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Geotechnical Investigation

Proposed Hi-Rise Building
2829 DuMaurier Avenue
Ottawa, Ontario

Prepared for

3223701 Canada Inc.

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Report PG4928-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 3223701 Canada Inc. to conduct a geotechnical investigation for the proposed hi-rise building to be located at 2829 DuMaurier Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsurface and groundwater conditions by means of boreholes and existing soils information.

- ☐ Provide geotechnical recommendations pertaining to 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. The report contains Paterson's findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

The development is understood to consist of a 30 storey multi-use residential complex with up to five levels of underground parking with a ground floor mezzanine and commercial areas. It is further understood that the proposed building will encompass the majority of the subject site. Associated at-grade access lanes, car parking and landscaped areas are also anticipated. The proposed building is anticipated to be municipally serviced.

The subject property is presently occupied by a slab on grade commercial building. It is expected that the existing building within the north portion of the site will be demolished as part of the proposed project.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed between November 10 and 11, 2019. At that time, 3 boreholes were advanced to a maximum depth of 18.2 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development taking into consideration existing site features. The borehole locations are shown on Drawing PG4928-1 - Test Hole Location Plan included in appendix 2.

Sampling and In-Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sample 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 Paterson's 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 in Appendix 1.

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

Rock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to 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.

Groundwater

A 32 mm diameter groundwater monitoring wells were installed in all boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test hole locations were determined and located in the field by Paterson. The locations of the boreholes and the ground surface elevations for each borehole location are presented on Drawing PG4928-1 -Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and bedrock cores were recovered from the subject site and visually examined in Paterson's laboratory to review the field logs.

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

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject property is presently occupied a slab on grade commercial building with associated above ground parking structure.

The ground surface across the subject site is relatively flat and at grade with the Dumaurier Avenue. A slope was noted on the north side of the property. The residential property situated to the west of the subject site is slightly elevated. The site is bordered by high rise residential building and parking lot to the west, Dumaurier Avenue and Ramsey Crescent to the east and south, and an institutional property to the north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the boreholes consist of asphaltic concrete overlying a fill layer consisting of crushed stone with sand and trace concrete and brick. A thin layer of brown silty clay was encountered at BH1A underlying the above noted layers. Glacial till was encountered below the above noted layers consisting of a compact to a very dense silty sand with clay, gravel, cobbles, and boulders.

Bedrock

Bedrock was cored at all borehole locations. Weathered shale bedrock was encountered at depths ranging between 3.9 and 5.3 m below the existing ground surface. Upon review of the core hole samples, the upper first meter of the bedrock was found to be of fair quality.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale and dolomite of the Rockcliffe Formation. The overburden drift thickness is anticipated to be between 2 to 5 m in depth.

4.3 Groundwater

Three groundwater monitoring wells were installed as part of our geotechnical investigation. Groundwater level measurements were recorded at the monitoring well locations and our findings are presented in Table 1. It should be noted that no groundwater was encountered in any of the monitoring wells. It should also be noted that the groundwater level is subject to seasonal fluctuations. Therefore, groundwater could vary at the time of construction.

Table 1 - Groundwater Measurements at Monitoring Well Locations

Test Hole Location	Ground Surface Elevation (m)	GW Level Reading (m)	GW Level Elevation (m)	Date
BH 1	74.80	7.12	67.68	**November 20, 2019**
BH 2	73.38	5.75	67.63	**November 20, 2019**
BH 3	73.56	5.94	67.62	**November 20, 2019**

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed hi-rise building is anticipated to be founded on spread footings placed directly or indirectly by the use of a lean concrete in-filled trench on a clean, surface sounded bedrock bearing surface.

Bedrock removal may be required to complete the underground level. Hoe ramming is an option where only small quantities of bedrock need to be removed. 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 further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Since the building will occupy the entire boundaries of the subject site, all overburben will be removed to bedrock.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming. Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 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 also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

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 building structure, it is possible that it will affect the proposed basement floor slabs founded close to the shale bedrock and the basement floor slab of the adjacent hotel.

A potential for heaving and rapid deterioration of the shale bedrock exists at this site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air.

To accomplish this a 50 mm thick concrete mud slab should be placed on the exposed bedrock surface within a 48 hour period of being exposed. A 17 MPa sulphate resistant lean concrete may be used. As an alternative to the mud slab, keeping the shale surface covered with granular backfill is also acceptable.

Selected excavated vertical sides of the exposed bedrock can be protected using a sprayed elastomeric coating or shotcrete to seal the bedrock from exposure to air and dewatering.

