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Geotechnical Investigation

Proposed Multi-Storey Building
Churchill Avenue North and Byron Place
Ottawa, Ontario

Prepared For

2592532 Ontario Inc.

February 25, 2019

Report PG4712-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 2592532 Ontario Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 433 and 435 Churchill Avenue North and 468 and 472 Byron Place in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

❏ determine the existing subsoil and groundwater conditions at this site by means of boreholes.

❏ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

Environmental considerations for this site, such as potential contamination, have been prepared under separate cover.

2.0 Proposed Project

The proposed development is understood to consist of one multi-storey residential building with two levels of underground parking. The remainder of the site will consist of access lanes, walking pathways and landscaped areas. It is understood that all existing buildings occupying the subject site will be demolished as part of the proposed development including a detached garage partially occupying the subject site along the north boundary. It is also anticipated that municipal services will also be constructed as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was completed on January 7, 2019. At that time, 3 boreholes were drilled to a maximum depth of 9.1 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site. The borehole locations are presented on Drawing PG4712-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon or the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to the laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are presented as SS and AU, respectively, on the Soil Profile and Test Data sheets.

Standard Penetration Tests (SPT) were conducted and 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 sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was carried out at all borehole locations to assess the bedrock quality. Rock samples were recovered from BH 1 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report and discarded unless otherwise directed.

Groundwater

One monitoring well consisting of 50 mm diameter rigid PVC pipe was installed at BH 1 to permit monitoring of the groundwater levels and groundwater sampling subsequent to completion of the sampling program. Flexible polyethylene standpipes were installed in the other boreholes to permit groundwater level monitoring.

3.2 Field Survey

The borehole locations and ground surface elevations at the borehole locations were laid out by Paterson in the field. The ground surface elevations are referenced to a temporary benchmark (TBM) consisting of the top spindle of a fire hydrant in front of 434 Byron Place. A geodetic elevation of 77.69 m was provided to the TBM on the topographic survey plan provided by Farley, Smith & Denis Surveying Ltd. The borehole locations and the ground surface elevations at the borehole locations are presented on Drawing PG4712-1 - 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.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its sulphate and chloride concentrations, as well as its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located at the southwest corner of the intersection between Churchill Avenue North and Byron Place. The site is currently occupied by an existing multi-storey residential apartment building and several houses. The ground surface across the site is relatively flat and at grade with the surrounding roadways.

The site is bordered to the north by Byron Place and Byron Avenue beyond, to the east by Highcroft Avenue, to the south by additional houses and apartment buildings and to the west by Churchill Avenue North.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consisted of a layer of asphalt and/or granular fill overlying brown silty clay to silty sand deposits. Practical refusal to augering was encountered at all borehole locations at depths ranging between 1.3 and 2 m depth on grey limestone bedrock surface.

Bedrock

Bedrock was cored at BH 1 and indicated a weathered to good quality limestone bedrock. Approximately 1 m of the upper portion of the bedrock was noted to be weathered. The recovery values range from 94 to 100%, while the RQD values varied between 19 and 100%. Based on the results the bedrock quality ranges from poor to good.

Based on available geological mapping, the local bedrock consists of interbedded limestone and dolomite of the Gull River formation with an anticipated overburden thickness of 0 to 2 m.

Specific details of the subsoil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

Groundwater levels were recorded at the monitoring well and piezometers installed at the borehole locations on January 14, 2019. The groundwater level readings noted at that time are presented in Table 1 below. Based on these observations, the long-term groundwater level is anticipated to be located at or below the bedrock surface, at a depth ranging between 4.5 and 5.5 m below existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore levels could differ 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	77.50	4.94	72.56	January 14, 2019
BH 2	77.18	Dry	-	January 14, 2019
BH 3	77.93	Dry	-	January 14, 2019

Notes:
Ground elevations at borehole locations are referenced to a TBM consisting of the top spindle of a fire hydrant shown on the topographic survey plan prepared by Farley, Smith & Denis Surveying Ltd. A geodetic elevation of 77.69 m was provided to the TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey buildings. The proposed buildings are expected to be founded on conventional footings placed on clean, surface sounded bedrock bearing surface.

