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# Geotechnical Investigation

Proposed Multi-Storey Building
1950 Scott Street
Ottawa, Ontario

Prepared For

EBC Inc.

**Paterson Group Inc.**
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

March 29, 2018

Report PG4394-1

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## Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by EBC Inc. to conduct a geotechnical investigation for the proposed multi-storey building which is to be located at 1950 Scott Street, 312 Clifton Road, and 314 Clifton Road 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.
- ❑ To provide geotechnical recommendations for 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.

Paterson has completed an environmental site assessment for the subject site and the findings and recommendations are presented under separate cover.

# 2.0 Proposed Development

It is understood that the proposed development will consist of a 20 storey building with two to three levels of underground parking with the garage footprint occupying the majority of the subject site. Associated paved access lanes and landscaped areas are also anticipated around the proposed building.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the investigation was carried out on March 6, 2018. A total of 3 boreholes were advanced to a maximum depth of 10.2 m. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The approximate locations of the boreholes are shown on Drawing PG4394 -1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger 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. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

### Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented 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.

Rock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to our laboratory. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data sheets. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The 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.

**Monitoring Well Installation**

Groundwater monitoring wells were installed in the three (3) boreholes upon completion of the sampling program. Typical monitoring well construction details are described below:

❏ 3 m of slotted 32 mm diameter PVC screen at the base of each borehole.
❏ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
❏ No.3 silica sand backfill within annular space around screen.
❏ 300 mm thick bentonite hole plug directly above PVC slotted screen.
❏ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

**Sample Storage**

All 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 borehole locations were selected, determined in the field and surveyed by Paterson. The location of the boreholes and the ground surface elevations at the boreholes are presented on Drawing PG4394-1 - Test Hole Location Plan in Appendix 2.

The ground surface elevation at each borehole location was referenced to the top spindle of the fire hydrant located in front of 320 McRae Avenue which has a geodetic elevation of 64.44 m.

## 3.3 Laboratory Testing

Soil and rock 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 analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

# 4.0 Observations

## 4.1 Surface Conditions

The site consists of three properties (1950 Scott Street, 312 Clifton Road, and 314 Clifton Road) and is bordered by a commercial property to the west, residential property to the south, Clifton Road to the east, and Scott Street to the north. The site is currently occupied by a 2-storey commercial building (1950 Scott Street) and residential dwellings (312 and 314 Clifton Road). The remainder of the site is occupied by asphalt paved access lanes and parking areas with landscaped margins.

## 4.2 Subsurface Profile

### Overburden

The subsurface profile encountered in the boreholes consists of asphalt and fill overlying a glacial till deposit at depths of up to 0.4 m below the existing ground surface. The glacial till was generally observed to consist of a compact to very dense, brown silty sand with some gravel, cobbles, and boulders. Bedrock was encountered underlying the glacial till at approximate depths of 1.5 to 2.1 m. Specific details of the soil profile at each test hole location can be seen on the Soil Profile and Test Data sheets in Appendix 1.

### Bedrock

Bedrock, consisting of limestone with shale seams, was cored at each borehole location to a maximum depth of 10.2 m. The rock quality designation (RQD) values ranged from 40 to 100%, and generally increased with depth. These results indicate that the quality of the bedrock ranges from poor to excellent.

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and dolostone of the Gull River formation with a drift thickness of 1 to 3 m.

## 4.3 Groundwater

The groundwater level readings were recorded at the borehole locations on March 16, 2018 and are presented in Table 1 below and on the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

**Table 1**
**Summary of Groundwater Level Readings**

| Borehole Number | Ground Elevation (m) | Groundwater Levels (m) | | Recording Date |
|---|---|---|---|---|
| | | Depth | Elevation | |
| BH 1 | 64.34 | 7.79 | 56.55 | March 19, 2018 |
| BH 2 | 64.79 | 6.05 | 58.74 | March 19, 2018 |
| BH 3 | 64.23 | 5.94 | 58.29 | March 19, 2018 |

**Note:** 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 in front of 320 McRae Avenue, which has a geodetic elevation of 64.44 m.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated that the proposed multi-storey building will be founded on shallow footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground parking levels. Hoe ramming could be used where only small quantities of bedrock need to be removed. Line drilling and controlled blasting could be used where large quantities of bedrock need to be removed. The blasting operations should be planned and carried out under the guidance of a professional engineer with experience in blasting operations.