5.3 Foundation Design

Bearing Resistance Values

Auxiliary footings placed on an undisturbed, **dense glacial till bearing surface** can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Footings placed on the **upper levels of the fractured shale and dolomite bedrock** a clean, surface sounded shale bedrock bearing 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. Where the design underside of footing is slightly above the bedrock surface, footings can be placed over concrete in-filled (17 MPa). zero entry, near vertical trenches extended to a surface sounded bedrock bearing surface using the same bearing resistance values. The concrete in-filled trenches should extend a minimum 300 mm beyond the footing faces in all directions.

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 **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5 if founded on **sound dolomite and shale 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.

Settlement

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

Soil/Bedrock Transition

It's expected that all footings will be founded on bedrock. However, between the footings for the main building and any auxiliary footings (canopy, vent shafts, etc.) where the building is founded on bedrock the auxiliary footings on the glacial till deposit, it is recommended to decrease the soil bearing capacity by 25% for the footing placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soi/bedrock and bedrock/soil transitions, it is recommended that a 2 m transition zone composed of 0.5 m layer of nominally compacted OPSS Granular A or Granular B type II be placed directly on sound bedrock. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition should be placed in the top part of the footing and foundation walls.

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 weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. However, A higher site class (**Class A**) can be achieved. The higher site class will require a site specific shear wave velocity test to be completed in confirmation of the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and a rigid pavement structure designed by a structural engineer will be applicable. 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.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.01 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.

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

Static Conditions

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 Conditions

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 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 of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

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.

It should be further noted that center to center spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

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

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

Grout to Rock Bond

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 30 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. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 64** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	30 MPa
Rock Mass Rating (RMR) - Good quality interbedded **Shale and Doloston bedrock** Hoek and Brown parameters	64 m=0.574 and s=0.00293
Unconfined compressive strength - black shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15.2 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 below.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Corehole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.1	0.7	2.8	500
	3.4	0.6	4.0	800
	4.5	0.5	5.0	1,000
125	1.3	1.1	2.4	500
	2.1	1.2	3.3	800
	2.7	1.2	3.9	1,000

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.

Bedrock Excavation Face Stabilisation

Due to the poor quality of bedrock near surface and potential founding of the proposed development, bedrock stabilization may be required when the proposed foundation extends into the shale bedrock.

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

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

5.8 Pavement Structure

For design purposes, the flexible pavement structure presented in the following table could be used for the design of car only parking areas in the lower level of the parking garage.

Table 4 - Recommended Pavement Structure - Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL 3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ silty clay or sand or crushed stone material placed over in situ soil.	

Table 5 - Recommended Pavement Structure - Local Roadways, Access Lanes and Heavy Vehicle Parking

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ silty clay or sand or crushed stone material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for parkibg areas and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

The proposed pavement structure, where it abuts the existing pavement, should match the existing pavement layers. It is recommended that a 300 mm wide and 50 mm deep stepped joint be provided where the new asphalt layer joins with the existing asphalt layer to provide more resistance to cracking at the joint.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be poured against a drainage system placed against the shoring face.

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 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. The spacing of the underfloor drainage system should be confirmed at the time of excavation when water infiltration can be better assessed. For design purposes, we suggest a 150 mm in diameter perforated pipe with a geotextile sock be placed in each bay.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

The parking garage is expected to not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may required to be insulated against the deleterious effect of frost action.

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority of the excavation to be constructed 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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. A trench box is recommended 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 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 6 - 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 $K \gamma H$ for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of $K \gamma H$ for a cantilever shoring system. H is the height of the excavation.

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

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

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

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 elevations of the existing foundations will be in close proximity to the bedrock surface (less than 1.5 m) and conventional concrete underpinning may be used to support the full width and length of the foundation.

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.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the 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 and compacted to 95% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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) Category 3 Permit to Take Water (PTTW) may be required if more than 400,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application 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 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

The analytical testing results indicate that the sulphate content is less tan 0.1%. This results indicates that Type 10 Portland Cement (i.e. normal cement) would be appropriate for this site. The chloride content and pH of the samples indicate that they are not significant factors in creating a corrosive environment, whereas the resistivity is indicative of an 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 of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed foundation drainage design and requirements.

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

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

☐ Observation of all subgrades prior to backfilling.

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

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

8.0 Statement of Limitations

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 3223701 Canada Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
December 18, 2019
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

- ❏ 3223701 Canada Inc.
- ❏ Paterson Group

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
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 11

