

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Residential Building
263 Greensway Avenue
Ottawa, Ontario

Prepared For

Manor Park Management

Paterson Group Inc.

Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

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

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Report PG4461-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Manor Park Management to conduct a geotechnical investigation for the proposed multi-storey building to be located at 263 Greensway Avenue 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 subsurface soil, bedrock and groundwater conditions 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 findings and includes geotechnical recommendations pertaining to the design and construction of the residential development as understood at the time of writing this report.

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

2.0 Proposed Development

It is understood that the proposed development will consist of a six storey building with two (2) levels of underground parking. Associated paved parking areas, access lanes and landscaped areas are also anticipated as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the present investigation was conducted on April 10 and 11, 2018. At that time, seven (7) boreholes were advanced to a maximum depth of 8.5 m. The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the proposed building taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG4461-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a truck mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson field personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of hollow stem augering to the required depths at select locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are 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 completed at five locations during the current investigation, BH 1 to BH 5, to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Groundwater monitoring wells were installed in BH 1, BH 4, and BH 5 to permit the monitoring of water levels subsequent to the completion of the sampling program. Flexible piezometers were installed in the remaining boreholes to permit the monitoring of water levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the test hole locations were referenced to a job benchmark (TBM), consisting of a mag nail located along the concrete curb at the northeast corner of the site. A geodetic elevation of 56.296 m was provided for the TBM on the survey plan prepared by Fairhall, Moffatt and Woodland Ltd..

The location and ground surface elevation at each test hole location is presented on Drawing PG4461-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two storey building with one basement level towards the east portion of the site. The remainder of the site consists of paved car parking areas, an access lane and landscaped areas. The ground surface is relatively flat and slightly above the grade of Greensway Avenue.

The site is bordered to the north by residential properties, to the east by Greensway Avenue, to the west by Vanier Parkway and to the south by a commercial property.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of a pavement structure and varying fill material overlying a brown to grey silty clay layer. The silty clay layer is generally underlain by a glacial till layer consisting of silty sand to silty clay mixed with gravel, cobbles and boulders. The glacial till layer is followed by interbedded limestone and shale bedrock. Bedrock and/or practical refusal to augering was encountered at all borehole locations, at depths ranging between 3.3 m to 3.9 m. Based on the RQD values, the bedrock quality varies between poor to excellent quality depending on location and depth.

Based on available geological mapping, bedrock in the area of the subject site consists of shale of the Billings Formation. The overburden thickness is estimated to be between 3 to 5 m depth.

4.3 Groundwater

Groundwater monitoring wells were installed at BH 1, BH 4, and BH 5 and flexible piezometers were installed in the remaining boreholes to measure groundwater levels. The following table presents our groundwater level measurements. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Levels		
Borehole Number	Measured Groundwater Depth (m)	Date
BH 1-18	3.82	April 20, 2018
BH 2-18	piezometer damaged	April 20, 2018
BH 3-18	3.69	April 20, 2018
BH 4-18	3.68	April 20, 2018
BH 5-18	3.66	April 20, 2018
BH 3-18	3.12	April 20, 2018
BH 7-18	2.11	April 20, 2018

5.0 Discussion

5.1 Geotechnical Assessment

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

Bedrock removal will be required to complete the lower underground parking level. 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

Based on the bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

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

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. A minimum of 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.

Vibration Considerations

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

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

Two parameters determine the recommended vibration limit; the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

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 will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

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

5.3 Foundation Design

Footings placed on a clean, surface sounded bedrock surface could be designed for 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 should not contain surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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 is **Class C** for the shallow foundations at the subject site. A higher seismic site class, such as Class A or B is available for the subject site. However, a site specific seismic shear wave test is required to provide the higher site classes according to the 2012 Ontario Building Code. Soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed for the proposed building and the basement floor slab will be founded on a granular pad placed over a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of a 19 mm clear crushed stone.

A sub-floor drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Walls

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

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3 . The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3 , where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

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

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

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

5.7 Pavement Structure

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

Table 2 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
	SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

Table 3 - Recommended Pavement Structure Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
	SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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 structure. It is expected that insufficient room will be available for exterior backfill below the bedrock surface. It is suggested that this system could be as follows:

- Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas or alternative material approved by the geotechnical consultant to fill in cavities and smooth out angular features at the bedrock surface)
- composite drainage layer

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 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

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 100 or 150 mm diameter perforated pipes be placed at 3 to 4.5 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

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

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

6.3 Excavation

Temporary Side Slopes

The temporary excavation side slopes anticipated 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 cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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.