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

## 5.2 Site Grading and Preparation

**Stripping Depth**

Due to the relatively shallow depth of the bedrock at the subject site and the anticipated founding level for the proposed building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed building. As noted above, bedrock excavation will also be required for the construction of the underground parking levels.

**Bedrock Removal**

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on existing utilities, buildings and other structures in the vicinity of the site should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

**Vibration Considerations**

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Construction of a shoring system, such as soldier piles, would require the use of this equipment. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

## 5.3 Foundation Design

**Bearing Resistance Values**

Footings placed on a clean, surface sounded bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **4,000 kPa** could be used if founded on bedrock which is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the major footing footprint(s). The deeper elevator pit can also be used to assess the quality of the bedrock below the proposed founding elevation for footings. The drill hole inspection should be completed by the geotechnical consultant.

**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 sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passing through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or shallower).

**Settlement**

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

## 5.4 Design for Earthquakes

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

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

## 5.5 Basement Slab

For the subject site development, all overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the basement floor slab. It is recommended that the upper 200 mm of sub-slab fill consists 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 at least 95% of its standard Proctor maximum dry density.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor.

## 5.6 Basement Wall

There are several combinations of backfill and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions could be well-represented by assuming the retained soil consists of a material with a coefficient of at-rest earth pressure ($K_o$) of 0.5 and a dry unit weight of 20 kN/m$^3$ (effective unit weight of 13 kN/m$^3$). A large portion of the basement walls are expected to be poured against a composite drainage blanket which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure ($K_o$) of 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m$^3$ (effective unit weight of 15.5 kN/m$^3$) for this condition.

A seismic earth pressure component will not be applicable for the foundation wall which is to be poured against the bedrock face. In this case, the seismic earth pressure is expected to be transferred to the underground floor slabs, which should be designed to accommodate the pressures.

Undrained conditions are anticipated below the groundwater level. Therefore, the applicable effective unit weight of the retained soil or bedrock should be utilized below the groundwater level. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Two distinct conditions, static and seismic, should be reviewed for the soil profile above the bedrock. The parameters for design calculations for the two conditions are presented below.

### Static Conditions

The static horizontal earth pressure ($p_o$) could be calculated with 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 or bedrock
$\gamma$ = unit weight of fill of the retained soil or bedrock ($kN/m^3$)
H = height of the wall (m)

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

### Seismic Conditions

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

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

$a_c$ = $(1.45 - a_{max}/g)a_{max}$
$\gamma$ = unit weight 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 the Ontario Building Code (OBC) 2012. The vertical seismic coefficient is assumed to be zero.

The earth force component ($P_o$) under seismic conditions could be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$.

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 Rock Anchor Design

It is recommended that foundation uplift resistance, should it be required, be provided using rock anchors. Design recommendations for rock anchors are provided in the following subsections.

**Overview of Anchor Features**

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

Anchors can be of the “passive” or the “prestressed” type, depending on whether the anchor tendon is provided with prestress load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the prestressed type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed tower, it is recommended that the rock anchors for this project be provided with double corrosion protection.

### Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength of the weaker of the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the unconfined compressive strength of limestone ranges between about 30 and 80 MPa, which is stronger than most routine grouts. It is recommended that an allowable grout to rock bond stress of **1,000 kPa** be used, for Working Stress Design (WSD), for grouted anchor design at this site.

For Limit States Design (LSD), a factored tensile grout to rock bond resistance value at ULS of **1.2 MPa**, incorporating a resistance factor of 0.4, can be used.

### Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Typical anchor capacities for simple grouted deformed rebar anchors are provided in following table.

### Grout Strength Required

The allowable grout to steel bond strength for deformed reinforcing steel bars, in grouted anchors is 10% of the cylinder compressive strength, f'c, of the grout, for WSD. This bond is developed over the perimeter of the bar for the length of the grouted anchor. A minimum grout strength of 30 MPa is recommended.