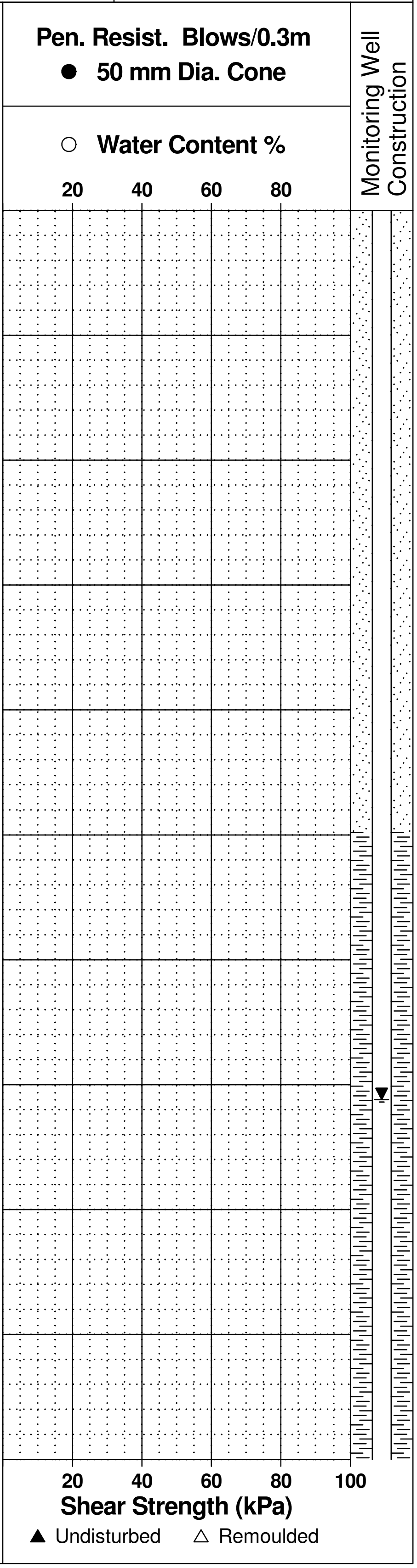
FILE NO. PG4928

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.18
FILL: Brown clayey silt, some sand and gravel	0.86
FILL: Brown silty sand	1.52
FILL: Brown sand, trace gravel	2.90
Loose, brown **FINE SAND** with silty clay	3.81
GLACIAL TILL: Dense, brown silty sand with gravel, clay and shale fragments	5.28
BEDROCK: Fair to excellent quality, green shale interbedded with dolomite	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	7
SS	3	62	5
SS	4	8	3
SS	5	75	5
SS	6	92	35
SS	7	78	50+
RC	1	100	67
RC	2	93	88
RC	3	100	84
RC	4	97	92

DEPTH (m)	ELEV. (m)
0	74.80
1	73.80
2	72.80
3	71.80
4	70.80
5	69.80
6	68.80
7	67.80
8	66.80
9	65.80
10	64.80



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 11

FILE NO. PG4928

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
BEDROCK: Fair to excellent quality, green shale interbedded with dolomite	RC	5	97	92
	RC	6	100	98
13.51				
BEDROCK: Good excellent quality, grey dolostone	RC	7	100	93
	RC	8	92	74
	RC	9	95	85
18.19				
End of Borehole (GWL @ 7.12m - Nov. 20, 2019)				

DEPTH (m)	ELEV. (m)
10	64.80
11	63.80
12	62.80
13	61.80
14	60.80
15	59.80
16	58.80
17	57.80
18	56.80

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
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

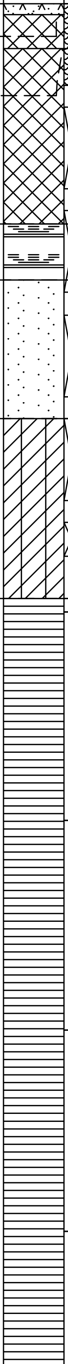
BORINGS BY CME 55 Power Auger

DATE 2019 November 11

FILE NO. PG4928

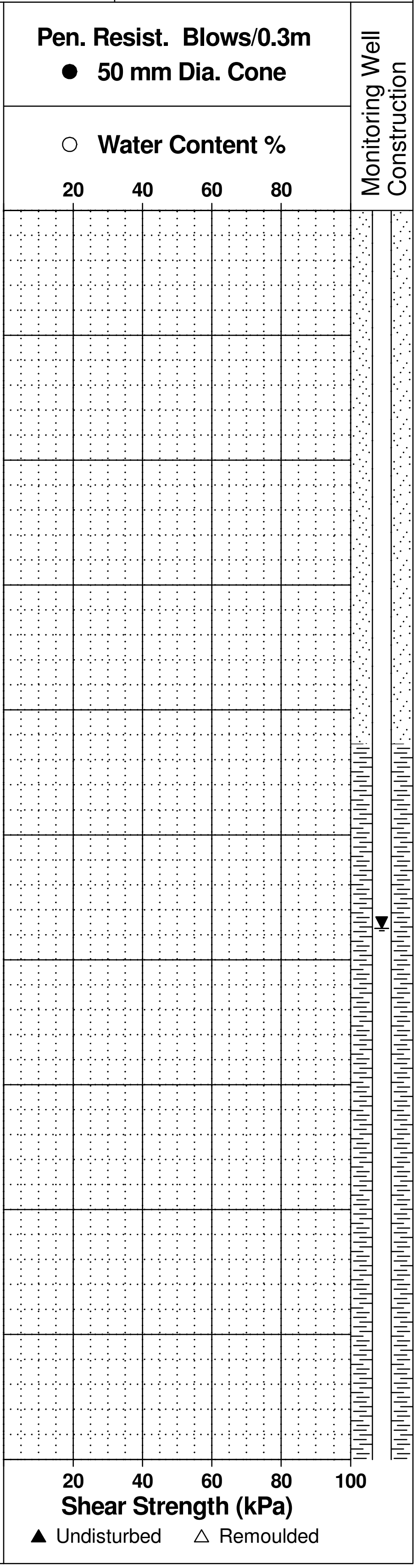
HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown sand with gravel, crushed stone	0.33
FILL: Brown silty sand, trace clay	1.62
PEAT	2.03
Loose, grey **FINE SAND** with silt and clay	3.05
Grey **SILTY CLAY,** trace sand	4.37
BEDROCK: Excellent quality, green shale interbedded with dolostone	