Bedrock removal will be required to complete the underground parking levels. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed buildings, all existing overburden material should be excavated from within the proposed building footprint.

Topsoil, deleterious fill, such as those containing organic materials and construction debris should be stripped from under any buildings and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Bedrock Removal

Based on the bedrock encountered in the area, 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.

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.

Prior to considering construction 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.

Precaution should be taken to limit blasting effects within the excavation sidewalls, which could increase infiltration rates of groundwater into the excavation.

To further reduce the potential increase of infiltration rates into the excavation sidewalls, it is recommended that the final 150 to 300 mm of the bedrock removal be carried out using a rock grinder mounted to a hydraulic excavator. This method of bedrock grinding will provide a smoother surface to finalized the shape of the sidewall and will also lessen the potential for over breaks that typically occur with the use of high energy mechanical methods such as hoe-ramming.

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

Any bedrock removed via hoe-ramming or blasting methods may be stockpiled at the site and reviewed by the geotechnical consultant for use as backfill below the building footprints and as general landscaping fill.

Vibration Considerations

Construction operations could be the cause 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 equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, all vibrations are recommended to be limited during construction.

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 buildings.

Horizontal Rock Anchors

Horizontal rock anchors may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors should be evaluated during the excavation operations and discussed with the structural engineer during the design stage.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa** incorporating a geotechnical resistance factor of 0.5.

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 could be used if founded on limestone bedrock and the bedrock 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 footing footprint(s). One drill hole should be completed per footing. 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) passes 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 for 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** for foundations considered at this site. A higher seismic site class, such as Class A or B, may be available for foundations placed on the bedrock surface. However, the higher seismic site class would have to be confirmed by site-specific shear wave velocity testing. The subsoils at this site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

All overburden soil will be removed for the proposed building and the basement floor slab will be founded on a bedrock medium. It is expected that the basement area will be mostly parking and a rigid pavement structure designed by a structural engineer will be applicable. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. However, if storage or other uses of the lower level where a concrete floor slab will be used 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, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

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 dry unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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

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 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 $P_o = 0.5 \cdot K_o \cdot \gamma \cdot H^2$, 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 Rock Anchor Design

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 from a 60 to 90 degree cone with the apex near the middle of the anchor bonded length. Interaction may develop between the failure cones of adjacent anchors resulting in a total group capacity less than the sum of the individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

Regardless of whether an anchor is of the passive or post tensioned type, the anchor is recommended to be provided with a fixed length at the anchor base, which will provide the anchor capacity, and a free length between the rock surface and the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor has a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is at the bottom portion of the anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. 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 buildings, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone bedrock typically exceeds 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1,000 kPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

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. A **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively. For design purposes, all rock anchors were assumed to be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For calculations the parameters given in Table 2 were used.

Table 2 - Parameters Used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair Quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.1	0.9	2	250
	2.3	0.8	3.1	500
	4.4	0.6	5	1000
125	1.7	1.6	3.3	250
	2.3	1.6	3.9	500
	3.3	1.7	5	1000

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

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.

5.8 Pavement Structure

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

Table 4 - 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 5 - 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

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 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 structures. It is expected that insufficient room is available for exterior backfill. It is suggested that this system could be as follows:

- ❑ Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface);
- ❑ A waterproofing membrane will be required to lessen the effect of water infiltration for the lower P-2 basement level. The waterproofing membrane can be placed and fastened to the bedrock vertical face and should extend from 1 m above the groundwater level and down to the bottom of the excavation at the founding level of the exterior foundation. The requirement of a waterproofing membrane can be confirmed once the excavation completed and the water infiltration can be better assessed;
- ❑ composite drainage layer

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) 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 the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration at the proposed founding elevation. For preliminary design purposes, Paterson recommends to place a series of 150 mm diameter perforated pipes spaced at 6 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

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and are not recommended for placement as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

Exterior unheated footings, such as 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 combination of soil cover and foundation insulation.