**Table 2 - Typical Grouted Rock Anchor Capacities (WSD)**

| Anchor Bar Size | Drill Hole Diameter (mm) | Anchor Lengths (m) | | | Anchor Groups and Capacity (kN) | | |
|---|---|---|---|---|---|---|---|
| | | Free | Fixed | Total | Number and Spacing | Allowable Capacity | Factored Tensile Resistance |
| 25M | 50 | 4.15 | 0.8 | 4.95 | 4 at 1 m | 120 | 145 |
| | | 3.75 | 0.8 | 4.55 | 4 at 1.8 m | 120 | 145 |
| 35M | 75 | 3.65 | 1.05 | 4.7 | 2 at 1.8 m | 240 | 290 |
| | | 5.35 | 1.05 | 6.4 | 4 at 1 m | 240 | 290 |
| | | 4.95 | 1.05 | 6 | 4 at 1.8 m | 240 | 290 |
| | | 6.25 | 1.05 | 7.3 | 6 at 1 m | 240 | 290 |
| | | 5.85 | 1.05 | 6.9 | 6 at 1.8 m | 240 | 290 |

**Notes:**
1. Anchor capacities (per anchor) are estimated to provide full allowable tensile resistance (see note 2) for the anchor groups (number and spacing) noted.
2. Allowable capacities use 60% Fy for Grade 400 steel (per anchor).
3. Factored tensile resistance values incorporate a resistance factor of 0.4.
4. Multiply the anchor capacity by the number of anchors in the group to get the group capacity.
5. A minimum grout strength of 30 MPa is recommended.

## Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of grout tube to place grout from the bottom up in the anchor holes is recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request.

## 5.8 Pavement Structure

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

**Table 3 - Recommended Pavement Structure - Car Only Parking at Lower Parking Level**

| Thickness (mm) | Material Description |
|---|---|
| 125 | **Wear Course** - Concrete |
| 300 | **BASE** - OPSS Granular A Crushed Stone |
| **SUBGRADE** - Either OPSS Granular B Type I or Type II material placed over the bedrock | |

**Table 4 - Recommended Pavement Structure - Access Lanes and Ramp**

| Thickness (mm) | Material Description |
|---|---|
| 40 | **Wear Course -** HL3 or Superpave 12.5 Asphaltic Concrete |
| 50 | **Binder Course -** HL8 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 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 to a competent layer and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

### Foundation Drainage

It is understood that the building footprint will occupy the entire boundary of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a drainage system placed on the bedrock excavated face and shoring system.

Since the lower basement levels will be located below the expected groundwater level, consideration will be given to installing a waterproofing membrane or coating for the vertical surfaces from the bottom of the excavation (bedrock vertical face) up to 1 m above the long term groundwater level (approximately 9 m below the existing finished grade). The bedrock vertical surface will require bedrock grinding to create a smoother bedrock surface and lessen the potential of bedrock over breakage. By waterproofing the vertical excavation sides, it will be possible to lessen the groundwater volumes entering the excavation. A composite drainage system should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water breaching the waterproofing membrane system.

For preliminary design purposes, the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) should extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

### Foundation Backfill

For the upper portion of the foundation, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

### Underfloor Drainage

Underfloor drainage will be required to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, we recommend that 150 mm in diameter perforated pipes be placed along the interior perimeter of the foundation wall and one drainage line within each bay. The spacing of the underfloor drainage system should be confirmed at the time of backfilling the floor completing the excavation when water infiltration can be better assessed.

### Adverse Effects of Dewatering on Adjacent Properties

Since bedrock is located at a very shallow depth (1.5 to 2 m below the existing grade), the proposed development will be founded within the bedrock and any long term dewatering of the site will have have no adverse effects to the surrounding buildings or structures which are also founded on the bedrock. The short term dewatering during the excavation program will be managed by the excavation contractor and will be part of the PTTW requirements.

## 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 in this regard.

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

## 6.3 Excavation Side Slopes

### Unsupported Excavations

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.