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	25	15
SS	3	62	4
SS	4	62	6
SS	5	75	2
SS	6	0	50+
RC	1	100	0
RC	2	100	100
RC	3	98	89
RC	4	100	97
RC	5	100	100

DEPTH (m)	ELEV. (m)
0	73.38
1	72.38
2	71.38
3	70.38
4	69.38
5	68.38
6	67.38
7	66.38
8	65.38
9	64.38
10	63.38



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

BORINGS BY CME 55 Power Auger

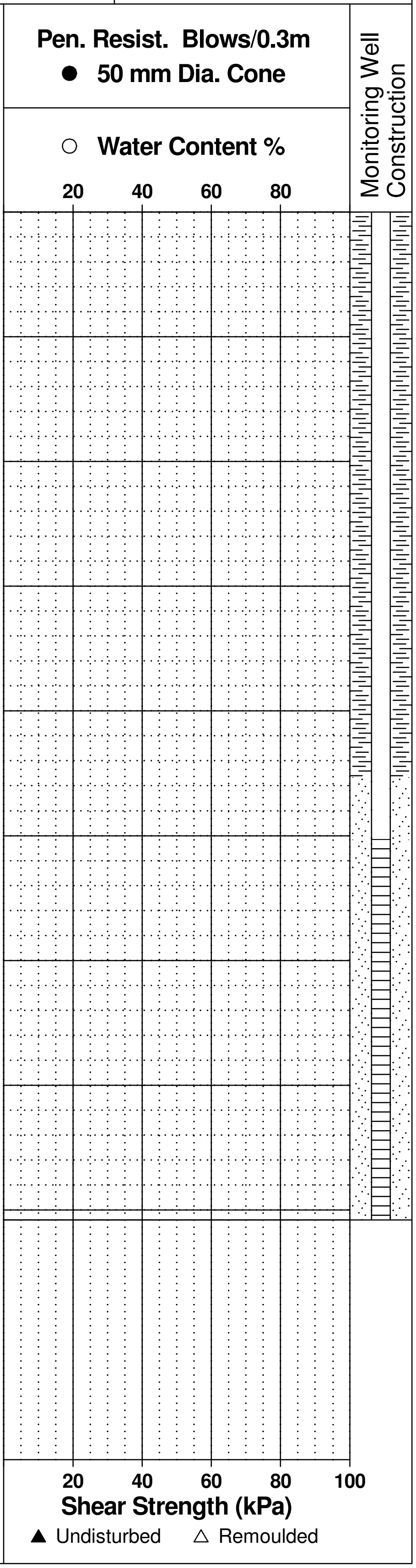
DATE 2019 November 11

FILE NO. PG4928

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
BEDROCK: Excellent quality, green shale interbedded with dolostone	RC	6	85	80
	RC	7	100	95
13.49				
BEDROCK: Good to excellent quality, grey dolostone	RC	8	95	88
	RC	9	100	98
	RC	10	98	98
18.08				
End of Borehole (GWL @ 5.75m - Nov. 20, 2019)				

