patersongroup

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation
Proposed Multi-Storey Building
78, 84, 86 and 90 Beechwood Avenue and 69, 73, 77, 83, 85, 89 and 93 Barrette Street Ottawa

Prepared For

Minto Communities Inc.

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

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

December 15, 2020

Report: PG4938-1 Revision 2

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Minto Communities Inc. (Minto) to conduct a geotechnical investigation for the proposed multi-storey building which will occupy the land between 78 through 90 Beechwood Avenue and 66 through 93 Barrette Street, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- ☐ determine the subsurface soil and groundwater conditions by means of boreholes.

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

The following report has been prepared specifically and solely for the aforementioned project which is described herein. The report contains the geotechnical findings and includes recommendations pertaining to the design and construction of the subject 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.

2.0 Proposed Development

The proposed development consists of three (3) multi-storey buildings with up to ten (10) storeys and two (2) underground parking levels. Associated access lanes, parking and landscaped areas are also anticipated as part of the development. The proposed development is anticipated to be municipally services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed on May 23 to 24 and July 15, 2019. At that time, a total of 10 boreholes were drilled to a maximum depth of 9.1 m below existing grade. A supplemental field program was completed August 24, 2020 which consisted of BH 11 through BH 14. At that time, a total of 4 boreholes were drilled to a maximum depth of 5.8 m. The test hole locations were selected and distributed in a manner to provide general coverage of the proposed subject site taking into consideration site features. Reference should be made to the Soil Profile and Test Data Sheets attached in Appendix 1.

The boreholes were completed using a low-clearance rubber-track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from our geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the auger flights. All soil samples were visually inspected and initially classified on site. The split-spoon samples were placed in sealed plastic bags. Sampling and testing the overburden was completed in general accordance with ASTM D5434-12 - Guide for Field Logging of Subsurface Explorations of Soil and Rock. Transportation of the samples was completed in general accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soil Samples.

The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Sample Storage

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

3.2 Field Survey

The test holes completed during the current field investigation were located in the field by Paterson personnel. The borehole locations were referenced to a temporary benchmark (TBM) consisting of a mag nail on the sidewalk at the south of Beechwood Avenue and east of St. Charles Street. A geodetic elevation of 59.29 m was provided for the TBM. The borehole locations and ground surface elevation at each test hole location are presented on Drawing PG4938-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. The subsurface soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

4.0 Observations

4.1 Surface Conditions

The subject site consists of 11 contiguous properties which are currently occupied by existing, mixed-use two (2) storey slab-on-grade buildings and associated asphalt or gravel parking areas. The neighbouring properties consist of a combination of commercial and residential properties. The ground surface across the subject property is at grade with the surrounding roadways and properties.

The subject site is bordered to the north by Beechwood Avenue, to the south by Barrette Street, and to the east and west by existing residential dwellings and/or commercial buildings.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a pavement structure or crushed stone overlying a brown silty sand fill material with some gravel. The fill was observed to extend to depths ranging from 0.3 to 3.1 m, generally increasing in depth from west to east across the subject site.

A compact to very dense glacial till deposit was encountered underlying the fill at the test hole locations, with the exception of BH 5 and BH 7. The glacial till consists of a brown clayey silt to silty sand with gravel, cobbles and boulders, and was observed to extend to the surface of the weathered shale bedrock at approximate depths ranging from 1.5 to 6.9 m below the existing ground surface.

Practical refusal to augering was encountered within the weathered shale bedrock at depths ranging from 3.3 to 6.9 m. However, within the northeast portion of the site, refusal was not encountered within the weathered shale bedrock at depths of up to 9.1 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock at the site consists of shale of the Billings formation with an anticipated overburden thickness of 1 to 3 m depth.

4.3 Groundwater

Monitoring wells were installed in 8 boreholes to permit the monitoring of groundwater levels subsequent to the completion of the sampling program. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations and groundwater measurements within the monitoring wells, it is estimated that the long-term groundwater table can be expected at approximately 5 to 6 m below ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 2	57.94	3.92	54.02	June 3, 2019
BH 3	58.66	6.60	52.06	June 3, 2019
BH 4	58.43	5.00	53.43	June 3, 2019
BH 7	58.59	4.50	54.09	July 19, 2019
BH 8B	58.67	6.96	51.71	July 19, 2019
BH 9	58.71	6.15	52.56	July 19, 2019
BH 10	58.66	5.48	53.18	July 19, 2019
BH 11	58.11	4.28	53.83	September 3, 2020
BH 13	58.73	3.20	55.53	September 3, 2020

Note: The ground surface elevations at the borehole locations were surveyed by Paterson based on a geodetic elevation consisting of a mag nail installed within the sidewalk south of Beechwood Avenue and east of St. Charles Street.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed multi-storey building. The building is anticipated to be founded on shallow footings placed on an undisturbed weathered bedrock, and/or clean, surface sounded bedrock bearing surface.

Expansive shale could be present on site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock. This is discussed further in Section 6.7.

Bedrock removal will be required to complete the underground parking levels. 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

Topsoil and deleterious fill, such as those containing organics, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Due to the anticipated number of underground parking levels and depth of the bedrock at the subject site, it is anticipated that all existing overburden material will be excavated from within the proposed building footprint and also from the underground parking area. Bedrock removal will be required for the construction of the underground parking area.

Bedrock Removal

As noted above, bedrock removal can be accomplished by hoe ramming where only a 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 excavated with almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden/weathered bedrock excavation and the sound bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden/weathered bedrock 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 a cooperative environment with the residents.

The following construction equipment 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 to be completed to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

Bedrock stabilization may be required where the proposed foundation extends into the sound bedrock.

Rock anchors and rock face protection 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 rock face protection and rock anchors within the sound bedrock should 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. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of standard Proctor maximum dry density (SPMDD). It is recommended that a well graded material with a maximum diameter of 300 mm be used.

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Excavated shale deteriorates upon exposure to air and is not generally suitable for re-use as an engineered fill. The use of imported granular fill is recommended.

5.3 Foundation Design

Bearing Resistance Values

Footings placed over a clean, surface sounded weathered shale bedrock surface can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **500 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed over a clean, surface sounded shale bedrock surface can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

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, such as Class A or B may be provided if a site specific shear wave velocity test is completed to confirm 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

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. The compaction efforts should be completed under dry conditions and above freezing temperatures and be approved by Paterson personnel at the time of construction.