The parking garage 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 temporary excavation side slopes should either be excavated to acceptable slopes or retained by shoring systems from the beginning 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

A trench box is recommended to be installed at all times to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain exposed for extended periods of time.

In bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors as per the geotechnical consultants recommendations. Since the parking garage structures may be located within close proximity to neighbouring properties, rock bolts and other temporary support may be required during construction.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided below is to assist the contractor in completing a safe shoring system. The shoring designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s consultants prior to implementation.

The temporary system could 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 included to the earth pressures described below. The shoring system could be cantilevered, anchored or braced. Generally, the shoring systems is provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. If consideration is given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes movements to tolerable levels.

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

Table 6 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible, while the at-rest pressure should be calculated if movement is not permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

A hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

Concrete Underpinning

Based on proximity of existing adjacent buildings, support in the form of concrete underpinning maybe required during excavation for the proposed building. It is expected that the founding level of the existing foundations will be on bedrock or in close proximity to the bedrock surface and conventional concrete underpinning may be used to support the full width and length of the foundation.

It is expected that the structural engineer along with the geotechnical engineer will review the site conditions at the time of construction and finalize the underpinning program based on their observations at that time, if required.

6.4 Pipe Bedding and Backfill

A minimum of a 150 mm of OPSS Granular A should be placed for pipe bedding for sewer and water pipes for a soil subgrade. The bedding thickness should be increased to 300 mm for areas where the subgrade consists of bedrock. 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. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the SPMDD.

The site excavated material may be placed above cover material if the excavation operations are completed in dry weather conditions and the site excavated material is approved by the geotechnical consultant. All cobbles greater than 200 mm in the longest dimension should be removed prior to the materials being reused.

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 differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD. Within the frost zone (1.8 m below finished grade), non frost susceptible materials should be used when backfilling trenches below the original bedrock level.

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 moderate to high through the excavation face, depending on the local groundwater table. The groundwater infiltration will be controllable with open sumps and pumps.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 and 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 stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Any groundwater encountered along the building's perimeter or sub-slab 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, the expected long-term groundwater flow should 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. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Structures

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building.

Due to the shallow local bedrock, the neighbouring structures are expected to generally be founded directly over a bedrock bearing surface. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the low permeability of the bedrock. Therefore, adverse effects to the surrounding buildings or properties are not expected with the lowering of the groundwater in this area.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. 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 in the contract documents should be provided to protect the excavation walls 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 excavation base 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 that 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a slightly aggressive corrosive environment.

7.0 Recommendations

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

❏ Review the bedrock stabilization and excavation requirements.

❏ Review groundwater infiltration at the time of construction.

❏ Review probe holes within the footing locations to determine bearing resistance values.

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

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

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling.

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

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

A report confirming that the construction has been conducted in general accordance with Paterson's recommendations could be issued upon the completion of a thorough inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in the report are in accordance with Paterson's present understanding of the project. Paterson request permission to review the recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the borehole locations, Paterson requests immediate notification to permit reassessment of the recommendations.

The recommendations provided should only be used by the design professionals associated with this project. The recommendations are not intended for contractors bidding on or constructing the project. The latter should evaluate the factual information provided in the report. The contractor should also determine the suitability and completeness for the intended construction schedule and methods. Additional testing may be required for the contractors purpose.

The present report applies only to the project described in the report. The use of the report for purposes other than those described above or by person(s) other than 2592532 Ontario Inc. or their agents is not authorized without review by Paterson.

Paterson Group Inc.

Nathan F. S. Christie, P.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ 2592532 Ontario 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
433 & 435 Churchill Ave. N. and 468 & 472 Byron Place
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Byron Place, in front of subject site. Geodetic elevation = 77.69m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 7 January 2019

FILE NO. PG4712

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic cocnrete	0.04	AU	1		
FILL: Crushed stone	0.10				
FILL: Brown silty clay, trace sand and gravel	0.76	SS	2	100	50+
GLACIAL TILL: Brown silty sand, trace clay and gravel	1.35	RC	1	100	0
BEDROCK: Grey limestone	9.12	RC	2	94	27
		RC	3	100	81
		RC	4	99	81
		RC	5	100	76
		RC	6	100	72