DEPTH (m)	ELEV. (m)
10	63.38
11	62.38
12	61.38
13	60.38
14	59.38
15	58.38
16	57.38
17	56.38
18	55.38



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

BORINGS BY CME 55 Power Auger

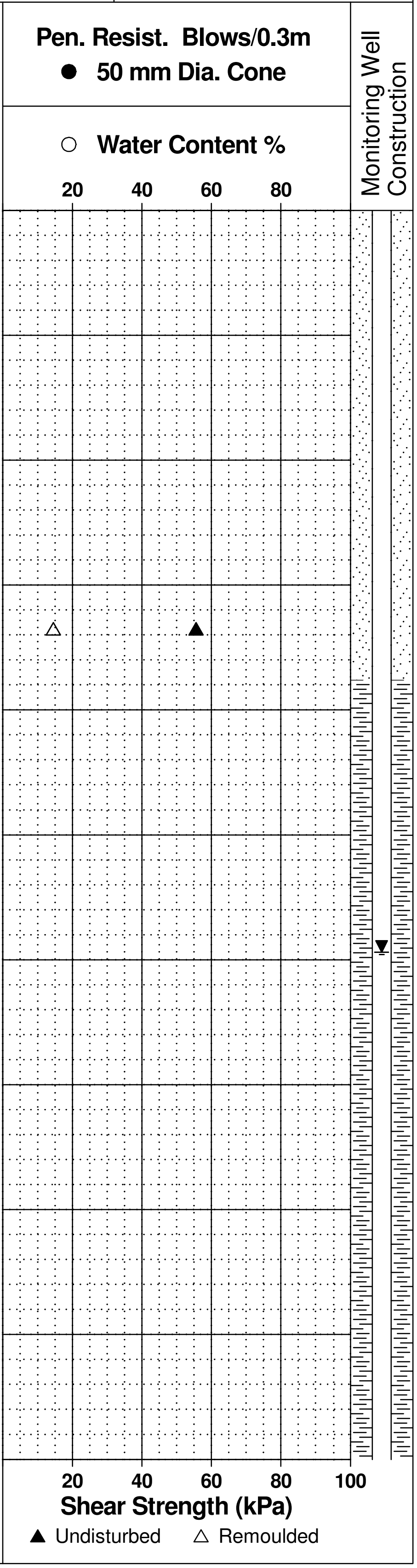
DATE 2019 November 11

FILE NO. PG4928

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown sand with gravel	0.28	AU	1		
FILL: Brown silty clay, trace sand	0.76				
FILL: Dark brown silt, some sand	0.97				
FILL: Grey-brown silty clay, some organics, trace sand		SS	2	50	4
		SS	3	0	3
	3.05	SS	4	62	1
GLACIAL TILL: Grey sandy silt, some clay and gravel	3.86	SS	5	83	P
BEDROCK: Good quality, green shale interbedded with dolostone		SS	6	100	50+
		RC	1	100	88
		RC	2	96	88
		RC	3	98	93
		RC	4	100	100
		RC	5	98	88

DEPTH (m)	ELEV. (m)
0	73.56
1	72.56
2	71.56
3	70.56
4	69.56
5	68.56
6	67.56
7	66.56
8	65.56
9	64.56
10	63.56



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2829 Dumaurier Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the northeast corner of subject site. Geodetic elevation = 74.952m.

REMARKS

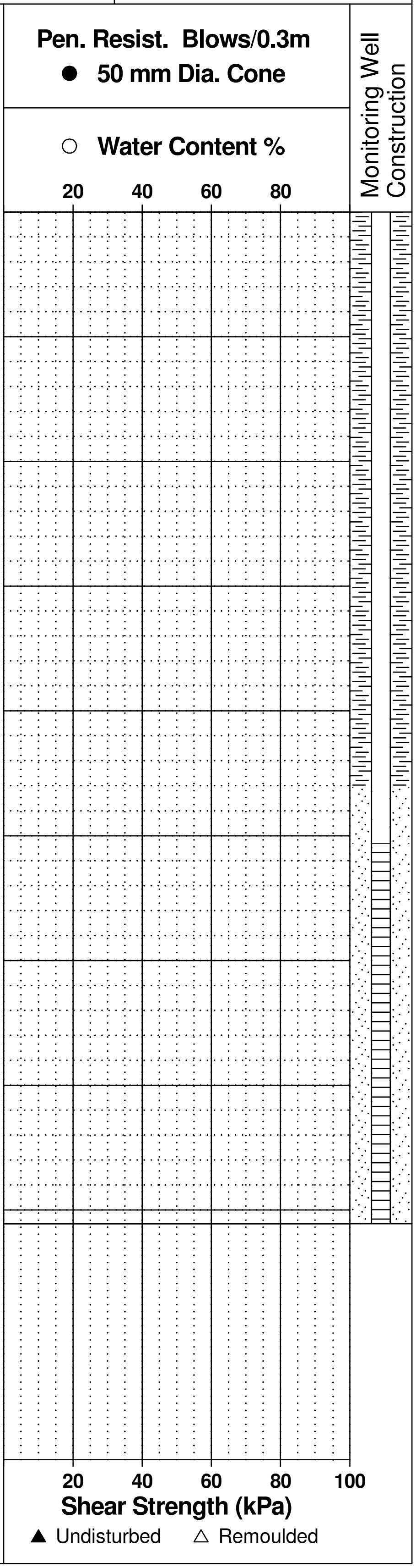
BORINGS BY CME 55 Power Auger

DATE 2019 November 11

FILE NO. PG4928

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						10	63.56
BEDROCK: Good quality, green shale interbedded with dolostone		RC	6	98	81	11	62.56
12.20						12	61.56
		RC	7	90	57	13	60.56
		RC	8	100	92	14	59.56
BEDROCK: Fair to excellent quality, grey dolostone						15	58.56
		RC	9	95	95	16	57.56
		RC	10	97	97	17	56.56
18.11						18	55.56
End of Borehole (GWL @ 5.94m - Nov. 20, 2019)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the 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)