5.6 Basement Walls

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. However, 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 $P_o = 0.5 \cdot K_o \cdot \gamma \cdot H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. 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 individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed. Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

Each anchor be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

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

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project, where required, are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of good quality shale ranges between about 50 and 80 MPa, which is stronger than most routine grouts.

A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 3.

Table 3 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Shale	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 4. The factored tensile resistance values given in Table 4 are based on a single anchor with no group influence effects.

A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2	0.8	2.8	450
	2.6	1	3.6	600
	3.2	1.2	4.4	750
	4.5	2	6.5	1000
125	1.6	0.6	2.2	600
	2	1	3	750
	2.6	1.4	4	1000
	3.2	1.8	5	1250

Other considerations

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

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

5.8 Pavement Structure

For design purposes, the following pavement structures presented below are recommended for the design of pedestrian walkways and car parking areas and access lanes.

Table 5 - Recommended Hard Landscaping - Pedestrian Walkways

Thickness (mm)	Material Description
Specified by Others	**Wear Course** - Interlocking Stones/Brick Pavers
25 - 40	**Leveling Course** - Stone Dust or Sand
300	**SUBBASE** - OPSS Granular A
SUBGRADE - Either concrete podium slabs, fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 6 - 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 - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 7 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated 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 100% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is understood that insufficient room is available for exterior backfill. Therefore, the foundation drainage system should consist of a composite drainage layers placed directly against the prepared bedrock surface and/or temporary shoring system.

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

A groundwater infiltration suppression system should also be provided for any elevator shaft and sump pump pits (pit bottom and walls) located within the lowest basement level.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration due to groundwater infiltration at the proposed founding elevation. For preliminary design purposes, Paterson recommends that 150 mm in diameter perforated pipes be placed at approximately 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection Against Frost Action

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

A minimum thickness of 2.1 m of soil cover (or equivalent) should be provided for other exterior unheated footings. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action and should be reviewed by the geotechnical consultant once detailed plans are avaialble,

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes in the overburden and weathered bedrock should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structures are 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.

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 may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation 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 impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 8 - 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 are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

Underpinning of Adjacent Structures

If the footings of the proposed building are anticipated to extend within the lateral support zone of adjacent building foundations, underpinning of these structures may be required. The depth of the underpinning will be dependent on the depth of the neighbouring foundations relative to the foundation depths of the proposed building at the subject site.

Founding conditions of adjacent structures bordering the site should be assessed prior to construction and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with City of Ottawa standards and specifications.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material when placed over a soil subgrade. The bedding layer should be increased to a minimum thickness of 300 mm when placed over a bedrock subgrade. The material should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A crushed stone, should extend from the spring line of the pipe to a minimum of 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

Generally, the dry glacial till could be placed above the cover material if the excavation and backfilling operations are completed in dry weather conditions. The wet glacial till materials could be difficult to place and compact, due to the presence of cohesive soil and moisture.

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 consist of the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

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. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 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.

Impact on Neighboring Structures

Based on our observations, 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 minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within the native glacial till and/or over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long-term damage to adjacent sound structures surrounding the proposed building. However, underpinning requirements for adjacent structures should be evaluated at the time of excavation.

6.6 Winter Construction

Precautions should be provided if winter construction is considered for this project. The subsurface soil conditions 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 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.

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

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

6.7 Protection of Potential Expansive Bedrock

The presence of 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.

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 development 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 is recommended for this purpose. 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.

7.0 Recommendations

The following is recommended to be completed once the site plan and development are determined:

☐ Review the proposed protection system for the potentially expansive shales.

☐ Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction.

☐ 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.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to ensure that the specified level of compaction has been achieved.

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

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

A report confirming the construction has been completed in general accordance with the recommendations could be issued upon request, following the completion of a satisfactory material 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.

The recommendations are based on information gathered at the specific test locations and could only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities.

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 Minto Communities Inc. 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.

Drew Petahtegoose, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Dec 15, 2020
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution

☐ Minto Communities Inc. (1 copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 23, 2019

FILE NO. PG4938

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.33
FILL: Crushed stone (to 0.46)	AU	1				
FILL: Brown silty sand, some gravel (to 1.52)	SS	2	50	19	1	57.33
GLACIAL TILL: Very dense, brown clayey silty sand with gravel and shale (to 2.29)	SS	3	62	52	2	56.33
Weathered shale **BEDROCK** (to 3.25)	SS	4	100	50+		
	SS	5	100	50+	3	55.33

End of Borehole

Practical refusal to augering at 3.25m depth

(BH dry upon completion)

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

○ Water Content %

20 40 60 80

Shear Strength (kPa)

20 40 60 80 100

▲ Undisturbed △ Remoulded

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 23, 2019

FILE NO. PG4938

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	57.94
FILL: Crushed stone, some sand (0.33)	AU	1				
GLACIAL TILL: Compact to very dense, brown clayey silt with sand, shaley gravel, cobbles and boulders	SS	2	79	14	1	56.94
	SS	3	100	35	2	55.94
	SS	4	100	50+		
	SS	5	100	50+	3	54.94
	SS	6	100	82	4	53.94
(4.50) Weathered black shale **BEDROCK**	SS	7	100	50+	5	52.94
	SS	8	100	50+	6	51.94
(6.86) End of Borehole						

End of Borehole

Practical refusal to augering at 6.86m depth.

(GWL @ 3.92m - June 3, 2019)

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
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