End of Borehole

(GWL @ 5.18m - Jan. 14, 2019)

DEPTH (m)	ELEV. (m)
0	77.50
1	76.50
2	75.50
3	74.50
4	73.50
5	72.50
6	71.50
7	70.50
8	69.50
9	68.50

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**
20 40 60 80

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
433 & 435 Churchill Ave. N. and 468 & 472 Byron Place
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Byron Place, in front of subject site. Geodetic elevation = 77.69m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 7 January 2019

FILE NO. PG4712

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.18
FILL: Crushed stone	0.05						
FILL: Brown clayey sand, trace gravel and brick	0.38	AU	1				
GLACIAL TILL: Brown silty sand, some clay and gravel		SS	2	58	32	1	76.18
	2.01	SS	3	100	50+	2	75.18
End of Borehole							

End of Borehole

Practical refusal to augering at 2.01m depth

(BH dry upon completion)

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ Lower Explosive Limit %
20 40 60 80

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
433 & 435 Churchill Ave. N. and 468 & 472 Byron Place
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Byron Place, in front of subject site. Geodetic elevation = 77.69m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 7 January 2019

FILE NO. PG4712

HOLE NO. BH 3

SOIL DESCRIPTION	Depth of layer base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.93
Asphaltic concrete	0.04						
FILL: Crushed stone	0.08	AU	1				
FILL: Brown silty sand, trace gravel	0.48						
FILL: Brown silty clay	1.07	SS	2	88	50+	1	76.93
GLACIAL TILL: Brown silty sand, some gravel and clay	1.73	SS	3	60	50+		

End of Borehole

Practical refusal to augering at 1.73m depth

(BH dry upon completion)

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

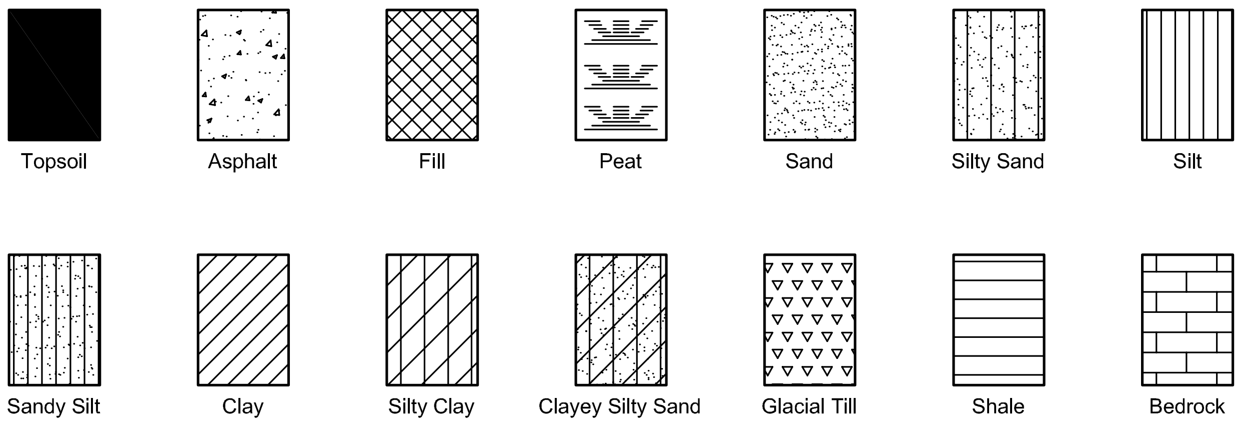
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