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 Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered. A minimum 1 m horizontal bench should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or a stable base for the overburden shoring system.

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.

**Temporary Shoring**

Temporary shoring may be required to support the overburden soils. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is 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 shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

For preliminary design purposes, the temporary shoring may consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

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

| Table 5 - 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 |
| Unit Weight ($\gamma$), kN/m$^3$ | 21 |
| Submerged Unit Weight ($\gamma$), kN/m$^3$ | 13 |

### Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K $\gamma$ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K $\gamma$ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

If a soldier pile and lagging system is used, the piles could be socketed into the bedrock in pre-augered holes. The augered holes should be advanced at least 2 m into the bedrock and at least 2 m below the bottom of the excavation. A minimum factor of safety of 1.5 should be used.

## 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.

A minimum of 300 mm of OPSS Granular A should be placed for bedding of sewer or water pipes when placed on a bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the pipe obvert 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 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 reduce the potential 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 SPMDD.

## 6.5 Groundwater Control

**Groundwater Control for Building Construction**

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium. Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) 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 Category application package and issuance of the permit by the MOECC.

For typical ground or surface water volumes being pumped during the construction phase, typically between 25,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

**Long-term Groundwater Control**

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or subfloor drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e. less than 25,000 L/day) with peak periods noted after rain 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 2 to 3 levels of underground parking are planned for the proposed building with the lower portion of the foundation having a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place against the bedrock face, long-term groundwater lowering is anticipated to be negligible for the area.

Based on our observations, the long term groundwater level is anticipated at a depth of 6 m below the existing grade within the bedrock. Therefore, no adverse effects to neighbouring properties is expected.

## 6.6 Winter Construction

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

Where excavations are completed in proximity to existing structures, they may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, 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.

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.

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

## 6.7 Corrosion Potential and Sulphate

The results of the analytical testing show that the sulphate content is less than 0.1%. This result indicates that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and pH of the samples 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 an aggressive to very aggressive corrosive environment.

# 7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

- ❑ Review the bedrock stabilization and excavation requirements.
- ❑ 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 and follow-up 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 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 its 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 EBC Inc. 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.**

Scott S. Dennis, P.E.


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

**Report Distribution**

- ❏ EBC Inc. (3 copies)
- ❏ Paterson Group (1 copy)

# APPENDIX 1

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TESTING RESULTS**

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Multi-Storey Building - 1950 Scott Street**
**Ottawa, Ontario**

**DATUM** TBM - Top spindle of fire hydrant located in front of 320 McRae Avenue. Geodetic elevation = 64.44m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** March 6, 2018

**FILE NO.** PG4394

**HOLE NO.** BH 1

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 64.34 |
| Asphaltic concrete | 0.10 | AU | 1 | | | | |
| **FILL:** Crushed stone with silt and sand | 0.36 | | | | | | |
| **GLACIAL TILL:** Brown silty sand, some gravel and cobbles | 1.50 | SS | 2 | 50 | 17 | 1 | 63.34 |
| **BEDROCK:** Grey limestone | 9.91 | RC | 1 | 95 | 70 | 2 | 62.34 |
| | | RC | 2 | 98 | 91 | 3 | 61.34 |
| | | | | | | 4 | 60.34 |
| | | RC | 3 | 99 | 99 | 5 | 59.34 |
| | | RC | 4 | 96 | 95 | 6 | 58.34 |
| | | | | | | 7 | 57.34 |
| | | RC | 5 | 98 | 98 | 8 | 56.34 |
| | | RC | 6 | 100 | 100 | 9 | 55.34 |

End of Borehole

(GWL @ 7.79m - March 19, 2018)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

# patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Multi-Storey Building - 1950 Scott Street**
**Ottawa, Ontario**

**DATUM** TBM - Top spindle of fire hydrant located in front of 320 McRae Avenue. Geodetic elevation = 64.44m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** March 6, 2018

