, if required.

Table 2 - Recommended Flexible Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Flexible Pavement Structure - Heavy Truck Parking Areas and 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 compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 150 mm in 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 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 drainage geocomposite, such as Miradrain G100N or 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 of Footings Against Frost Action

Perimeter footings, of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided.

A minimum of 2.1 m thick soil cover, or equivalent, should be provided for other exterior unheated footings.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The 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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system will consist of a combination of soldier pile and lagging system for open areas such as roadways and parking lots and interlocking steel sheet piling for areas adjacent or in close proximity to existing structures. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 4 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Concrete Underpinning

Based on proximity of existing adjacent building located to the south (22 Hamilton Avenue), support in the form of concrete underpinning may be required during excavation for the proposed building. It is expected that the founding elevations of the existing foundations will be in close proximity to the bedrock surface (less than 1.5 m) and conventional concrete underpinning may be used to support the full width and length of the foundation. The requirement for concrete underpinning can be further evaluated when the drawings and specifications have been finalized.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater infiltration will be directed to the proposed building's sump pit. Once steady state is achieved, it's expected that groundwater flow will be low (less than 10,000 L/day) with higher volumes during peak periods noted after significant precipitation events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that preliminary concepts indicate that a one basement level is planned for the proposed building. Based on the existing groundwater level, the extent of any significant groundwater lowering will not take place for the proposed building. Based on the proximity of neighbouring buildings and the existing groundwater level within the dense glacial till deposit, 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 adverse effects 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 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 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

7.0 Recommendations

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

☐ Review foundation drainage requirements if a temporary shoring system is required.

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

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

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided 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 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 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 Surface Developments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Colin Belcourt

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Carlos P. Da Silva, P.Eng., ing., QP$_{ESA}$

LICENSED PROFESSIONAL ENGINEER
October 22, 2018
C. P. DA SILVA
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 13, 2018

FILE NO. PG4664

HOLE NO. BH 1-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.92
Asphaltic concrete 0.05		AU	1				
FILL: Crushed stone 0.18							
FILL: Brown clayey topsoil, trace construction debris 1.52		SS	2	62	10	1	62.92
Very stiff, brown **SILTY CLAY** 2.29		SS	3	79	13	2	61.92
GLACIAL TILL: Brown silty sand, some clay, with gravel, cobbles, boulders 5.23		SS	4	96	22		
		SS	5	42	13	3	60.92
		SS	6	33	W	4	59.92
		SS	7	67	10	5	58.92

End of Borehole

Practical refusal to augering at 5.23m depth

(GWL @ 3.33m - Sept. 20, 2018)

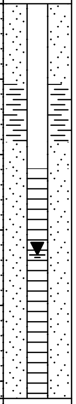
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

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

Monitoring Well Construction



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.
REMARKS
BORINGS BY CME 55 Power Auger
DATE June 13, 2018
FILE NO. PG4664
HOLE NO. BH 2-18

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Asphaltic concrete	0.08	AU	1			0	64.18
FILL: Crushed stone	0.18						
FILL: Brown clayey topsoil, trace ash/cinders	1.52	SS	2	54	6	1	63.18
Stiff, brown **SILTY CLAY**	2.29	SS	3	50	9	2	62.18
		SS	4	71	50+		
GLACIAL TILL: Brown silty sand, some clay, with gravel, cobbles, boulders		SS	5	75	41	3	61.18
		SS	6	54	9	4	60.18
		SS	7	42	28	5	59.18
	5.61	SS	8	36	50+		

End of Borehole

Practical refusal to augering at 5.61m depth

(GWL @3.64m - Sept. 20, 2018)

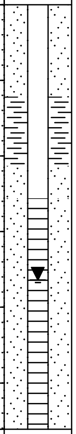
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