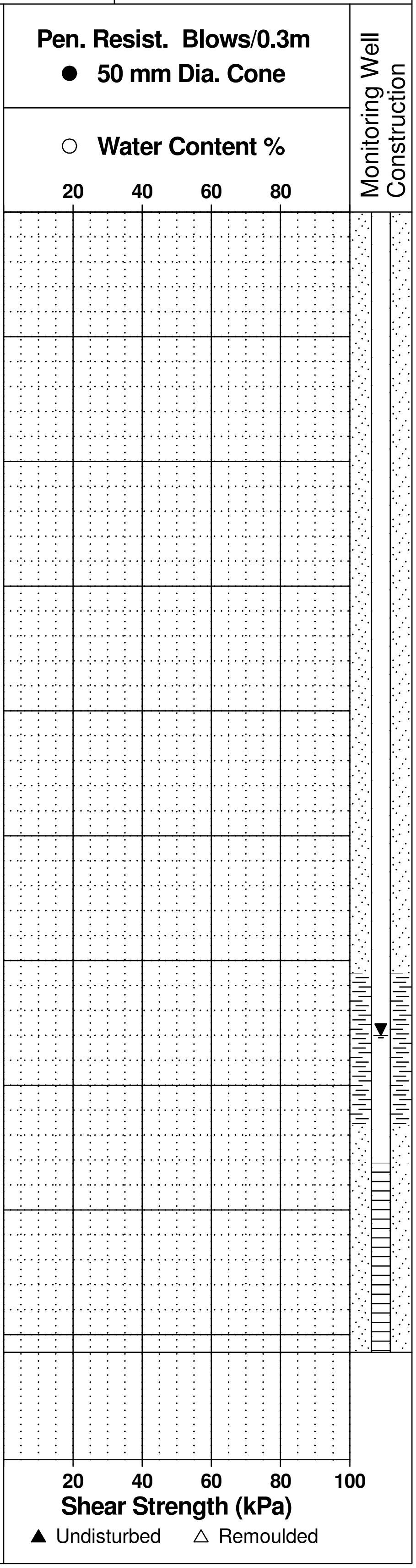
DATE May 23, 2019

FILE NO. PG4938

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Crushed stone, some sand 0.23	AU	1		
FILL: Brown silty sand, trace gravel, brick and concrete	SS	2	92	7
	SS	3	92	14
2.29				
FILL: Brown silty sand, some gravel, trace clay	SS	4	91	88
3.05				
	SS	5	80	50+
	SS	6	92	70
GLACIAL TILL: Very dense, brown sand silty/silty sand with gravel, cobbles and boulders	SS	7	100	50+
	SS	8	100	50+
	SS	9	100	50+
6.86				
Weathered black shale **BEDROCK**	SS	10	100	50+
9.14				
End of Borehole				
(GWL @ 6.60m - June 3, 2019)				

DEPTH (m)	ELEV. (m)
0	58.66
1	57.66
2	56.66
3	55.66
4	54.66
5	53.66
6	52.66
7	51.66
8	50.66
9	49.66



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 23, 2019

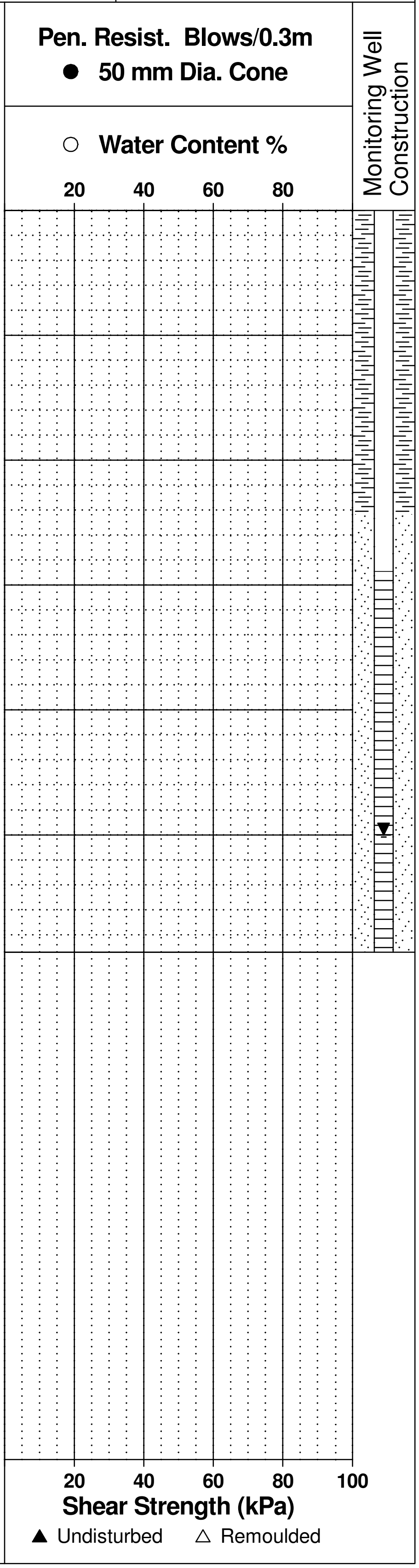
FILE NO. PG4938

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.06
FILL: Crushed stone, some sand	0.28
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders	2.29
Weathered black shale **BEDROCK**	5.94
End of Borehole	
(GWL @ 5.00m - June 3, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	79	13
SS	3	100	54
SS	4	100	50+
SS	5	91	50+
SS	6	80	50+
SS	7	100	50+

DEPTH (m)	ELEV. (m)
0	58.43
1	57.43
2	56.43
3	55.43
4	54.43
5	53.43



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 24, 2019

FILE NO. PG4938

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.74
Asphaltic concrete — 0.05						
FILL: Brown silty sand with crushed stone — 0.25	AU	1				
FILL: Brown silty sand, trace gravel	SS	2	88	10	1	57.74
	SS	3	92	15	2	56.74
	SS	4	88	23		
— 3.20	SS	5	100	50+	3	55.74
Weathered black shale **BEDROCK**	SS	6	67	50+	4	54.74
— 4.42						

End of Borehole

Practical refusal to augering at 4.42m depth

(BH dry upon completion)

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
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 24, 2019

FILE NO. PG4938

HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.06					0	58.45
FILL: Brown silty sand with crushed stone 0.30	AU	1				
	SS	2	79	16	1	57.45
GLACIAL TILL: Compact to very dense, brown clayey silt with shale, gravel, cobbles and boulders	SS	3	83	38		
					2	56.45
	SS	4	91	89		
3.05					3	55.45
Weathered black shale **BEDROCK**	SS	5	67	50+		
3.66						
End of Borehole						
Practical refusal to augering at 3.66m depth						
(BH dry upon completion)						

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
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 15, 2019

FILE NO. PG4938

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.59
FILL: Brown silty sand with crushed stone 0.25		AU	1				
FILL: Brown/black silty sand, some gravel 1.52		SS	2	25	8	1	57.59
Weathered black shale **BEDROCK**		SS	3	83	67	2	56.59
		SS	4	42	70	3	55.59
		SS	5	75	38		
		SS	6	88	50+	4	54.59
		SS	7	80	50+	5	53.59
		SS	8		50+		
6.10						6	52.59
End of Borehole (GWL @ 4.50m - July 19, 2019)							