Temporary Shoring

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

The 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. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method. The earth pressures acting on the shoring system may be calculated with the following parameters.

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

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

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

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

6.4 Pipe Bedding and Backfill

A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. Other subgrades anticipated at the subject site, such as the brown silty clay or glacial till layers, should have a minimum of 150 mm of OPSS Granular A placed for the bedding for sewer or water 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 pipe obvert should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to existing groundwater level and inferred depths of the proposed footings, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the excavations.

Permit to Take Water

A temporary Ministry of the Environment and Climate Change (MOECC) 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 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

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.

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, it is expected that groundwater flow will be low (i.e. - less than 30,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, the groundwater level is anticipated at a 3 to 3.5 m depth and close to the bedrock or within the glacial till layer. Therefore, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within the native glacial till and/or directly over a bedrock bearing surface. Due to the proximity of the neighbouring structures and since the long-term groundwater level is below the silty clay layer, no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

If excavations are completed in proximity of existing structures, these structures may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of a non-aggressive to slightly aggressive environment for exposed ferrous metals at this site.

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.
- 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 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 grading plan, drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Manor Park Management, or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Colin Belcourt, M.Eng.



David J. Gilbert, P.Eng.



Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL RESULTS

DATUM TBM - Set magnetic nail and washer on concrete (refer to Test Hole Location Plan for location). Geodetic elevation = 56.296m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 10, 2018

FILE NO.
PG4461

HOLE NO.
BH 2-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
25mm Asphaltic concrete over crushed stone with silt and sand	0.15	AU	1			0	56.16						
FILL: Brown silty sand, trace gravel, ash, coal and construction debris		SS	2	33	2	1	55.16						
	1.45												
Brown SILTY CLAY , trace sand		SS	3	79	9	2	54.16						
	1.98												
GLACIAL TILL: Brown silty sand, some clay, gravel, cobbles and boulders		SS	4	50	31	3	53.16						
		SS	5	50	50+	3	53.16						
	3.53												
		RC	1	100	42	4	52.16						
		RC	2	100	96	5	51.16						
BEDROCK: Interbedded limestone and shale		RC	3	96	86	6	50.16						
		RC	4	100	95	7	49.16						
						8	48.16						
	8.56												
End of Borehole													

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

DATUM TBM - Set magnetic nail and washer on concrete (refer to Test Hole Location Plan for location). Geodetic elevation = 56.296m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 11, 2018

FILE NO.
PG4461

HOLE NO.
BH 5-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
Asphaltic concrete	0.06					0	56.25						
FILL: Crushed stone with silt and sand	0.17	AU	1										
FILL: Brown silty sand, trace clay, gravel, brick and ash		SS	2	100	13	1	55.25						
		SS	3	46	8	2	54.25						
	2.21												
Grey SILTY CLAY , trace sand	2.74	SS	4	50	6								
		SS	5	38	15	3	53.25						
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders	3.94	SS	6	80	50+	4	52.25						
		RC	1	100	91	5	51.25						
BEDROCK: Interbedded limestone and shale		RC	2	100	87	6	50.25						
	6.86												
End of Borehole (GWL @ 3.66m - April 20, 2018)													

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
D _{xx}	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

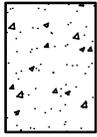
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.
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SYMBOLS AND TERMS (continued)

