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Geotechnical Investigation

Proposed Hi-Rise Mixed Use Development
208 to 212 Slater Street
Ottawa, Ontario

Prepared For

Broccolini

May 9, 2019

Report PG4608-1

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Broccolini to conduct a geotechnical investigation for the subject site located at 208 to 212 Slater Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❏ determine the subsurface soil and groundwater conditions based on borehole information.

- ❏ 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. Environmental information is provided under a separate cover.

2.0 Proposed Development

For the proposed development, it's expected that a 23 storey mixed use hi-rise building with one basement level for mechanical use is being considered. The footprint of the basement will occupy the entire boundaries of the subject site. It's expected that the proposed structure will be fully municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program was conducted on December 10, 2015 where 3 boreholes were advanced to a maximum depth of 6.4 m below existing grade. The test holes were distributed in a manner to provide general coverage of the subject site. The locations were determined in the field by Paterson personnel taking into consideration site features and underground services. Approximate locations of the test holes are shown in Drawing PG4608-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using truck-mounted auger drill rig and portable drilling equipment operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedure consisted of augering to the required depths at select locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. 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.

Standard Penetration Tests (SPT) were 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.

Diamond drilling was carried out at three borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. 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.

Groundwater

A 32 mm in diameter PVC groundwater monitoring well was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❏ Slotted 32 mm diameter PVC screen at the base of the aforementioned boreholes.
- ❏ 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.
- ❏ A minimum of 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❏ Clean backfill from top of bentonite plug to the ground surface.

The groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top of grate of a catch basin located within the parking lot of located east of the subject site. An assumed elevation of 100 m was assigned to the TBM. The borehole locations are presented on Drawing PG4608-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a commercial building and a pedestrian pathway. The ground surface across the site is generally flat with a slight downslope towards the west. The ground surface was noted to be at grade with the surrounding roadways.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of asphaltic concrete overlying a brown silty sand with crushed stone fill layer. The pavement structure is underlain by stiff brown silty clay followed by glacial till. The glacial till consists of brown silty clay with some sand, gravel and cobbles. Shale bedrock was encountered below the glacial till layer in all borehole locations at depth ranging between 3.1 and 5.92 m below existing grade. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Paterson completed studies within the local area of the subject site. Based on these studies, limestone bedrock was encountered below the underlying shale bedrock.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Billings Formation, and interbedded shale and limestone from the Ottawa Formation. Bedrock is expected to be encountered at depths varying between 2 and 5 m.

4.3 Groundwater

Groundwater levels were measured in monitoring wells within the boreholes on February 17, 2016. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole, which can lead to higher than normal groundwater level readings. Long-term groundwater level can also be estimated based on the observed moisture levels, colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected between 6 to 7 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	100.37	5.70	94.67	February 17, 2016
BH 2	99.20	Dry	n/a	February 17, 2016
BH 3	98.26	5.10	93.16	February 17, 2016

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. it is expected that the building will be founded on conventional spread footing foundations placed on a clean, surface sounded shale bedrock bearing surface.

Expansive shale of the Billings Formation could be present at shallow depths. Precautions should be taken during construction to reduce the risks associated with the potential for heaving of the shale bedrock. It will also be important to minimize groundwater lowering adjacent to existing buildings founded on the shale bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organics or construction debris, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the anticipated number of underground parking levels and depth of the bedrock at the subject site, it is anticipated that all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s 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 an experienced blasting consultant.

Vibration Considerations

Construction operations could cause 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 a cooperative environment with the residents.

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, 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). These guidelines are for current construction standards.

Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Rock Stabilization

Due to the proximity of existing structures to the north, east, and west of the proposed development, stabilization of bedrock will be required when the proposed foundation extends below the existing founding elevations.

It is expected that vertical rock reinforcement will be required prior to excavating the fractured and weathered bedrock below founding elevations. To accomplish this, consideration should be given to drilling vertical holes at 1 m or less from the founding elevation of the adjacent existing building and extending down to the proposed founding elevation of the new building. Steel reinforcement should be grouted within these vertical drill holes. The purpose of the vertical rock reinforcement will be to prevent or lessen the undermining of the vertical face of the excavation adjacent to existing buildings.

Furthermore, horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Protection of Potential Expansive Bedrock

It is possible that expansive shale will be encountered at the subject site. Although the effects of expansive shale will not affect the proposed development, it is possible that it will affect the adjacent buildings founded at higher elevations in the shale bedrock.