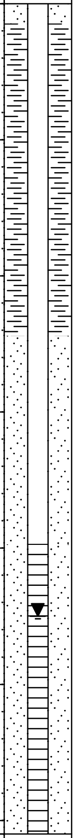
Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

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

Monitoring Well Construction



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 15, 2019

FILE NO. PG4938

HOLE NO. BH 8

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown slty sand with gravel, trace organics	0.30	AU	1		
FILL: Brown silty sand with gravel and construction debris	1.52	SS	2	29	9
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and shale		SS	3	38	49
		SS	4	33	34
		SS	5	58	73
		SS	6	75	80
		SS	7	83	89
	6.00	SS	8	71	61
Weathered black shale **BEDROCK**	6.70	SS	9		56

End of Borehole

(BH dry - July 19, 2019)

DEPTH (m)	ELEV. (m)
0	58.67
1	57.67
2	56.67
3	55.67
4	54.67
5	53.67
6	52.67

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
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

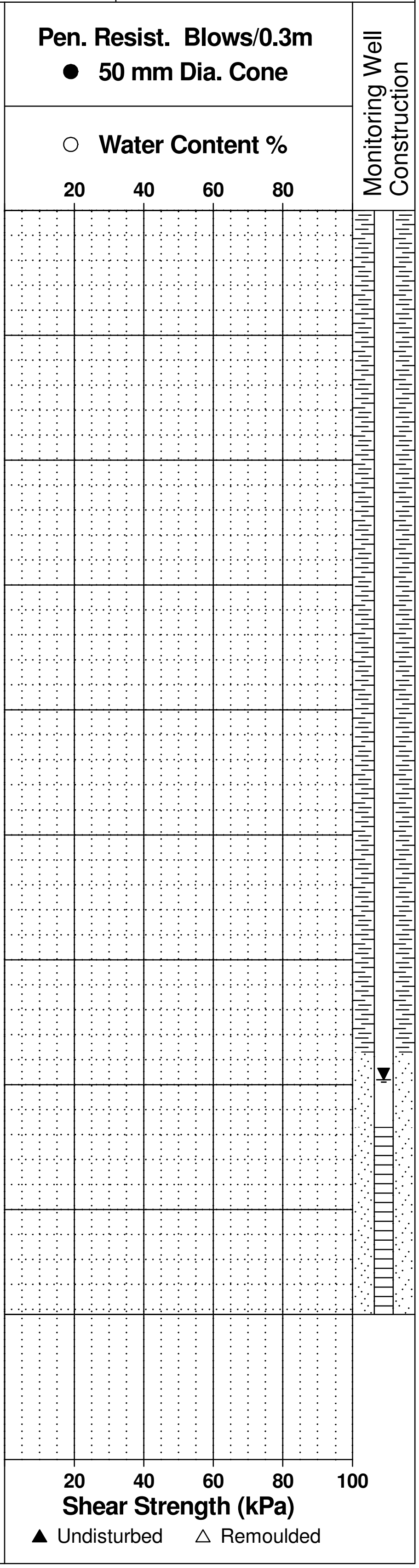
BORINGS BY CME 55 Power Auger

DATE July 15, 2019

FILE NO. PG4938

HOLE NO. BH 8B

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.67
FILL: Brown slty sand with gravel, trace organics — 0.30	▨						
FILL: Brown silty sand with gravel and construction debris — 1.52	▨					1	57.67
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and shale — 6.00	^^^					2	56.67
						3	55.67
						4	54.67
						5	53.67
Weathered black shale **BEDROCK** — 8.84	☰					6	52.67
						7	51.67
						8	50.67
End of Borehole							
(GWL @ 6.96m - July 31, 2019)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

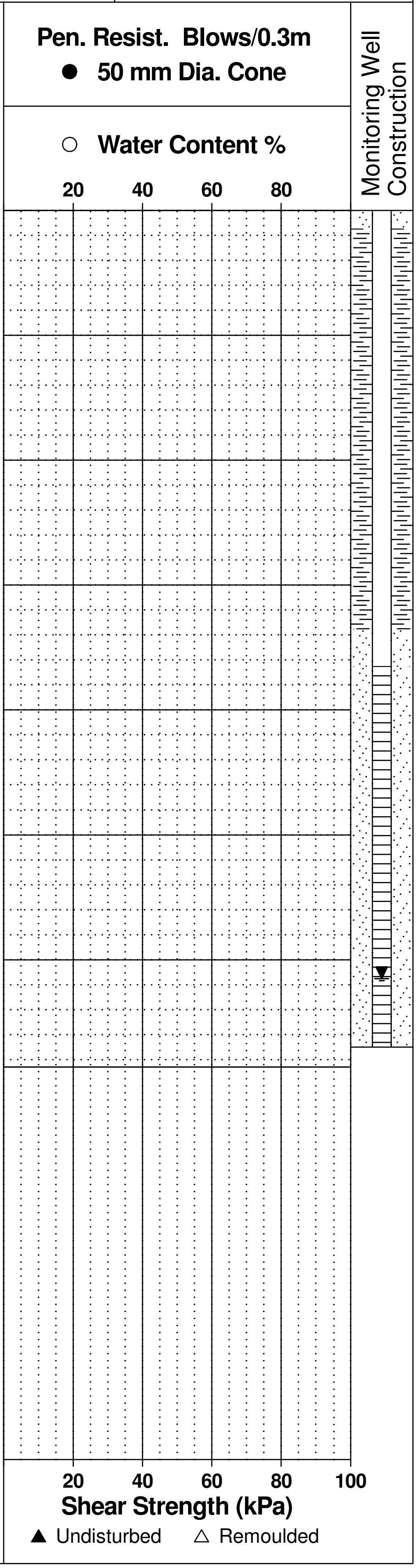
BORINGS BY CME 55 Power Auger

DATE July 15, 2019

FILE NO. PG4938

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.71
FILL: Brown silty sand with crushed stone 0.33		AU	1				
		SS	2	8	9	1	57.71
FILL: Brown silty sand							
		SS	3	42	18	2	56.71
2.30							
		SS	4	53	50+		
						3	55.71
		SS	5	80	50+		
						4	54.71
		SS	6	75	38		
GLACIAL TILL: Dense to very dense, grey silty sand with gravel and shale fragments							
		SS	7	71	51	5	53.71
		SS	8	71	52		
						6	52.71
		SS	8	64	66		
6.65							
Weathered black shale **BEDROCK** 6.86							
End of Borehole							
(GWL @ 6.15m - July 19, 2019)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 15, 2019