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

Monitoring Well Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 13, 2018

FILE NO. PG4664

HOLE NO. BH 3-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.05	AU	1			0	63.81
FILL: Crushed stone 0.15						
FILL: Brown clayey topsoil, trace construction debris 1.52	SS	2	38	11	1	62.81
Stiff, brown **SILTY CLAY** 2.29	SS	3	50	7	2	61.81
GLACIAL TILL: Brown silty sand, some clay, with gravel, cobbles, boulders	SS	4	92	26		
	SS	5	42	10	3	60.81
	SS	6	50	W	4	59.81
					5	58.81
5.74	SS	7	29	50+		

End of Borehole

Practical refusal to augering at 5.74m depth

(GWL @ 3.19m - Sept. 20, 2018)

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

○ Water Content %
20 40 60 80

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

Monitoring Well Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 10, 2018

FILE NO. PG4664

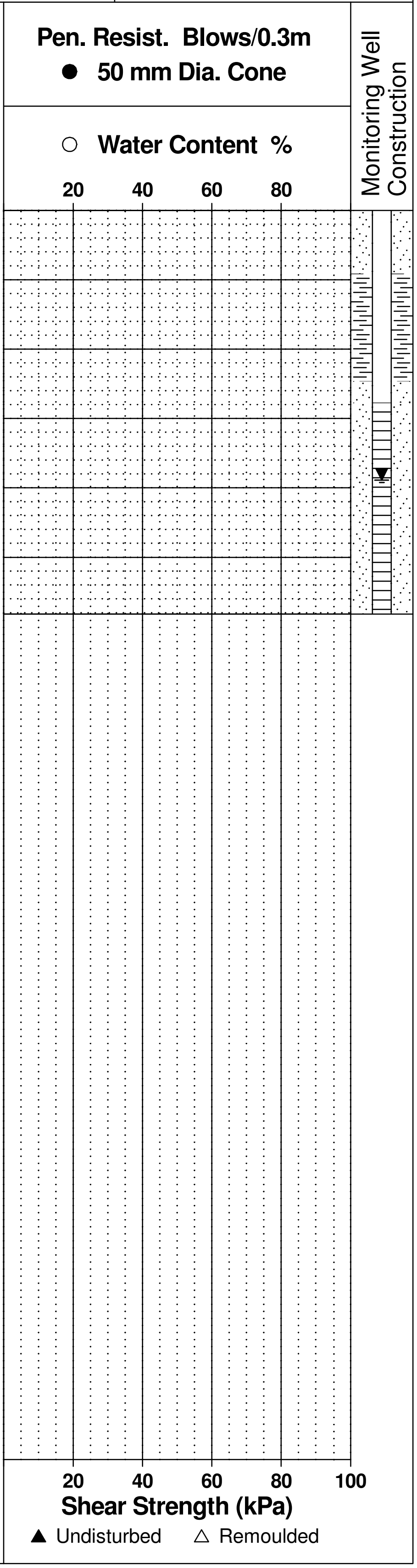
HOLE NO. BH 4-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.83
Asphaltic concrete 0.05	AU	1				
FILL: Brown silty sand, some gravel 1.52	SS	2	46	7	1	62.83
GLACIAL TILL: Brown silty clay with sand and gravel 3.05	SS	3	54	12	2	61.83
	SS	4	79	10		
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders, trace clay 5.82	SS	5	58	63	3	60.83
	SS	6	46	32	4	59.83
	SS	7	54	19	5	58.83
	SS	8	100	50+		

End of Borehole

Practical refusal to augering at 5.82m depth

(GWL @ 3.89m - Sept. 20, 2018)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 10, 2018

FILE NO. PG4664

HOLE NO. BH 5-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.90
Asphaltic concrete 0.05	AU	1				
FILL: Brown silty sand, some gravel, clay, trace organics 1.37	SS	2	54	22	1	62.90
Stiff, brown **SILTY CLAY** 2.29	SS	3	58	9	2	61.90
GLACIAL TILL: Brown to grey silty sand with clay, gravel, cobbles, boulders	SS	4	62	16		
	SS	5	75	14	3	60.90
	SS	6	17	15	4	59.90
5.18	SS	7	67	26	5	58.90
End of Borehole						