It is recommended that the exposed excavation vertical face of the bedrock below the groundwater level and adjacent to building foundations should be protected from excessive dewatering and exposure to ambient air. To accomplish this, it is suggested that the face of the exposed bedrock be covered with a 50 mm thickness of shotcrete within a 48 hour period of being exposed. The shotcrete should extend to at least 6 m on either side of the building. Once cured, the shotcrete should be sprayed with emulsion or a suitable membrane. It is expected that a composite drainage layer will also be incorporated into the design to relieve hydrostatic pressure. It is also recommended, if possible, to place an intermediate perimeter drainage perforated pipe at the bedrock/soil interface between the existing and proposed buildings to avoid trapping surface water infiltration. The above recommendations should be reviewed and most likely revised to accommodate the final design.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted using suitable compaction equipment.

Fill placed beneath the building should be compacted to a minimum of 98% of the 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 maximum 300 mm thick lifts 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

Auxiliary footings (canopy and shafts) placed over a clean, surface sounded limestone or shale 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.

For deeper footings within the sound bedrock, a factored bearing resistance value at ULS of **3,000 kPa** , incorporating a geotechnical resistance factor of 0.5, can be used if the bedrock is free of seams, fractures and voids within 1.5 m below the founding level.

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.

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 down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

For foundation placed on the glacial till deposit, the above noted bearing resistance values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing directly or indirectly on bedrock will be subjected to negligible settlements.

Bedrock Stabilization

Horizontal rock anchors or rock bolts may be required at specific locations where excavations are completed in bedrock below existing footings or vertically through the bedrock adjacent to footings to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

5.4 Design for Earthquakes

The proposed site can be taken as seismic site response **Class A** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). However, a site specific shear wave velocity testing will be required to confirm this site classification. The soil underlying the proposed shallow foundations are not susceptible to liquefaction.

5.5 Basement 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 98% of its SPMDD.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, may be required to be provided within the clear stone backfill under the lowest basement floor.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Basement Wall

It is expected that the basement walls are to be poured against a waterproofing and/or drainage system, which will be placed against the shoring face and exposed bedrock face, where encountered. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

The recommended pavement structures for the subject site are shown in Tables 2 and 3 below.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-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 with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room is available for exterior backfill. It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that a 150 mm in diameter perforated pipe be used for the perimeter drainage system with positive drainage to the storm sewer or sump pit. An interior perimeter drainage consisting of a minimum 150 mm diameter perforated, corrugated PVC pipe be placed along the interior side of the exterior footing. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Water Suppression

To prevent excessive lowering of the groundwater table that may affect the expansive shale for surrounding buildings, consideration should be given to placing a bentonite layer against the bedrock face before placing the composite drainage layer.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 to 9 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

For areas where sufficient space is available for backfill against the exterior sides of the foundation walls, the backfill material 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 be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior 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.

The underground parking area should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated 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

Temporary Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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 will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

Temporary shoring may be required 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. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. 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 4 - Soil Parameters for Shoring System Design

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 (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

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.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further 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. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

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 γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ 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.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and 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.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation 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 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.

Long-term Groundwater Control

Our preliminary 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 sub-slab drainage system will be directed to the proposed building's cistern/sump pit.

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighbouring structures are expected to be founded within the native stiff silty clay, glacial till deposit and/or over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the low permeability of the native soils. A bentonite layer will be placed against the bedrock vertical face to lessen the effects of long term dewatering.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

Where excavations are completed in proximity of existing structures which 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.

7.0 Recommendations

It is recommended that the following be carried out once the site development plans are finalized and during construction:

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Evaluation and assessment of the groundwater infiltration volumes during construction.
- ❑ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ❑ Review and inspection of the waterproofing membrane for the water suppression system prior to pouring foundation walls.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Inspection of below grade drainage system (french drain) and underfloor drainage system along with the sump pit installation.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 Broccolini or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 9, 2019
C. P. DA SILVA
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

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

Report Distribution

- ❑ Broccolini (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Building - 208-212 Slater Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

FILE NO. PG4608

REMARKS

HOLE NO. BH 1

BORINGS BY Portable Drill **DATE** December 10, 2015

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	G	1		
FILL: Crushed stone, some sand, occasional asphalt pieces	0.51				
FILL: Brown silty sand with crushed stone	2.44	SS	2	17	
		SS	3	4	
Stiff, brown **SILTY CLAY**	3.94	SS	4	88	
		SS	5	100	
		SS	6	100	
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles	5.92	RC	1		
BEDROCK: Black shale	7.37	RC	2	42	
		RC	3	27	0
		RC	4	4	0
End of Borehole					
(GWL @ 5.70m-Feb. 17, 2016)					