**FILE NO.** PG4394

**HOLE NO.** BH 2

| SOIL DESCRIPTION | Depth of change (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| 25mm Asphaltic concrete over crushed stone with silt and sand | 0.25 | AU | 1 | | |
| GLACIAL TILL: Brown silty sand, trace gravel, cobbles and boulders | | SS | 2 | 0 | 50+ |
| | | SS | 3 | 44 | 50+ |
| BEDROCK: Grey limestone | 2.06 | RC | 1 | 48 | 40 |
| | | RC | 2 | 98 | 75 |
| | | RC | 3 | 98 | 92 |
| | | RC | 4 | 100 | 98 |
| | | RC | 5 | 100 | 98 |
| | | RC | 6 | 100 | 100 |
| End of Borehole | 10.15 | | | | |

(GWL @ 6.05m - March 19, 2018)

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 64.79 |
| 1 | 63.79 |
| 2 | 62.79 |
| 3 | 61.79 |
| 4 | 60.79 |
| 5 | 59.79 |
| 6 | 58.79 |
| 7 | 57.79 |
| 8 | 56.79 |
| 9 | 55.79 |
| 10 | 54.79 |

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Multi-Storey Building - 1950 Scott Street**
**Ottawa, Ontario**

**DATUM** TBM - Top spindle of fire hydrant located in front of 320 McRae Avenue. Geodetic elevation = 64.44m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** March 6, 2018

**FILE NO.** PG4394

**HOLE NO.** BH 3

| SOIL DESCRIPTION | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 64.23 |
| Asphaltic concrete (0.10) | AU | 1 | | | | |
| **FILL:** Crushed stonew with silt and sand (0.36) | | | | | | |
| **GLACIAL TILL:** Brown silty sand, some gravel, cobbles and boulders (1.62) | SS | 2 | 46 | 42 | 1 | 63.23 |
| **BEDROCK:** Grey limestone | RC | 1 | 100 | 84 | 2 | 62.23 |
| | RC | 2 | 97 | 83 | 3 | 61.23 |
| | | | | | 4 | 60.23 |
| | RC | 3 | 100 | 99 | 5 | 59.23 |
| | RC | 4 | 97 | 96 | 6 | 58.23 |
| | | | | | 7 | 57.23 |
| | RC | 5 | 100 | 100 | 8 | 56.23 |
| | | | | | 9 | 55.23 |
| (10.19) | RC | 6 | 98 | 98 | 10 | 54.23 |

End of Borehole

(GWL @ 5.94m - March 19, 2018)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

# SYMBOLS AND TERMS

## SOIL DESCRIPTION

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

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

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

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

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

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

## SOIL DESCRIPTION (continued)

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

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

## ROCK DESCRIPTION

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

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

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

## SAMPLE TYPES

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

## SYMBOLS AND TERMS (continued)

## GRAIN SIZE DISTRIBUTION

| | | |
|---|---|---|
| MC% | - | Natural moisture content or water content of sample, % |
| LL | - | Liquid Limit, % (water content above which soil behaves as a liquid) |
| PL | - | Plastic limit, % (water content above which soil behaves plastically) |
| PI | - | Plasticity index, % (difference between LL and PL) |
| Dxx | - | Grain size which xx% of the soil, by weight, is of finer grain sizes<br>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

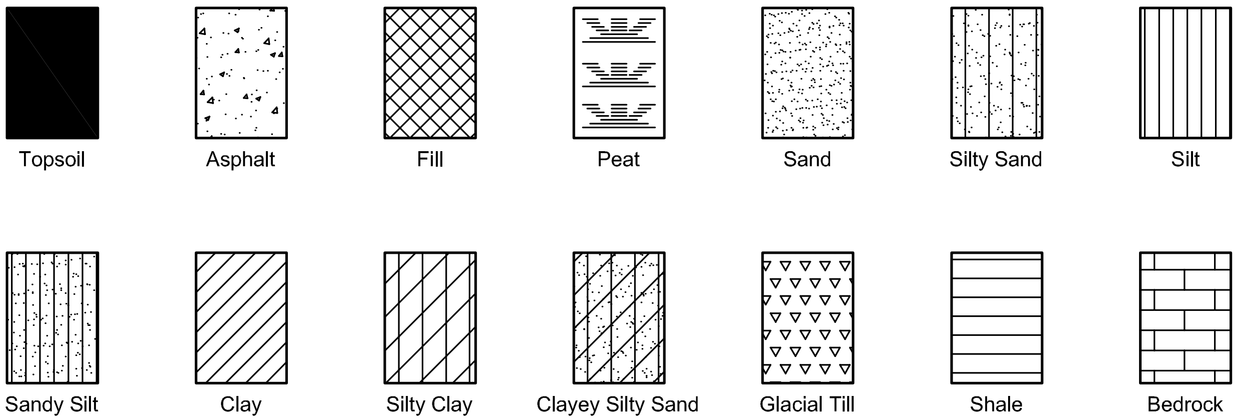
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above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