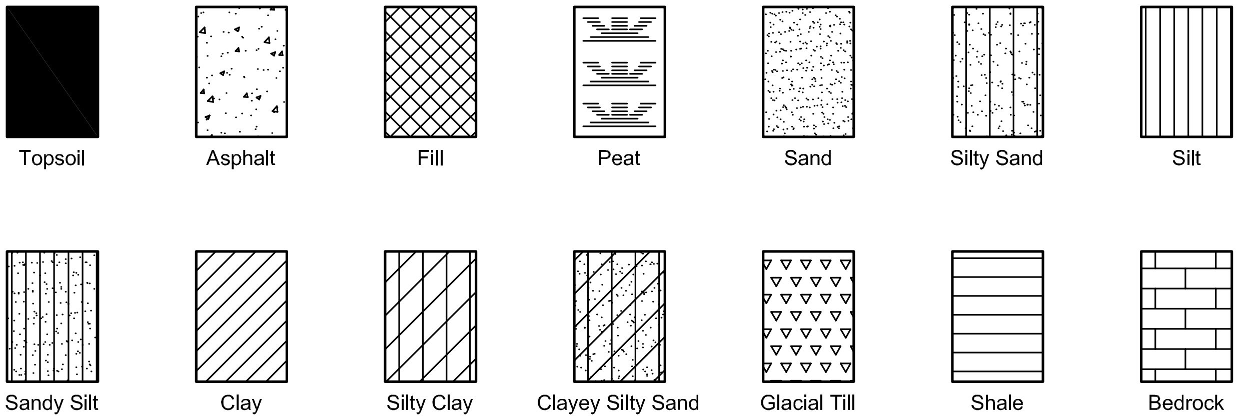
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

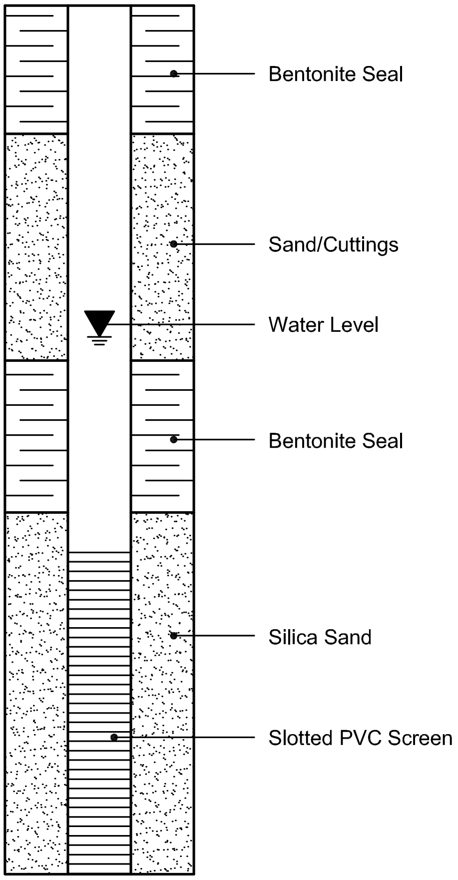
SYMBOLS AND TERMS (continued)