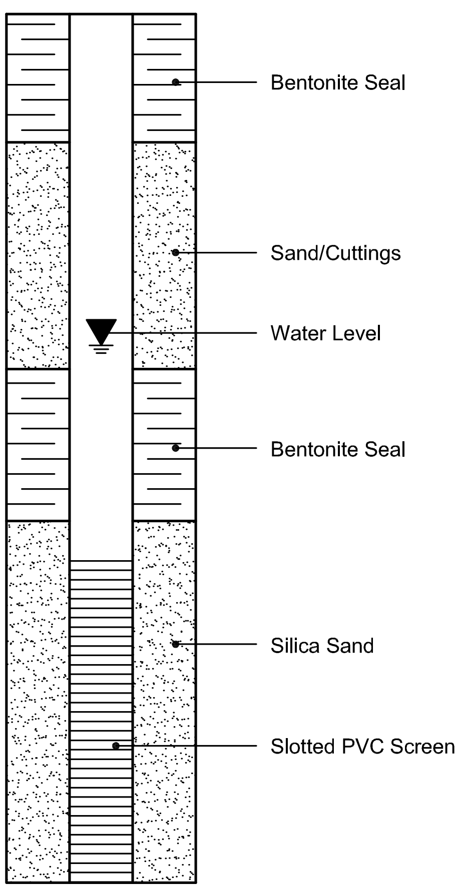
SYMBOLS AND TERMS (continued)

STRATA PLOT

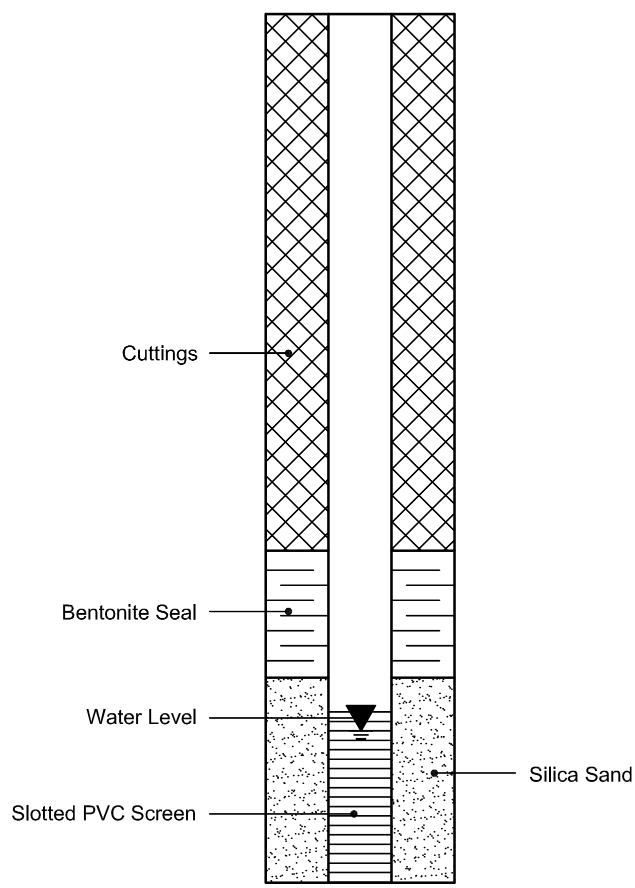


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO:

Report Date: 29-Jan-2019

Order Date: 24-Jan-2019

Project Description: PG4712

	Client ID:	BH1-SS2	-	-	-
	Sample Date:	01/07/2019 09:00	-	-	-
	Sample ID:	1904379-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.90	-	-	-
Resistivity	0.10 Ohm.m	50.4	-	-	-
Anions					
Chloride	5 ug/g dry	14	-	-	-
Sulphate	5 ug/g dry	21	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4712-1 - TEST HOLE LOCATION PLAN