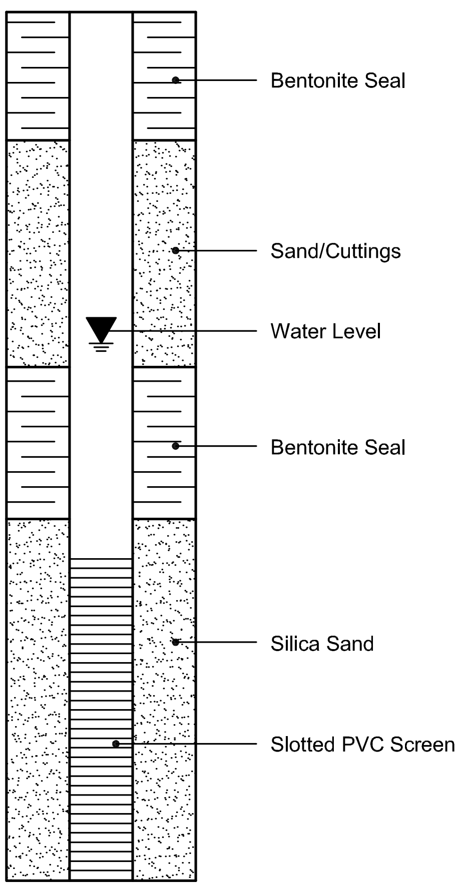
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

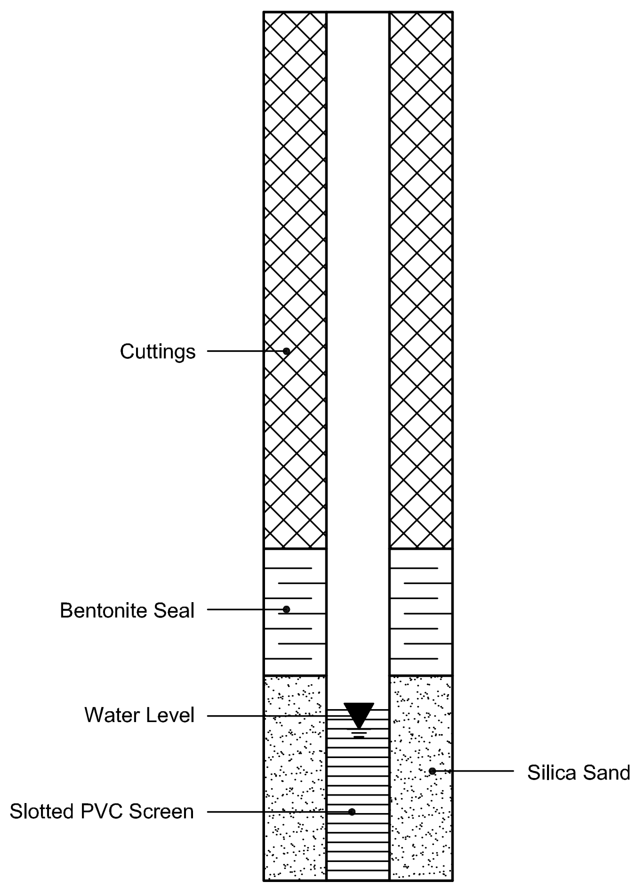



## MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION




PIEZOMETER CONSTRUCTION



Certificate of Analysis
**Client: Paterson Group Consulting Engineers**
**Client PO: 23596**