FILE NO. PG4938

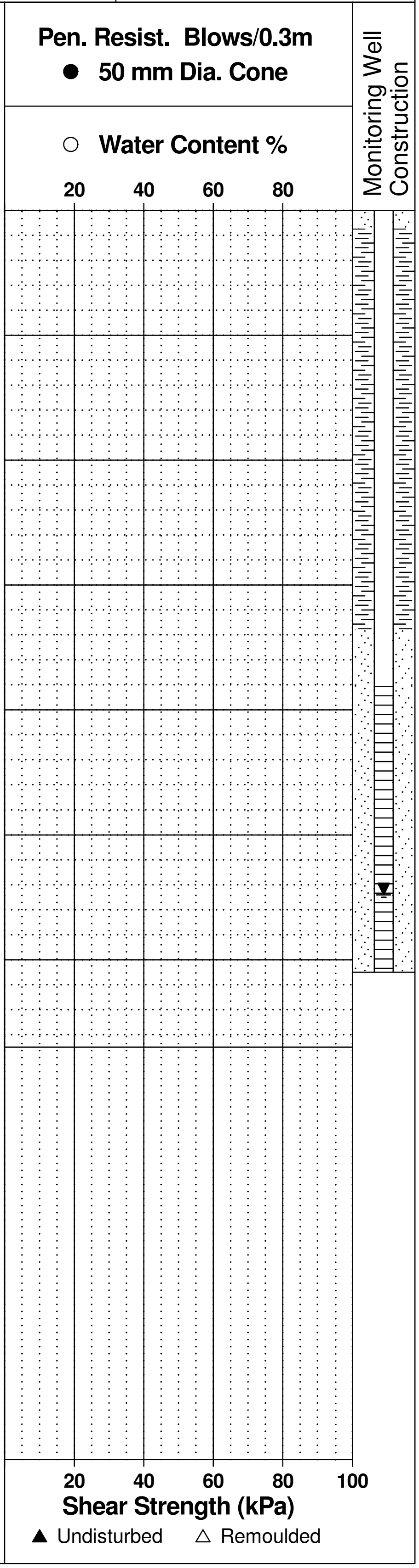
HOLE NO. BH10

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone (0.25)	AU	1		
FILL: Brown silty sand, some gravel, trace organics (1.50)	SS	2	62	9
GLACIAL TILL: Dense to very dense, brown silty sand, some gravel and shale (3.05)	SS	3	100	34
	SS	4	100	50+
Weathered black shale **BEDROCK** (6.70)	SS	5	70	50+
	SS	6	100	50+
	SS	7	100	50+
	SS	8	100	50+
	SS	9	50	50+

End of Borehole

(GWL @ 5.48m - July 19, 2019)

DEPTH (m)	ELEV. (m)
0	58.66
1	57.66
2	56.66
3	55.66
4	54.66
5	53.66
6	52.66



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

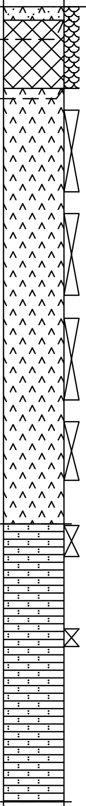
DATE August 24, 2020

FILE NO. PG4938

HOLE NO. BH11

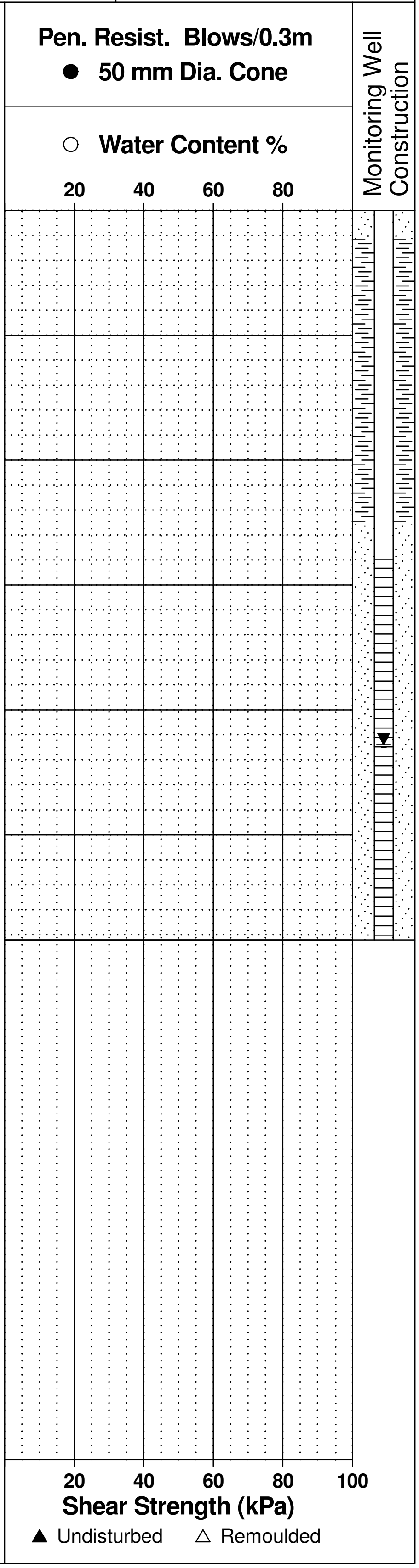
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.60
GLACIAL TILL: Dense to very dense, brown silty sand with shale	3.80
Possible weathered **BEDROCK**	5.84
End of Borehole	

(GWL @ 4.28m - Sept. 3, 2020)



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	62	6
SS	3	54	40
SS	4	79	44
SS	5	76	50+
SS	6	78	50+
SS	7	100	50+

DEPTH (m)	ELEV. (m)
0	58.11
1	57.11
2	56.11
3	55.11
4	54.11
5	53.11



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 24, 2020

FILE NO. PG4938

HOLE NO. BH12

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	57.57
Asphaltic concrete 0.10						
FILL: Brown silty sand with crushed stone 0.63	AU	1				
GLACIAL TILL: Compact to very dense, brown silty sand with shale	SS	2	46	10	1	56.57
	SS	3	58	52	2	55.57
2.30						
Weathered black shale **BEDROCK**	SS	4	71	50+		
					3	54.57
3.28	SS	5	11	50+		
End of Borehole						