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
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DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 10, 2018

FILE NO. PG4664

HOLE NO. BH 6-18

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Gravel, some silty sand	0.15
FILL: Brown silty sand, some gravel	0.91
Very stiff, brown **SILTY CLAY**	1.52
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles, boulders	2.90
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders, some clay	5.00

End of Borehole

Practical refusal to augering at 5.00m depth

(GWL @ 3.14m - Sept. 20, 2018)

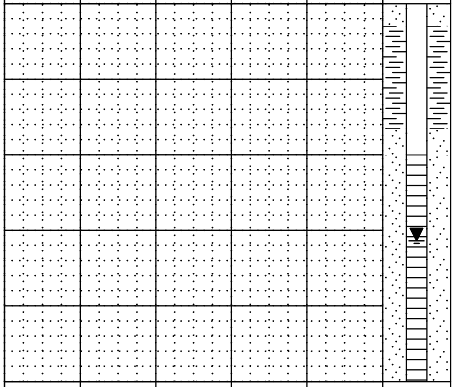
SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	14
SS	3	79	20
SS	4	42	58
SS	5	42	23
SS	6	33	7
SS	7	35	50+

DEPTH (m)	ELEV. (m)
0	63.63
1	62.63
2	61.63
3	60.63
4	59.63
5	58.63

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 10, 2018

FILE NO. PG4664

HOLE NO. BH 7-18

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 %	Monitoring Well Construction
GROUND SURFACE									
Asphaltic concrete 0.05		AU	1			0	63.74		
FILL: Brown silty sand, some gravel, trace organics 1.52		SS	2	67	11	1	62.74		
Very stiff, brown **SILTY CLAY** 2.29		SS	3	67	9	2	61.74		
		SS	4	33	50+				
GLACIAL TILL: Grey silty sand, some gravel, cobbles, boulders, trace clay 4.42		SS	5	83	42	3	60.74		
		SS	6	71	21	4	59.74		
End of Borehole									

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
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.
REMARKS

FILE NO. PG4664

HOLE NO. BH 8-18

BORINGS BY Portable Drill

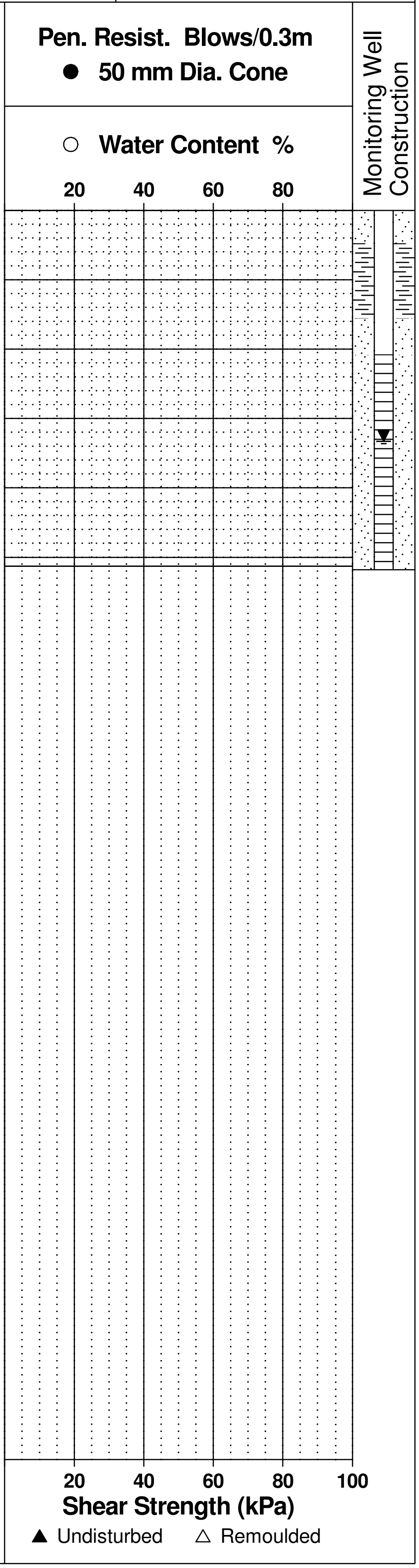
DATE September 7, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Concrete Slab 0.13	G	1			0	63.96
FILL: Brown silty sand with gravel, trace slag concrete 0.91	SS	2	75			
	SS	3	100		1	62.96
Grey **SILTY CLAY** trace gravel	SS	4	36			
					2	61.96
2.44	SS	5	29			
GLACIAL TILL: Dense, grey silty sand with clay, gravel, cobbles and boulders	SS	6	89			
					3	60.96
	SS	7	31			
	SS	8	96		4	59.96
	SS	9	68			
5.13					5	58.96