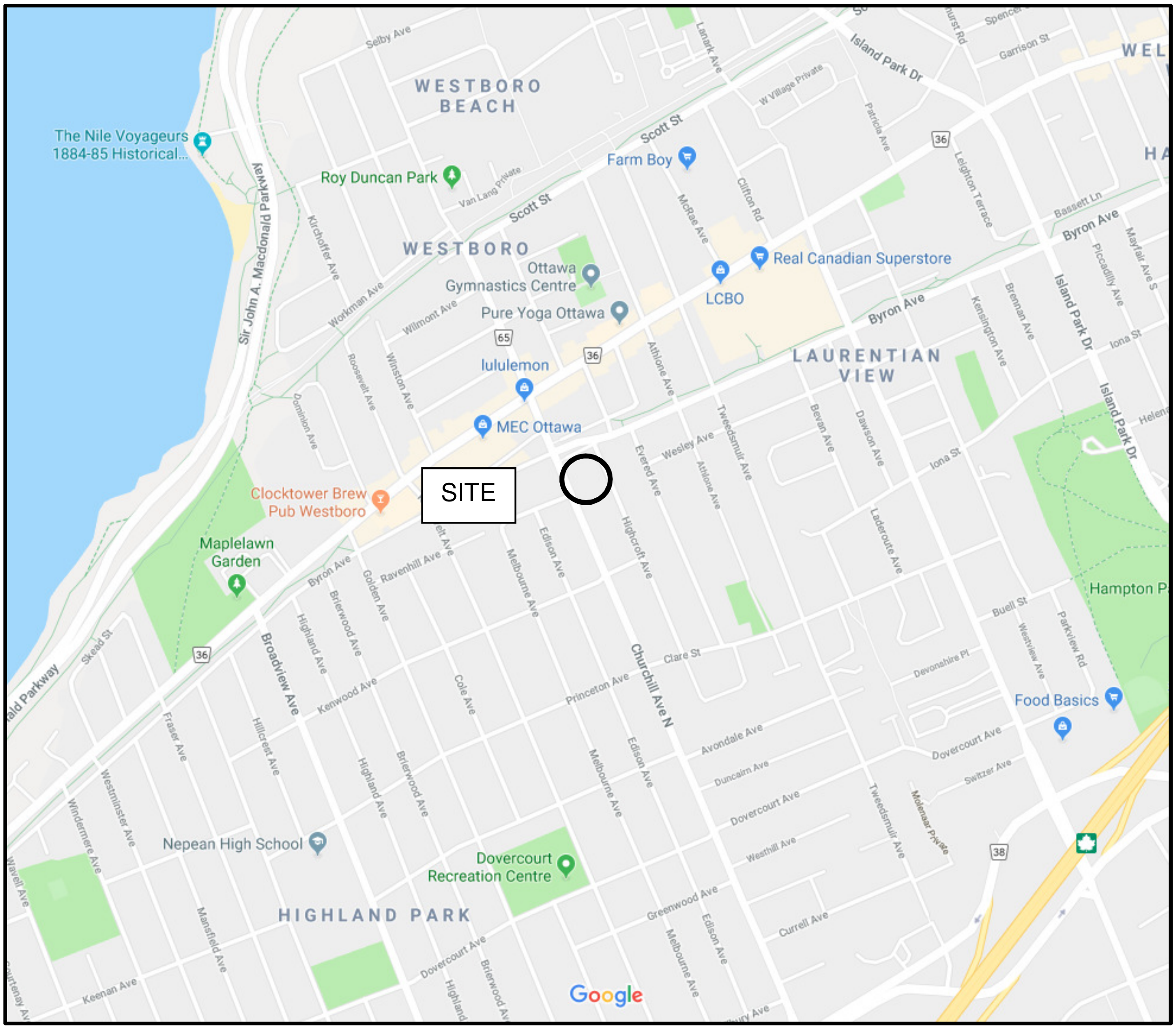
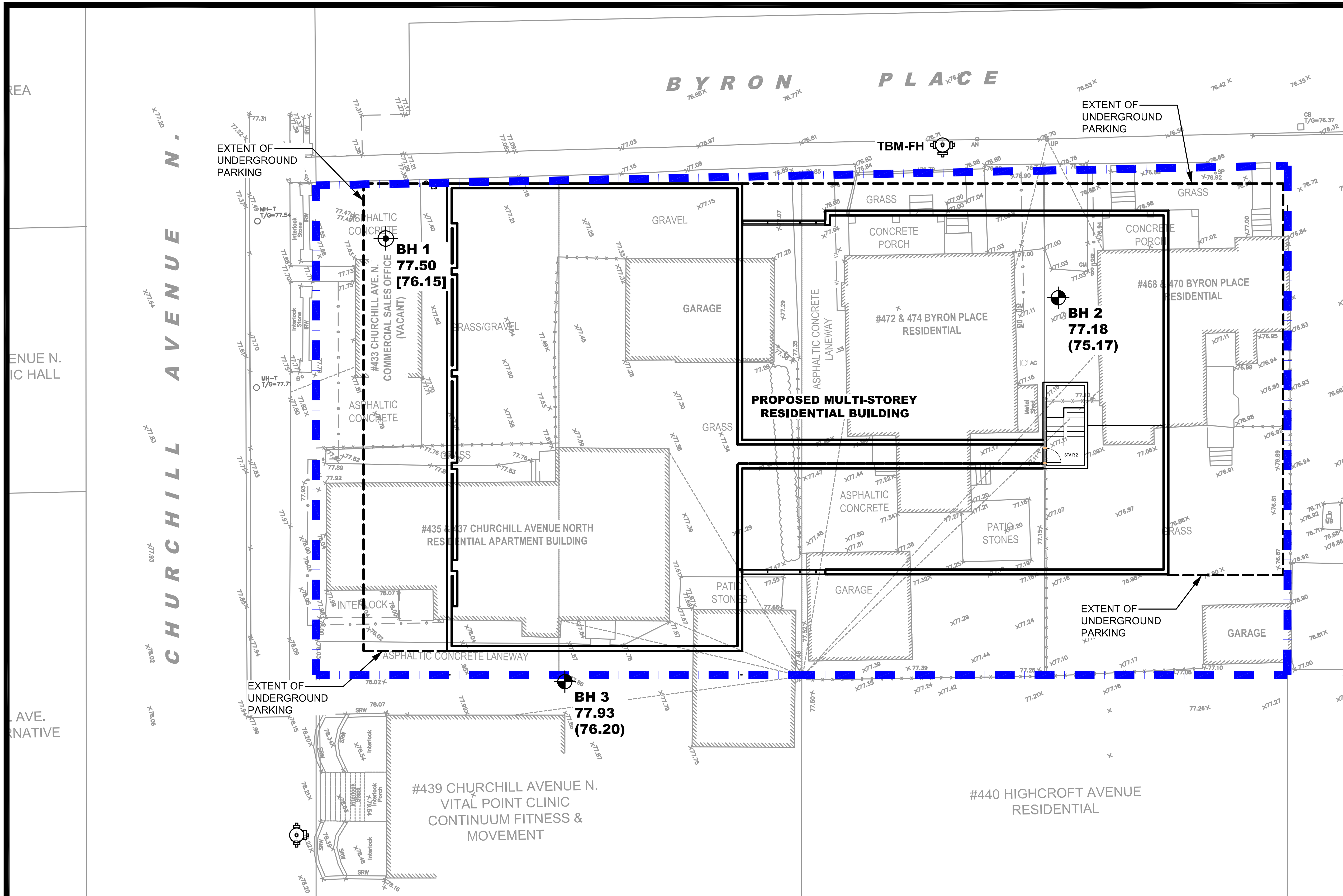


FIGURE 1

KEY PLAN



LEGEND:

(symbol)	BOREHOLE LOCATION
(symbol)	BOREHOLE WITH MONITORING WELL INSTALLED LOCATION
77.18	GROUND SURFACE ELEV(m).
[76.15]	BEDROCK SURFACE ELEV.(m)
(75.17)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP OF SPINDLE OF FIRE HYDRANT LOCATED ALONG BYRON PLACE IN FRONT OF SUBJECT SITE.
GEODETIC ELEVATION = 77.69m

BASEPLAN PROVIDED BY CHMIEL ARCHITECTS

SURVEY PLAN AND TOPOGRAPHIC MAPPING PROVIDED BY FARLEY, SMITH & DENIS SURVEYING LTD.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

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NO.	REVISIONS	DATE	INITIAL

2592532 ONTARIO INC,

GEOTECHNICAL INVESTIGATION - PROPOSED MULTI-STOREY BUILDING
433 & 435 CHURCHILL AVENUE N. AND 468 & 472 BYRON PLACE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 02/2019
Drawn by: RCG	Report No.: PG4712-1
Checked by: NC	**PG4712-1**
Approved by: DJG	Revision No.:

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