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
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

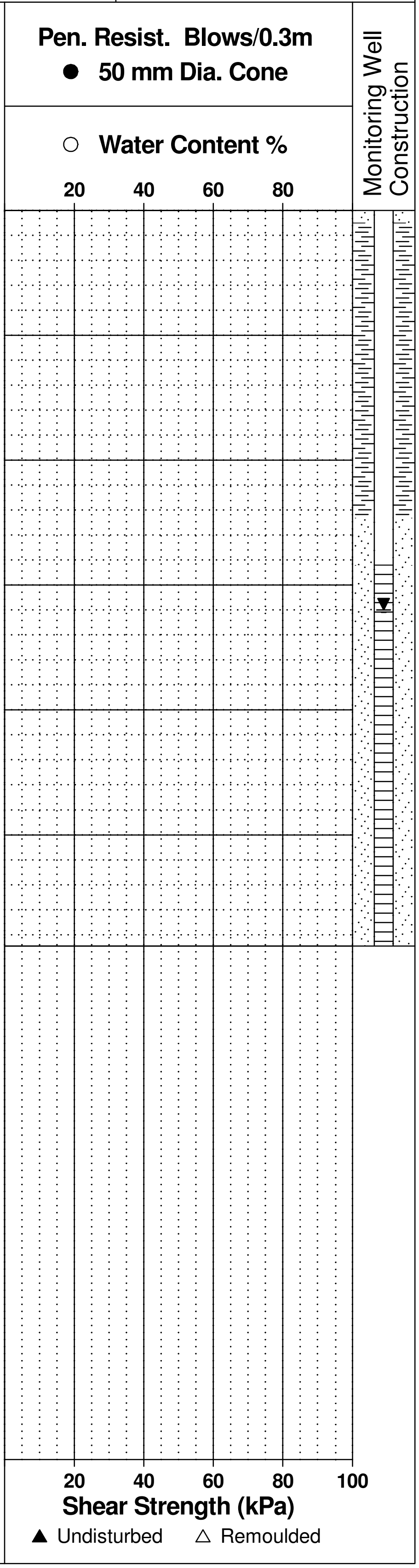
DATE August 24, 2020

FILE NO. PG4938

HOLE NO. BH13

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone 0.43	AU	1		
Compact, brown **SILTY SAND,** some gravel	SS	2	83	11
	SS	3	67	24
	SS	4	75	17
3.50	SS	5	62	20
GLACIAL TILL: Very dense, brown silty sand with shale	SS	6	90	50+
4.60				
Weathered black shale **BEDROCK**	SS	7	90	50+
5.89				
End of Borehole				
(GWL @ 3.20m - Sept. 3, 2020)				

DEPTH (m)	ELEV. (m)
0	58.73
1	57.73
2	56.73
3	55.73
4	54.73
5	53.73



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
78, 84, 86 and 90 Beechwood Avenue
83, 85, 89, 93 Barrette Street, Ottawa, Ontario

DATUM TBM - Mag nail on sidewalk, south of Beechwood Ave., east of St. Charles Street. Geodetic elevation = 59.29m.

REMARKS

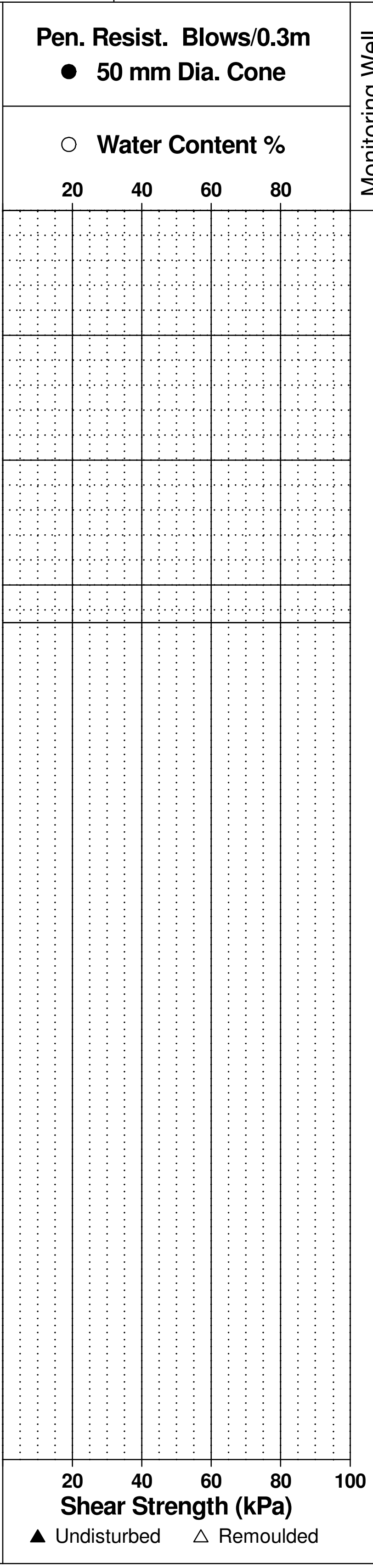
BORINGS BY CME-55 Low Clearance Drill

DATE August 24, 2020

FILE NO. PG4938

HOLE NO. BH14

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.25
FILL: Brown silty sand with crushed stone	AU	1				
1.07	SS	2	67	38	1	57.25
GLACIAL TILL: Very dense, brown/black silty sand with shale	SS	3	71	60	2	56.25
	SS	4	73	50+		
3.00					3	55.25
Weathered black shale **BEDROCK**	SS	5	50	50+		
3.30						
End of Borehole						



Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

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

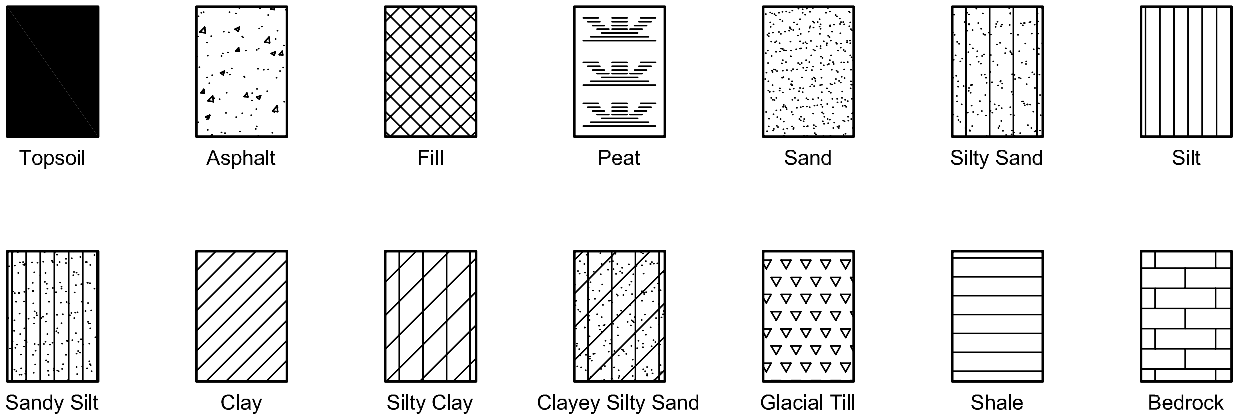
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

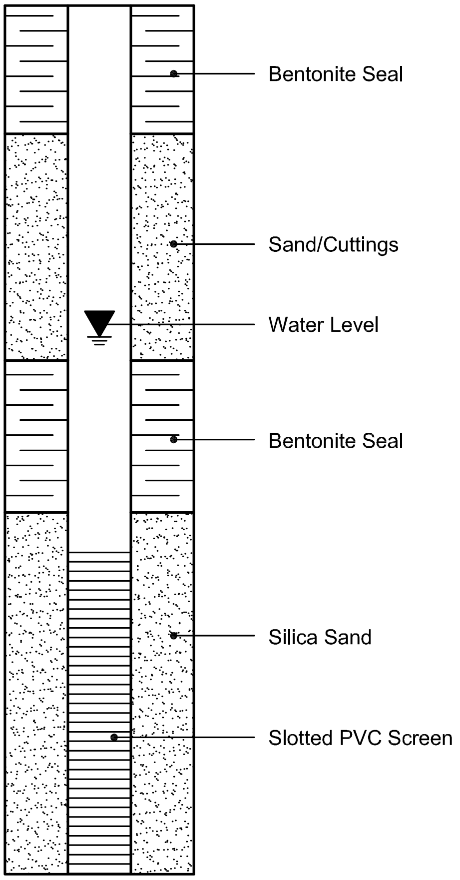
SYMBOLS AND TERMS (continued)

STRATA PLOT

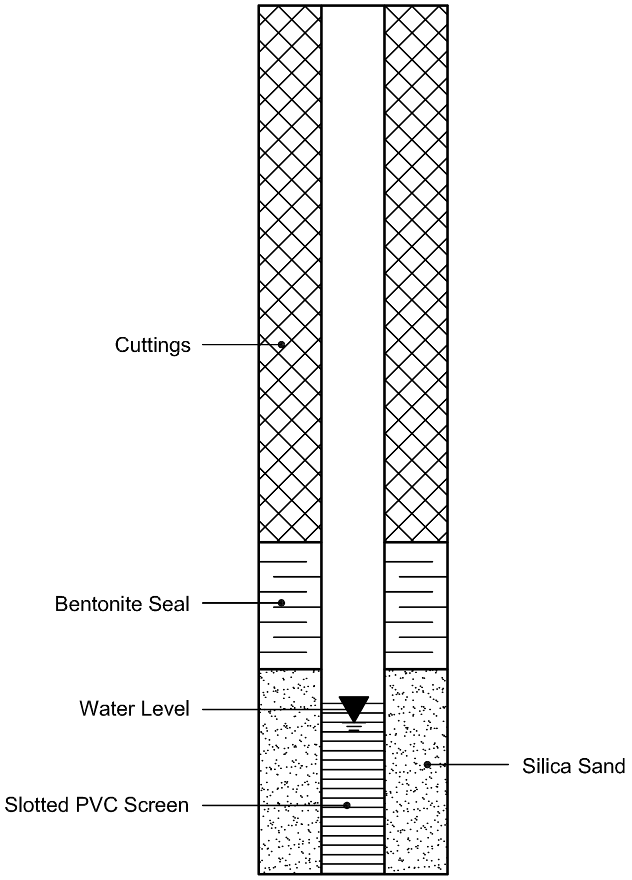


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4938-1 - TEST HOLE LOCATION PLAN