DEPTH (m)	ELEV. (m)
0	100.37
1	99.37
2	98.37
3	97.37
4	96.37
5	95.37
6	94.37
7	93.37

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. High-Rise Building - 208-212 Slater Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

FILE NO. PG4608

REMARKS

HOLE NO. BH 2

BORINGS BY Portable Drill **DATE** February 11, 2016

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.20
Concrete floor slab	0.05						
FILL: Crushed stone	0.51						
Brown **SILTY CLAY,** trace gravel	1.45					1	98.20
GLACIAL TILL: Brown silty sand with gravel	3.10	SS	1	79			
		SS	2	54		2	97.20
BEDROCK: Black shale	6.40	RC	1	73	39	3	96.20
		RC	2	92	54	4	95.20
		RC	3	84	32	5	94.20
						6	93.20
End of Borehole							
(Borehole dry - Feb. 17, 2016)							

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. High-Rise Building - 208-212 Slater Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

REMARKS

BORINGS BY Portable Drill

DATE February 10, 2016

FILE NO. PG4608

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Concrete floor slab	0.05				
FILL: Crushed stone, some cobbles	0.86				
Brown **SILTY CLAY,** trace gravel	2.11	G	1		
GLACIAL TILL: Black silty sand with shale and gravel, some clay	3.99	SS	2	56	
		SS	3	33	
		SS	4	69	
BEDROCK: Black shale	5.46	RC	1	100	0
		RC	2	100	0
		RC	3	100	17
End of Borehole					
(GWL @ 5.10m-Feb. 17, 2016)					

DEPTH (m)	ELEV. (m)
0	98.26
1	97.26
2	96.26
3	95.26
4	94.26
5	93.26

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 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 = $(D30)^2 / (D10 \times D60)$
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

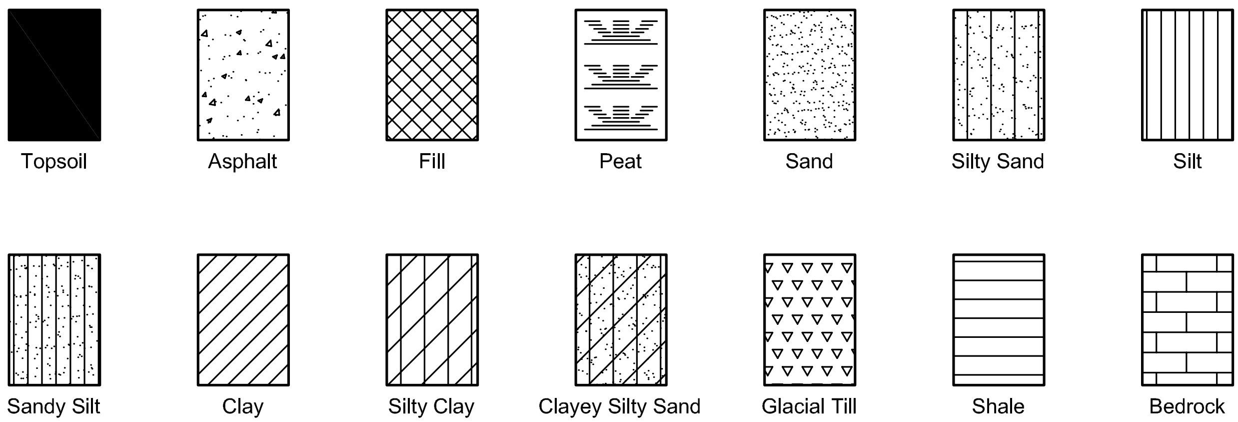
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

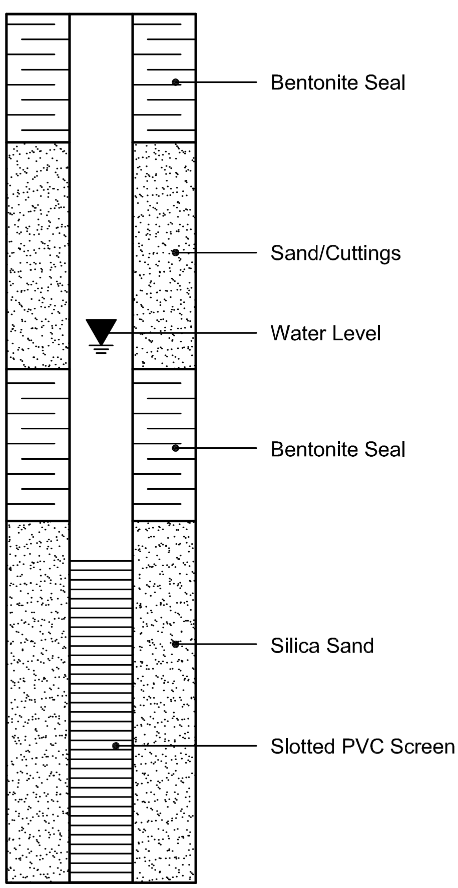
SYMBOLS AND TERMS (continued)