Report Date: 16-Mar-2018
Order Date: 12-Mar-2018
**Project Description: PG4394**

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH3 SS2 0'6"-4'6" | - | - | - |
| | **Sample Date:** | 06-Mar-18 | - | - | - |
| | **Sample ID:** | 1811058-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 94.9 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.91 | - | - | - |
| Resistivity | 0.10 Ohm.m | 45.1 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 14 | - | - | - |
| Sulphate | 5 ug/g dry | 38 | - | - | - |

# APPENDIX 2

**FIGURE 1 - KEY PLAN**

**DRAWING PG4394-1 - TEST HOLE LOCATION PLAN**

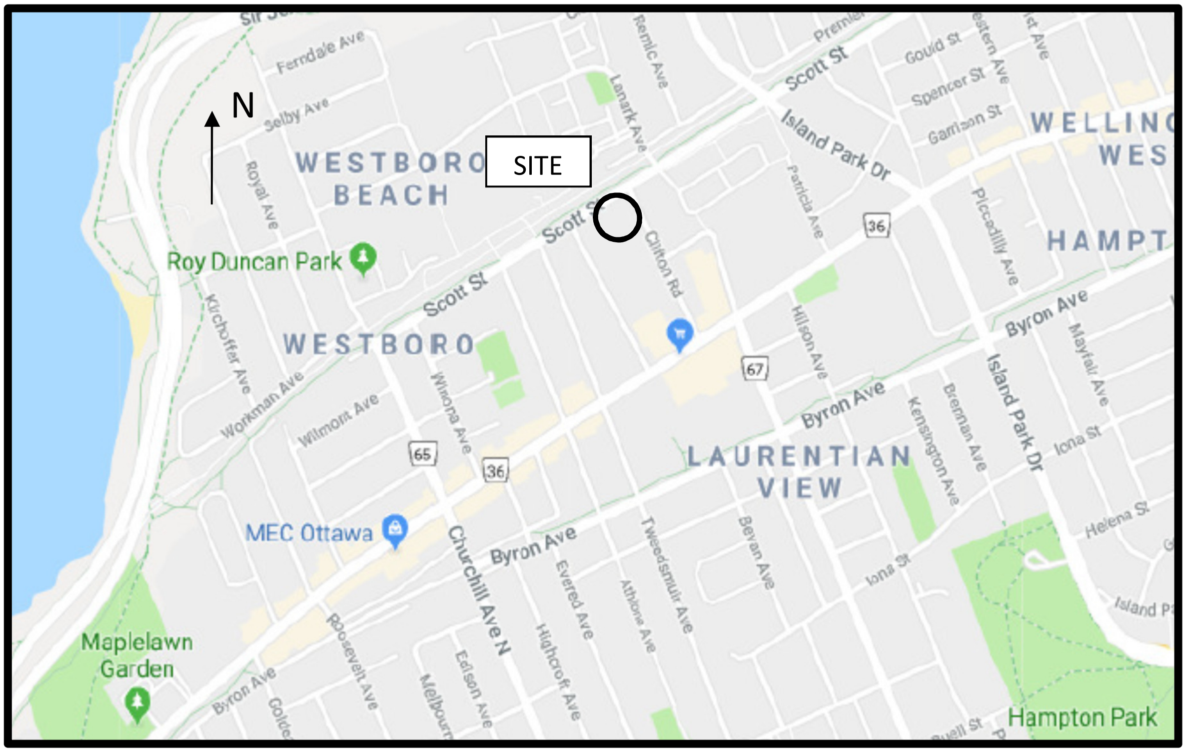
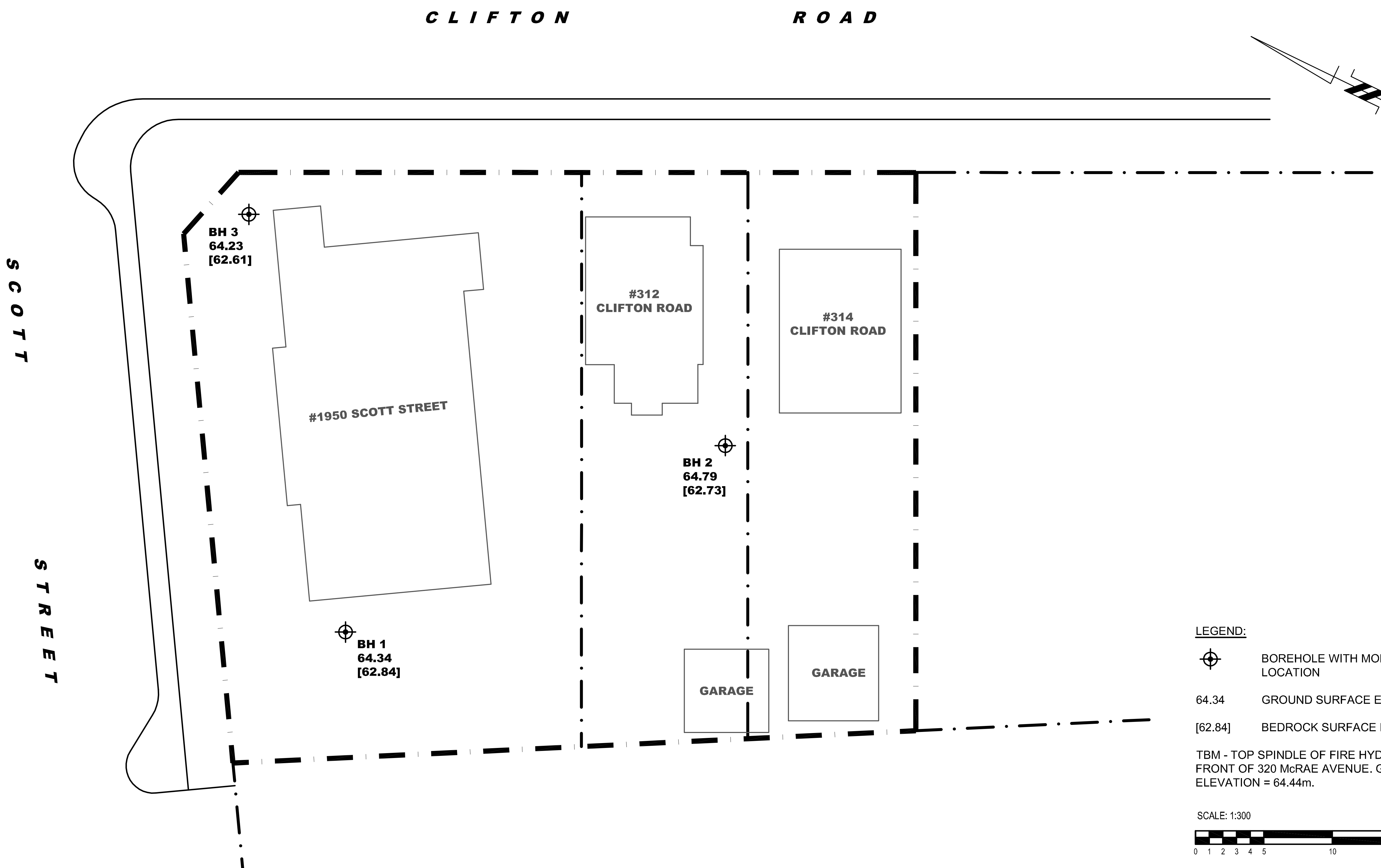



FIGURE 1

# KEY PLAN




LEGEND:

BOREHOLE WITH MONITORING WELL LOCATION

64.34 GROUND SURFACE ELEVATION (m)

[62.84] BEDROCK SURFACE ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED IN FRONT OF 320 McRAE AVENUE. GEODETIC ELEVATION = 64.44m.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m


**patersongroup**
**consulting engineers**

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344


| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 0 | | | |

EBC INC.
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 1950 SCOTT ST., 312 & 314 CLIFTON RD.
OTTAWA, ONTARIO

Title:
# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Scale: 1:300 | Date: 03/2018 |
| Drawn by: MPG | Report No.: PG4394-1 |
| Checked by: SD | Dwg. No.: **PG4394-1** |
| Approved by: DJG | Revision No.: 0 |

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