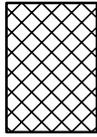
STRATA PLOT



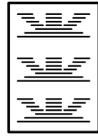
Topsoil



Asphalt



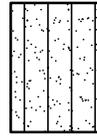
Fill



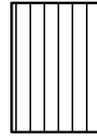
Peat



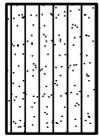
Sand



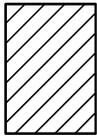
Silty Sand



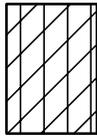
Silt



Sandy Silt



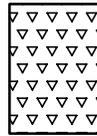
Clay



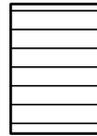
Silty Clay



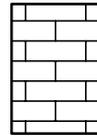
Clayey Silty Sand



Glacial Till



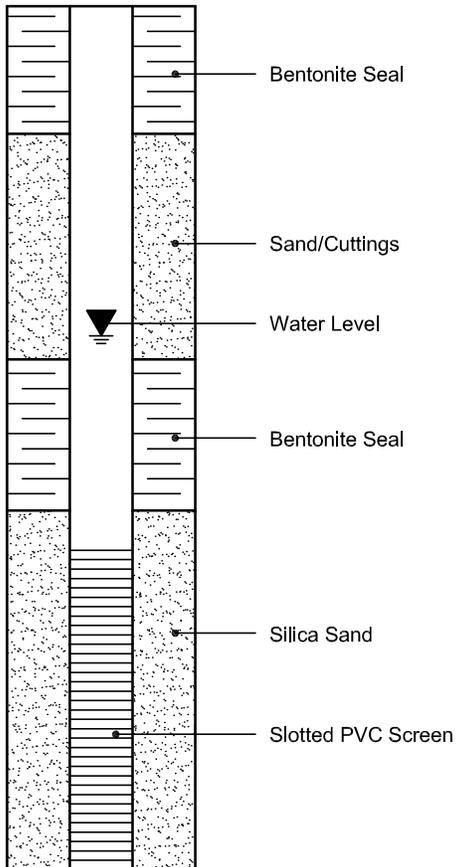
Shale



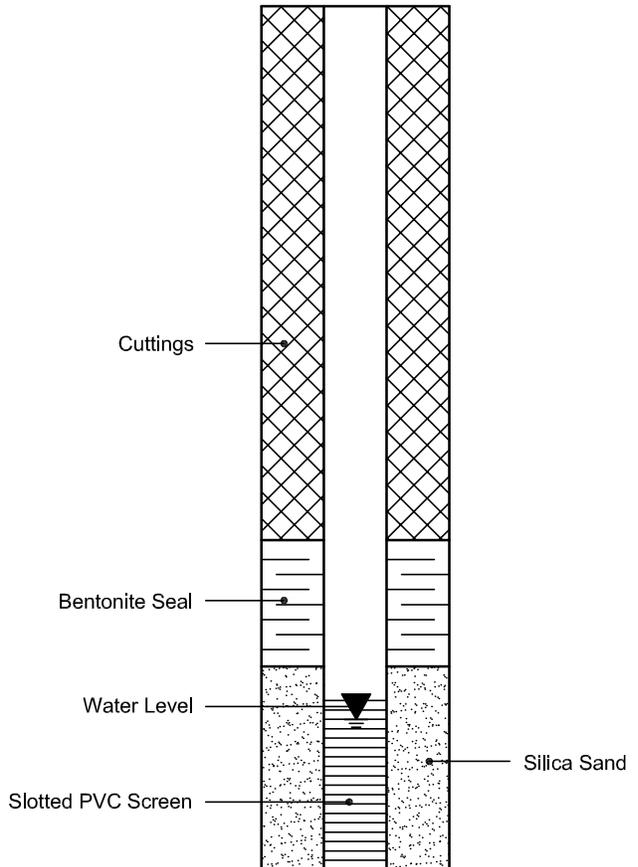
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
 Client: Paterson Group Consulting Engineers
 Client PO: 23292

Report Date: 18-Apr-2018

Order Date: 12-Apr-2018

Project Description: PG4461

Client ID:	BH6-18-SS5	-	-	-
Sample Date:	04/11/2018 09:00	-	-	-
Sample ID:	1815440-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	92.7	-	-	-
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General Inorganics

pH	0.05 pH Units	7.92	-	-	-
Resistivity	0.10 Ohm.m	41.6	-	-	-

Anions

Chloride	5 ug/g dry	44	-	-	-
Sulphate	5 ug/g dry	79	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4461-1 - TEST HOLE LOCATION PLAN