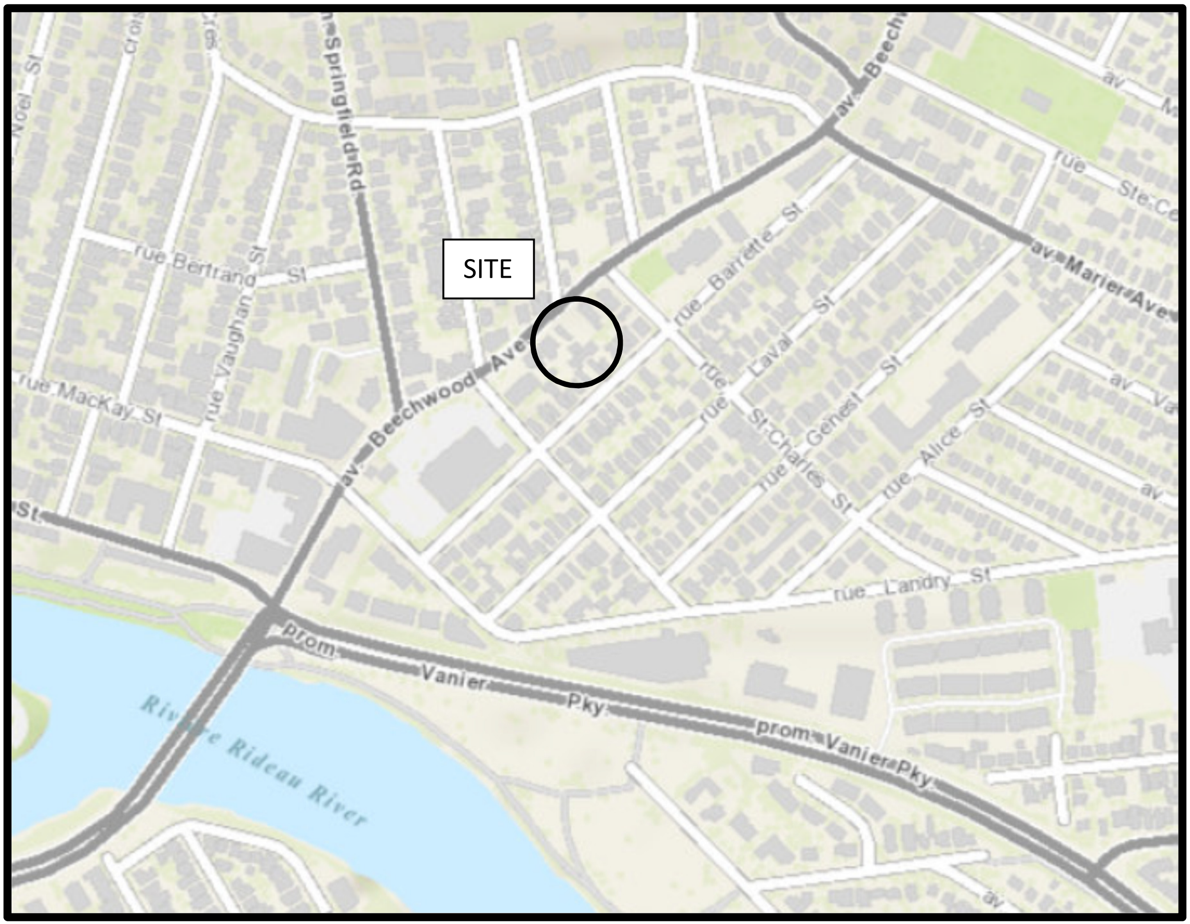
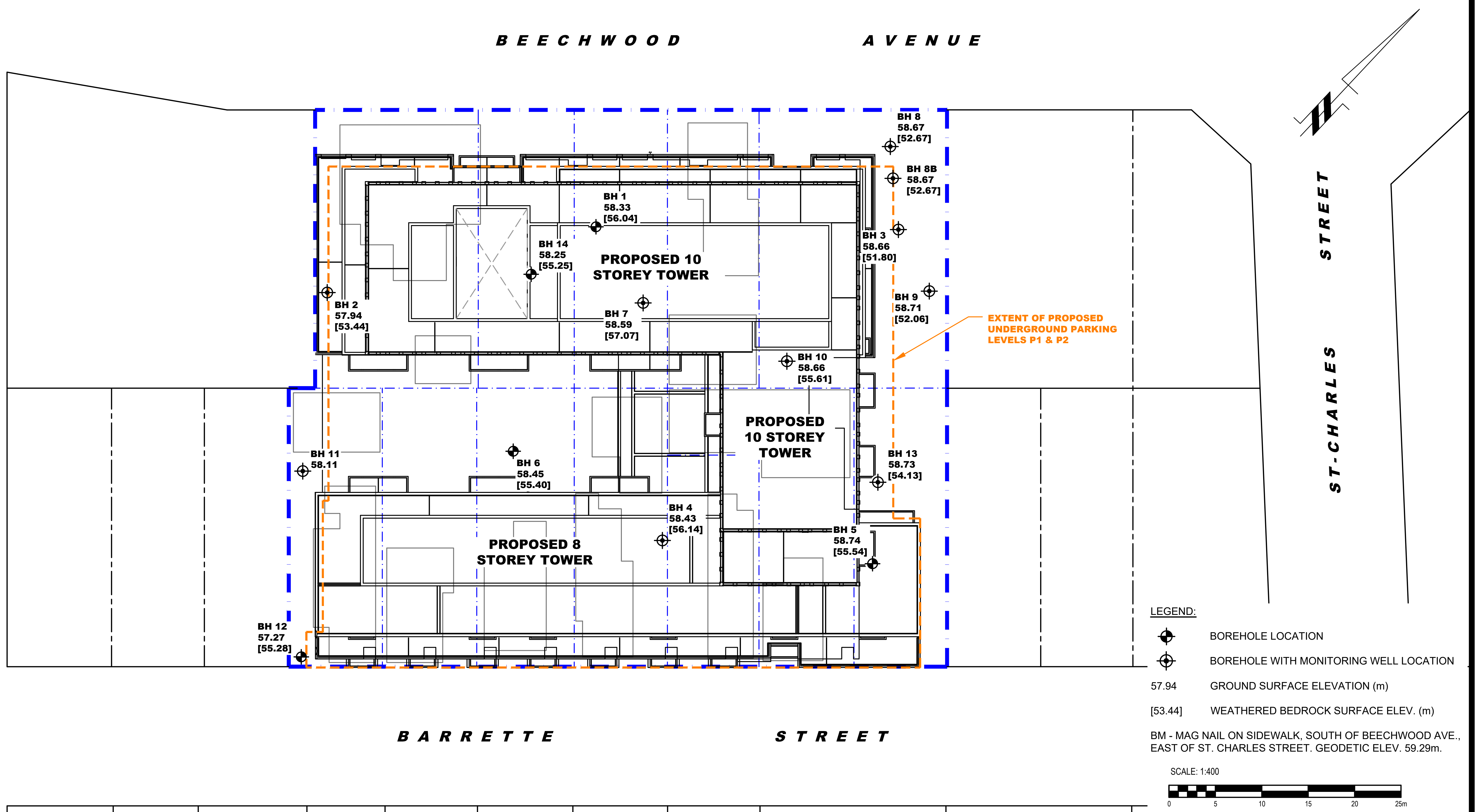


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

●	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
57.94	GROUND SURFACE ELEVATION (m)
[53.44]	WEATHERED BEDROCK SURFACE ELEV. (m)

BM - MAG NAIL ON SIDEWALK, SOUTH OF BEECHWOOD AVE., EAST OF ST. CHARLES STREET. GEODETIC ELEV. 59.29m.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
2	BH 11 & BH 12 ADDED, CONCEPTUAL SITE PLAN ADDED	26/11/2020	OM
1	BH 7 TO BH 10 ADDED	08/07/2019	FA

MINTO COMMUNITIES INC.
GEOTECHNICAL INVESTIGATION
78-88 BEECHWOOD AVE. AND 69-93 BARRETTE ST.
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 05/2020
Drawn by: NFRV
Report No.: PG4938-1
Checked by: DP
Approved by: DJG

PG4938-1

Revision No.: 2

c:\users\nicholasv\documents\old files\4938\pg4938-1 rev2 thlp.dwg