STRATA PLOT

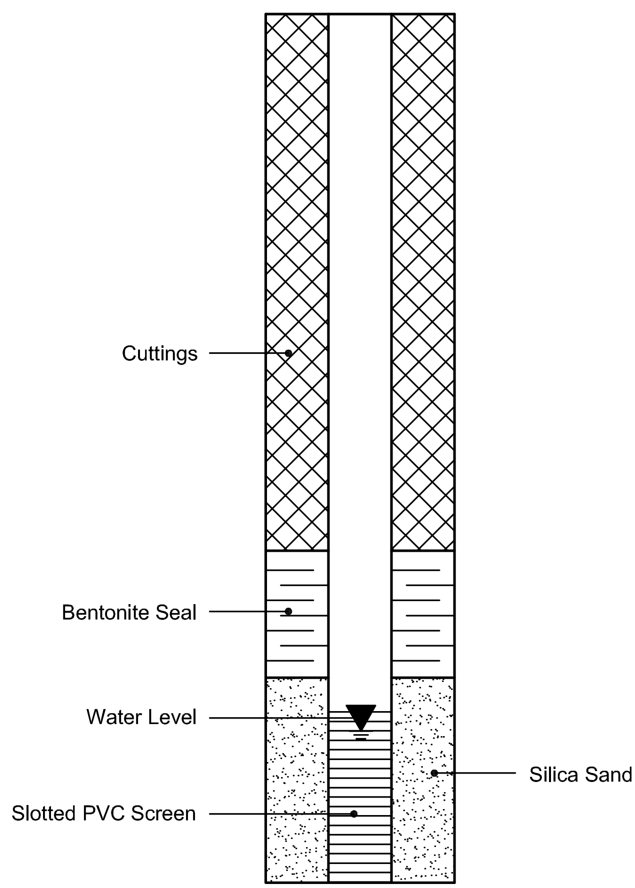


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4608-1 - TEST HOLE LOCATION PLAN

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Building - 208-212 Slater Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

FILE NO. PG4608

REMARKS

HOLE NO. BH 1

BORINGS BY Portable Drill **DATE** December 10, 2015

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.37
Asphaltic concrete 0.10						
FILL: Crushed stone, some sand, occasional asphalt pieces 0.51	G	1				
FILL: Brown silty sand with crushed stone	SS	2	17		1	99.37
	SS	3	4		2	98.37
2.44						
Stiff, brown **SILTY CLAY**	SS	4	88		3	97.37
	SS	5	100			
3.94	SS	6	100		4	96.37
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles	RC	1			5	95.37
5.92						
BEDROCK: Black shale	RC	2	42		6	94.37
	RC	3	27	0		
	RC	4	4	0	7	93.37
7.37						
End of Borehole						
(GWL @ 5.70m-Feb. 17, 2016)						

STRATA PLOT

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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Ottawa, Ontario

DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

FILE NO. **PG4608**

REMARKS

HOLE NO. **BH 2**

BORINGS BY Portable Drill **DATE** February 11, 2016

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.20
Concrete floor slab	0.05						
FILL: Crushed stone	0.51						
Brown **SILTY CLAY,** trace gravel	1.45					1	98.20
GLACIAL TILL: Brown silty sand with gravel	3.10	SS	1	79			
		SS	2	54		2	97.20
BEDROCK: Black shale	6.40	RC	1	73	39	3	96.20
		RC	2	92	54	4	95.20
		RC	3	84	32	5	94.20
						6	93.20
End of Borehole							
(Borehole dry - Feb. 17, 2016)							

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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DATUM TBM - Top of grate of catch basin. Assumed elevation = 100.00m.