End of Borehole

Practical refusal to augering at 5.13m depth

(GWL @ 3.33m - Sept. 20, 2018)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 16 and 20 Hamilton Ave. N.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the southwest corner of 366 Parkdale Avenue. Geodetic elevation = 64.86m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY Portable Drill

DATE September 7, 2018

FILE NO. PG4664

HOLE NO. BH 9-18

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.96
Concrete Slab	0.28	G	1				
FILL: Brown silty sand with sand	0.91	SS	2	38			
Brown **SILTY CLAY** trace sand		SS	3	96		1	62.96
		SS	4	96		2	61.96
	3.05	SS	5	100		3	60.96
GLACIAL TILL: Dense, grey silty sand with gravel, some cobbles		SS	6	33			
		SS	7	92		4	59.96
	5.03	SS	8	96		5	58.96

End of Borehole

Practical refusal to augeringl at 5.03m depth

(GWL @ 2.67m - Sept. 20, 2018)

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

○ **Water Content %**

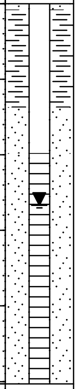
20 40 60 80

Shear Strength (kPa)

20 40 60 80 100

▲ Undisturbed △ Remoulded

Monitoring Well Construction



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

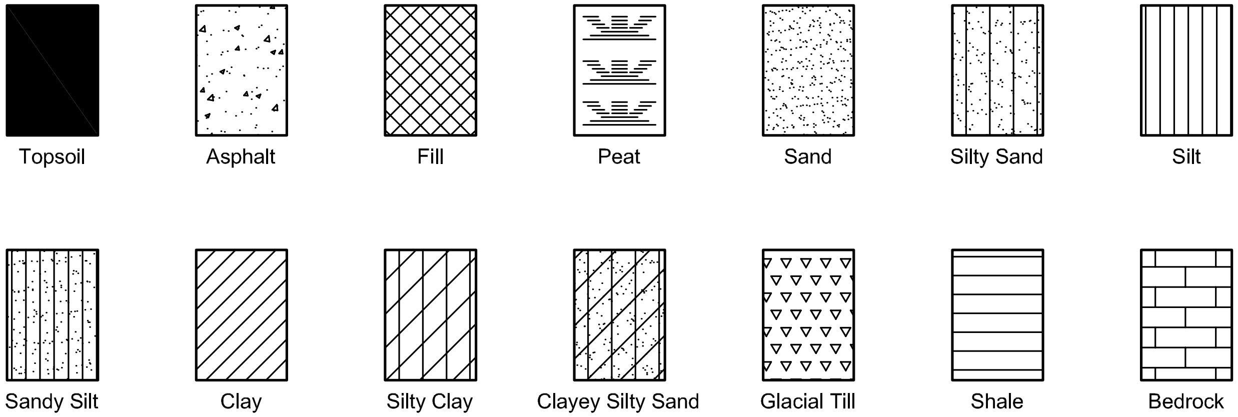
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