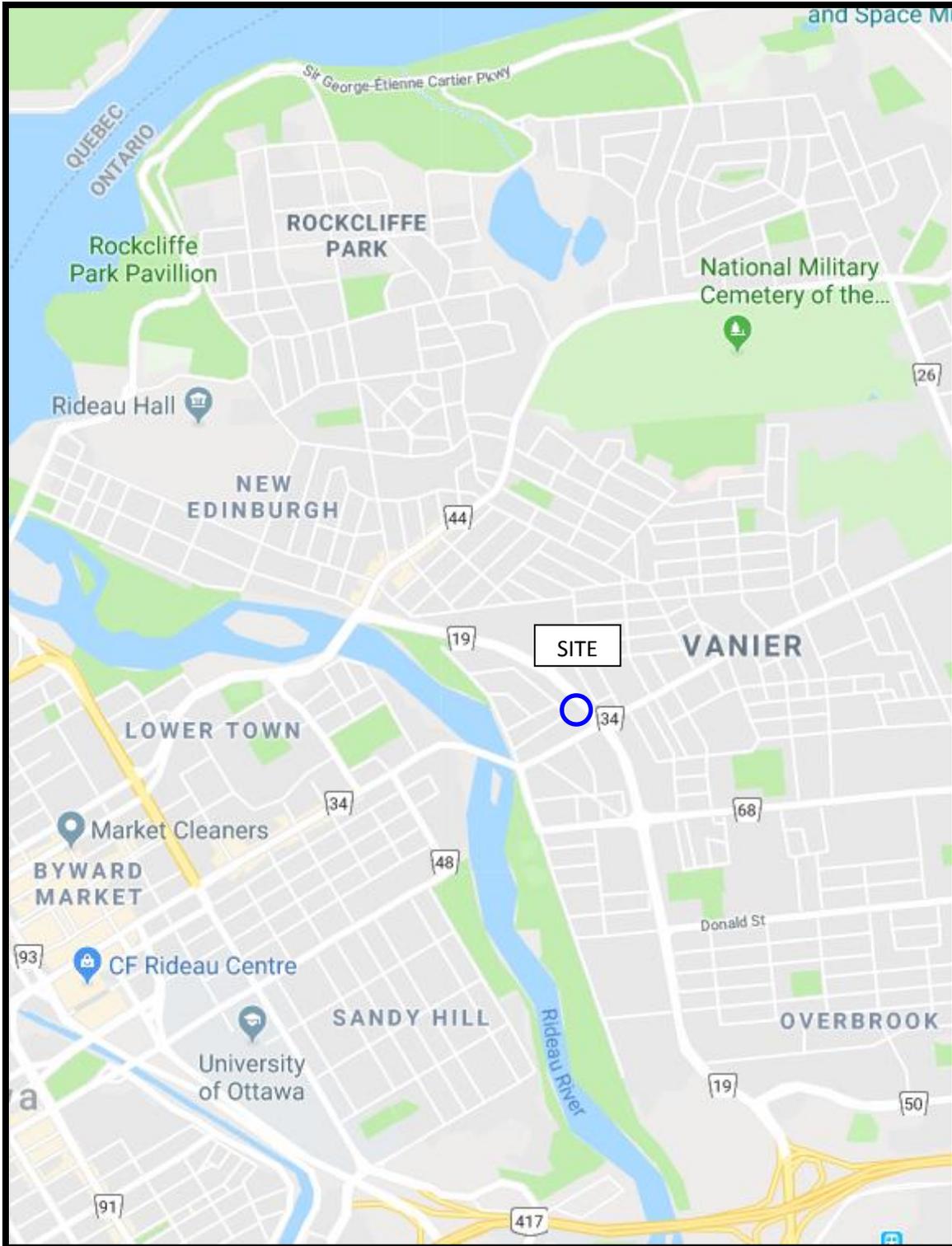
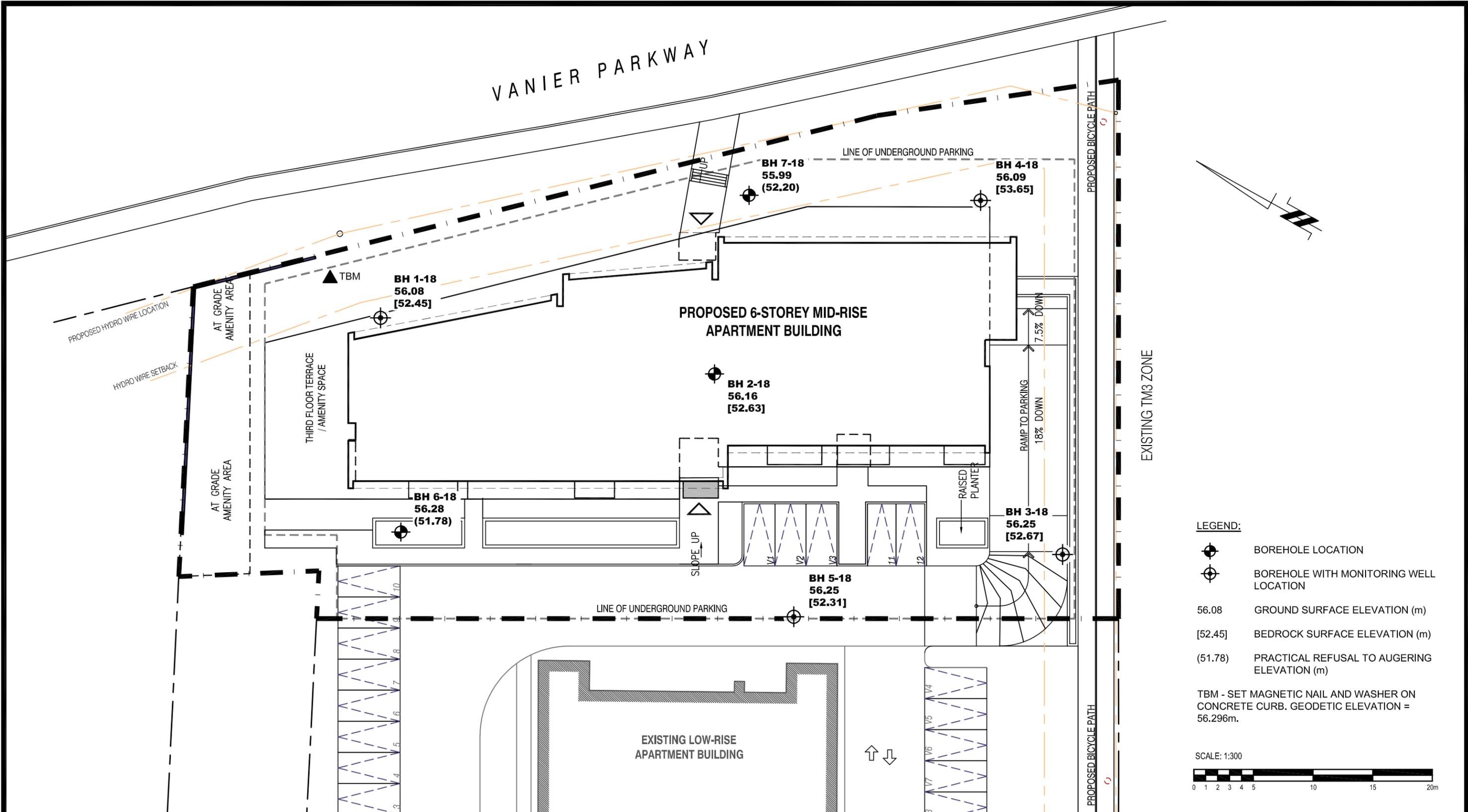
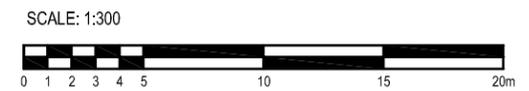


FIGURE 1
KEY PLAN



- LEGEND:**
- BOREHOLE LOCATION
 - BOREHOLE WITH MONITORING WELL LOCATION
 - 56.08 GROUND SURFACE ELEVATION (m)
 - [52.45] BEDROCK SURFACE ELEVATION (m)
 - (51.78) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
- TBM - SET MAGNETIC NAIL AND WASHER ON CONCRETE CURB. GEODETIC ELEVATION = 56.296m.



patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

MANOR PARK MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 263 GREENSWAY AVENUE

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:300	Date:	05/2018
Drawn by:	MPG	Report No.:	PG4461-1
Checked by:	CB	Dwg. No.:	PG4461-1
Approved by:	DJG	Revision No.:	0

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