STRATA PLOT

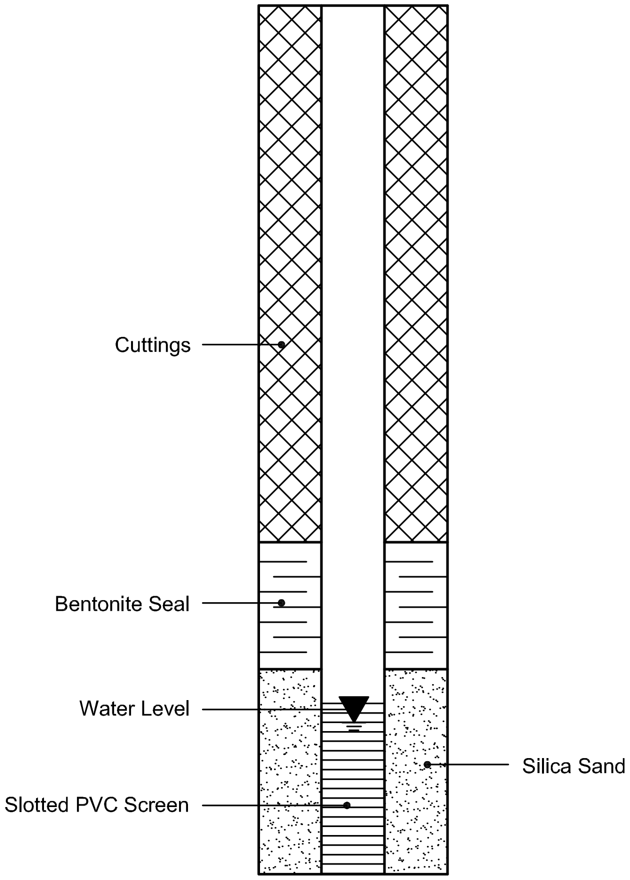


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29072

Report Date: 14-Nov-2019

Order Date: 12-Nov-2019

Project Description: PG4928

	Client ID:	BH1-SS3, 5'-7'	-	-	-
	Sample Date:	11-Nov-19 11:00	-	-	-
	Sample ID:	1946129-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.34	-	-	-
Resistivity	0.10 Ohm.m	15.8	-	-	-
Anions					
Chloride	5 ug/g dry	333	-	-	-
Sulphate	5 ug/g dry	21	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4928-1 - TEST HOLE LOCATION PLAN