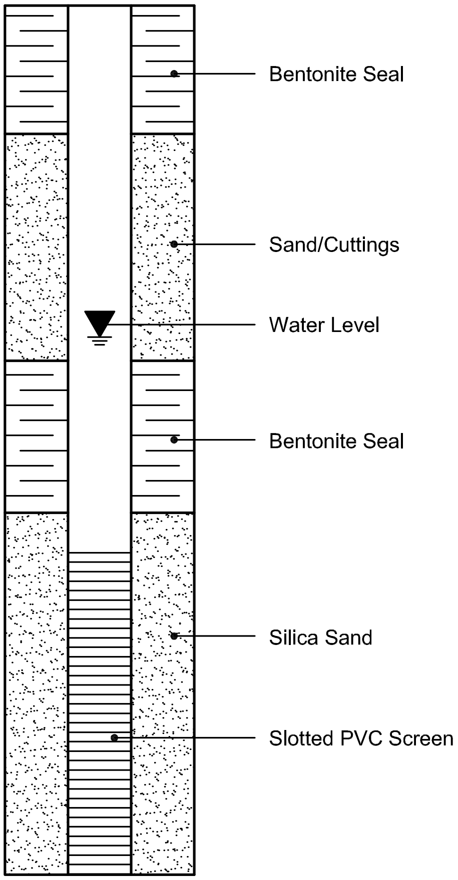
SYMBOLS AND TERMS (continued)

STRATA PLOT

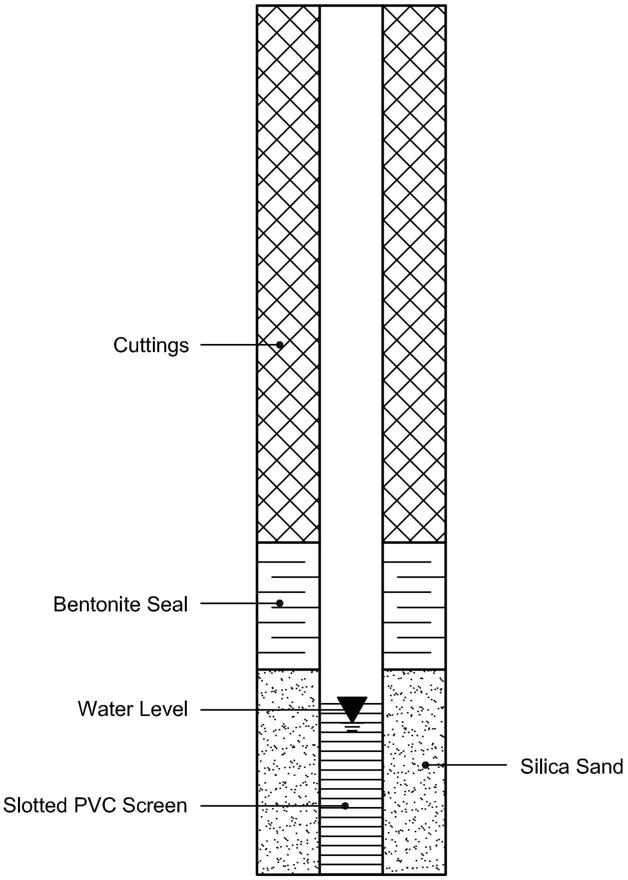


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4664-1 - TEST HOLE LOCATION PLAN