REMARKS

BORINGS BY Portable Drill

DATE February 10, 2016

FILE NO. PG4608

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.26
Concrete floor slab	0.05						
FILL: Crushed stone, some cobbles	0.86						
Brown **SILTY CLAY,** trace gravel	2.11	G	1			1	97.26
						2	96.26
GLACIAL TILL: Black silty sand with shale and gravel, some clay	3.99	SS	2	56			
		SS	3	33		3	95.26
		SS	4	69			
BEDROCK: Black shale	5.46	RC	1	100	0	4	94.26
		RC	2	100	0		
		RC	3	100	17	5	93.26

End of Borehole

(GWL @ 5.10m-Feb. 17, 2016)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
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20 40 60 80

Monitoring Well Construction

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Shear Strength (kPa)
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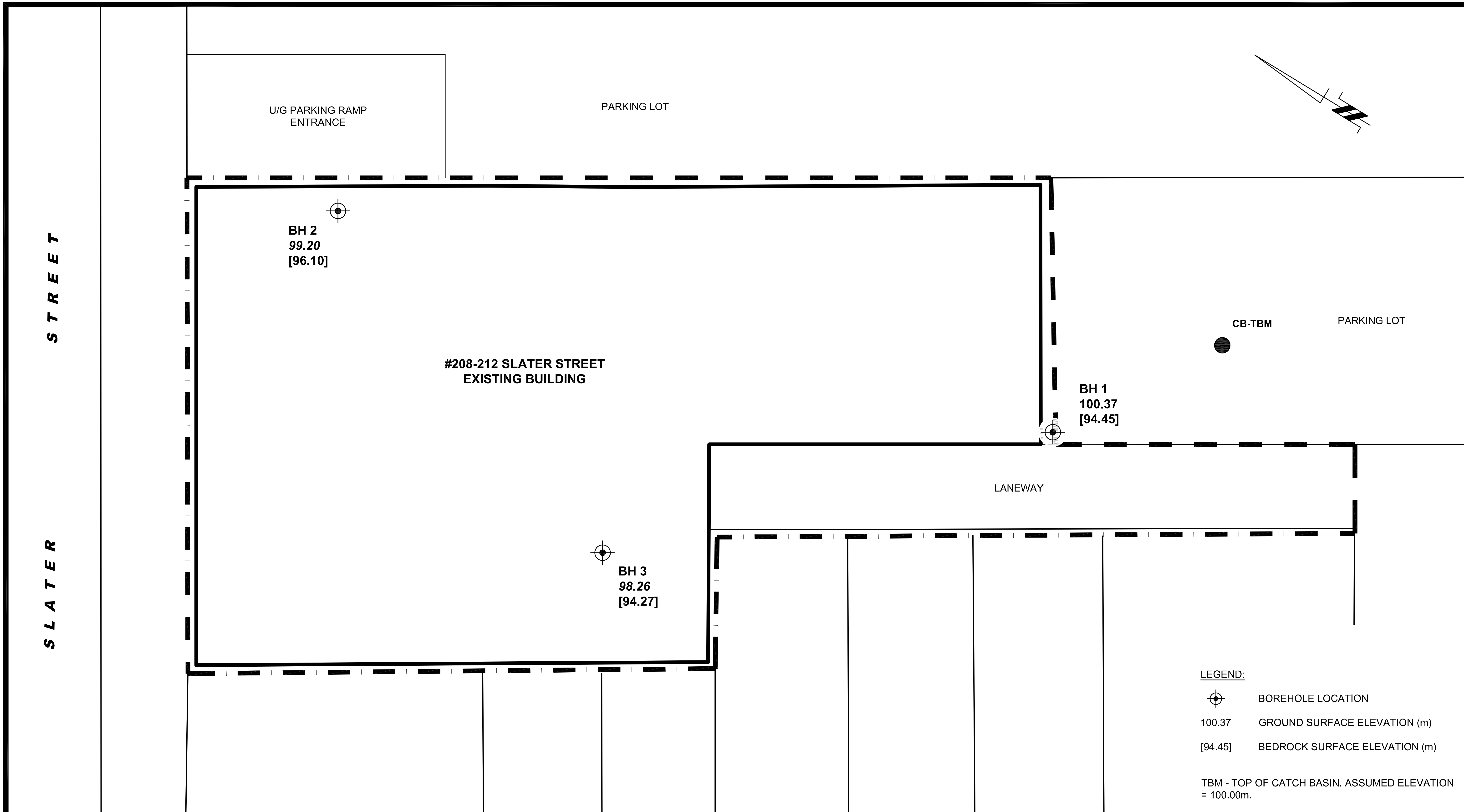

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NO.	REVISIONS	DATE	INITIAL
0			

BROCCOLINI

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDING - 208-212 SLATER STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 07/2018
Drawn by: MPG	Report No.: PG4608-1
Checked by: FA	Dwg. No.: PG4608-1
Approved by: FA	Revision No.:

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