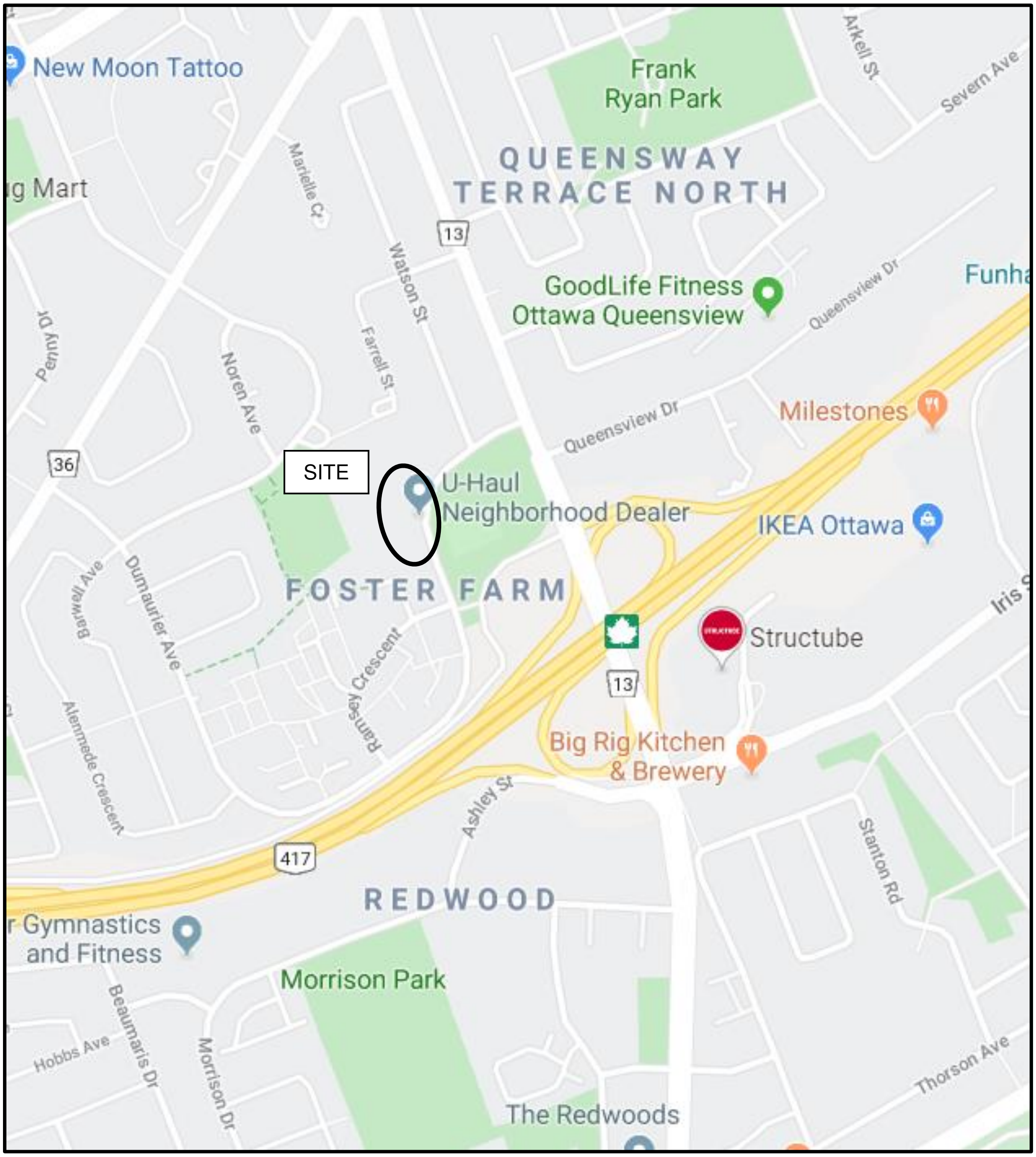
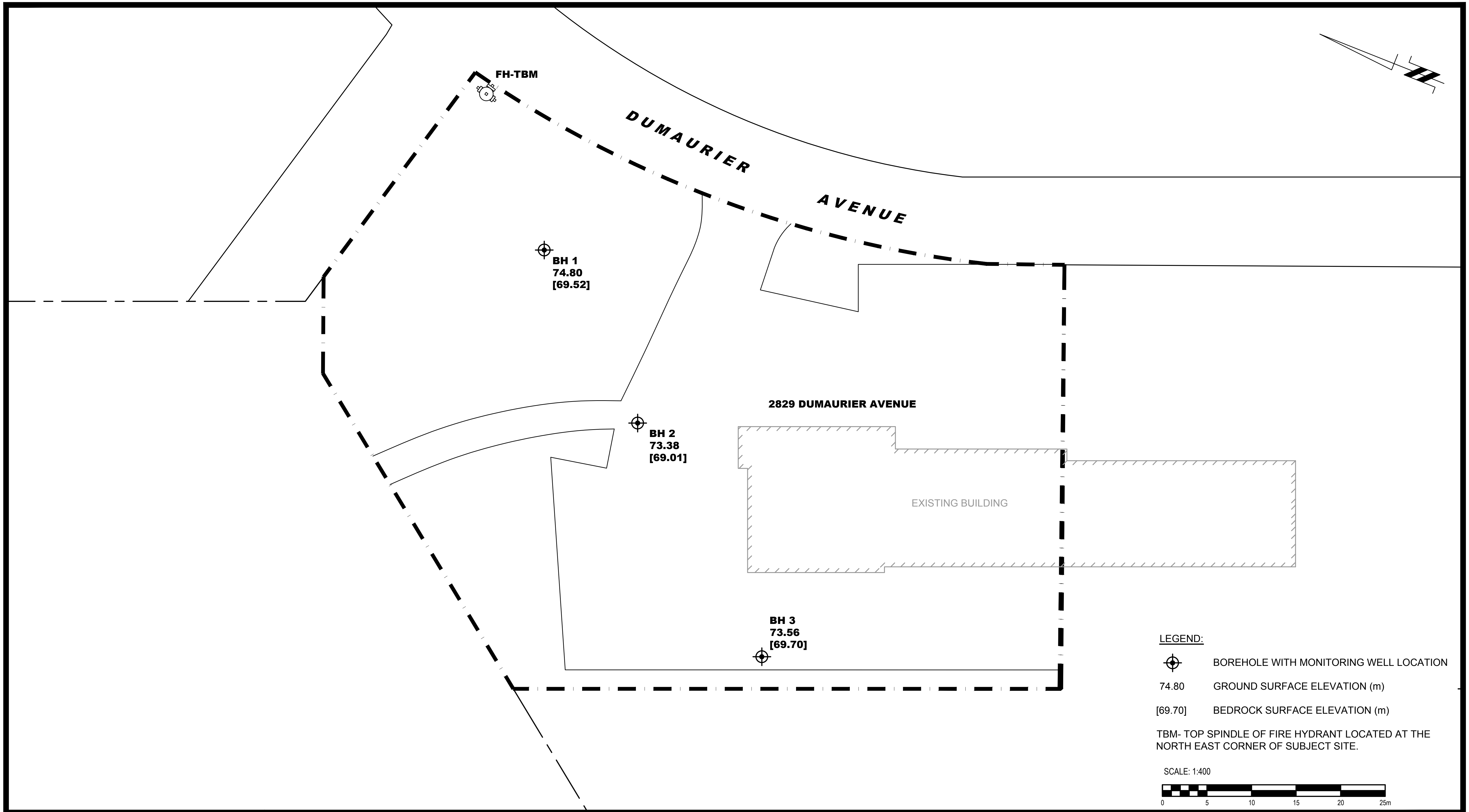


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
74.80	GROUND SURFACE ELEVATION (m)
[69.70]	BEDROCK SURFACE ELEVATION (m)

TBM- TOP SPINDLE OF FIRE HYDRANT LOCATED AT THE NORTH EAST CORNER OF SUBJECT SITE.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

3223701 CANADA INC.

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDING-2829 DUMAURIER AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 11/2019
Drawn by: YA
Report No.: PG4928-1
Checked by: JV
Dwg. No.: PG4928-1
Approved by: DJG
Revision No.:

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