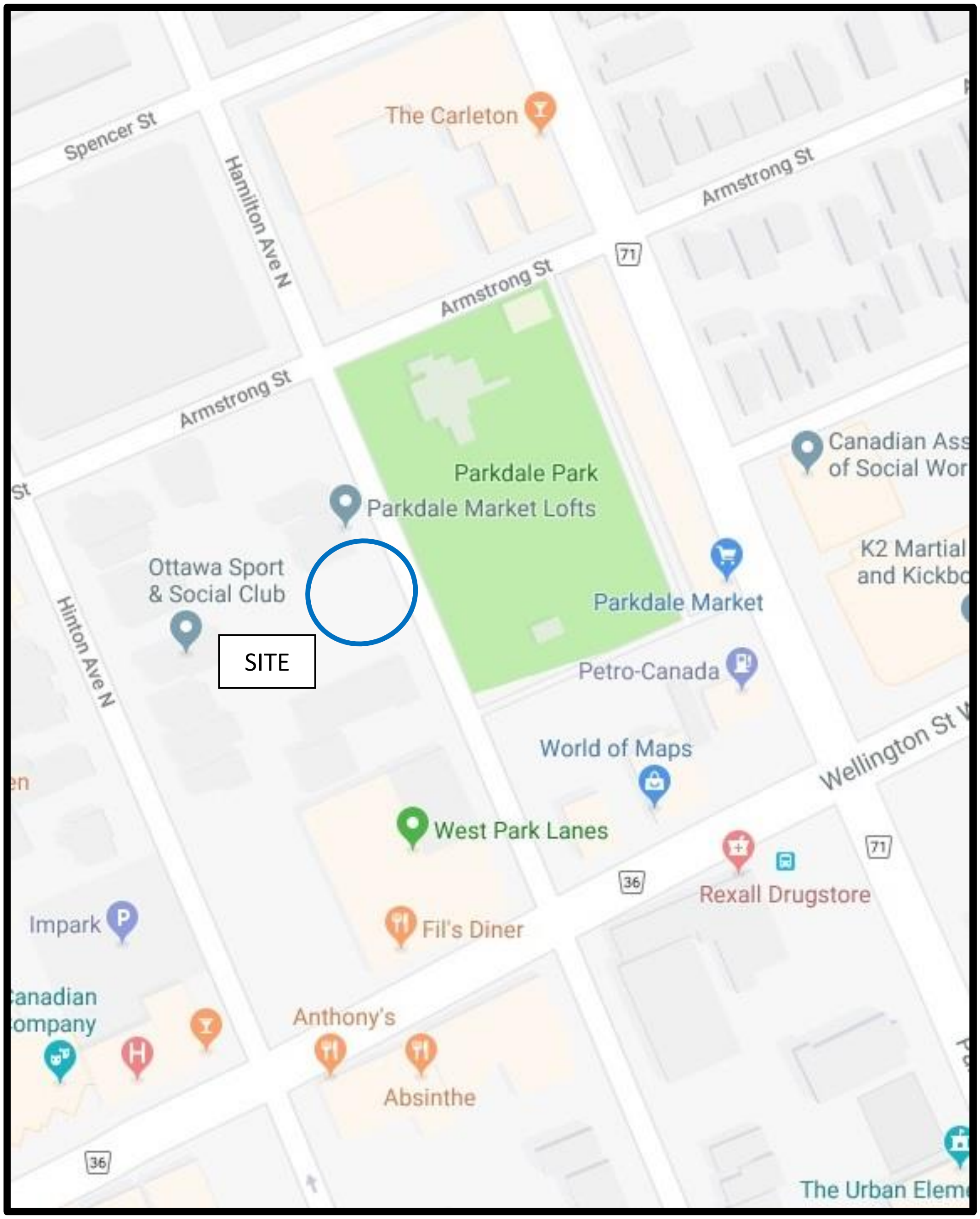


FIGURE 1

KEY PLAN

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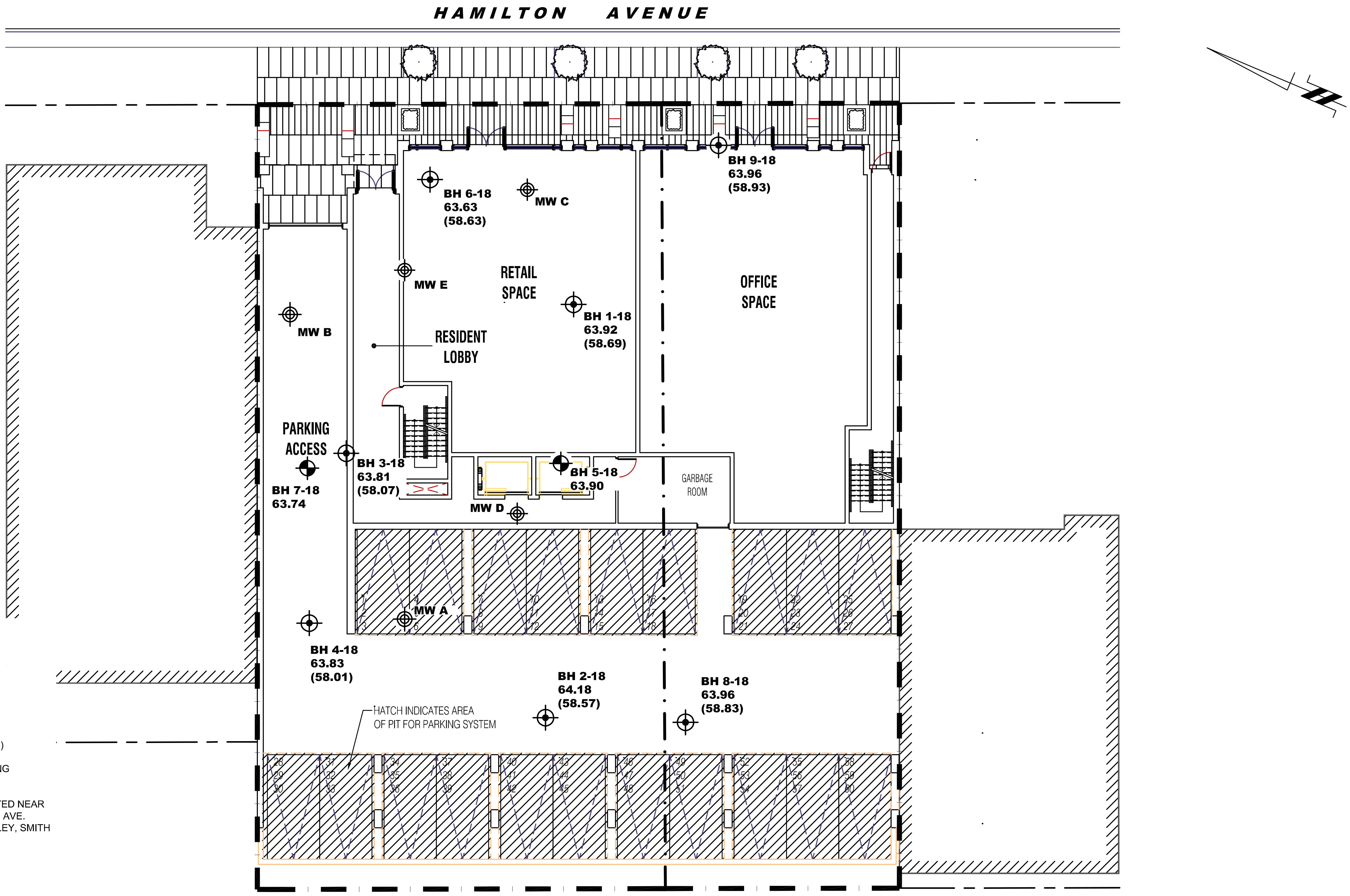

LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION
⌖	MONITORING WELL LOCATION BY OTHERS
63.83	GROUND SURFACE ELEVATION (m)
(58.01)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED NEAR THE SOUTHWEST CORNER OF 366 PARKDALE AVE. GEODETIC ELEVATION = 64.86m, AS PER FARLEY, SMITH AND DENNIS SURVEYING LTD.

SCALE: 1:200

0 1 2 3 4 5 10m

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NO.	REVISIONS	DATE	INITIAL
0			

SURFACE DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 16-20 HAMILTON AVENUE NORTH
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200
Drawn by: RCG
Checked by: CB
Approved by: DJG
Date: 10/2018
Report No.: PG4664-1
Dwg. No.: PG4664-1
Revision No.: 0

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