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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Multi-Storey Building
100 Argyle Avenue
Ottawa, Ontario

Prepared For

Colonnade BridgePort

July 4, 2021

Report PG4458-1 Revision 3

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade BridgePort to conduct a geotechnical investigation for the proposed multi-storey building to be located at 100 Argyle Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

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

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report. Environmental concerns for this site have been addressed under separate cover.

2.0 Proposed Project

It is understood that the proposed development will consist of a 12-storey building with 2 levels of underground parking, as well as associated at-grade parking areas, access lanes and landscaped areas. It is further understood that the subject site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on July 26 and 27, 2018. At that time, three (3) boreholes were advanced to a maximum depth of 39.6 m below ground surface. A supplementary field program was undertaken on November 30, 2018, at which time one (1) additional borehole was advanced to a depth of 40.6 m. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG4458-1 - Test Hole Location Plan in Appendix 2.

The boreholes were completed with a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of each 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at 3 borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

32 mm diameter rigid PVC monitoring wells were installed at each borehole location to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The ground surface elevations were referenced to a temporary benchmark (TBM) consisting of the top spindle of a fire hydrant to the north of the subject site. A geodetic elevation of 71.09 m was provided for the TBM by Annis, O'Sullivan, Vollebekk Ltd. The borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG4458-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two-storey commercial building with associated paved parking areas and access lanes, as well as landscaped areas. The site is bordered by Argyle Avenue to the north followed by the Canadian Museum of Nature and commercial properties to the east, south and west. The subject site is flat and approximately at grade with Argyle Avenue and the neighbouring properties to the east and west. A change in ground surface elevation of approximately 0.5 m down was noted from the subject property to the adjacent property to the south.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the test hole locations generally consists of an asphalt pavement structure followed by a fill layer extending to between 1.6 to 6.2 m depth. The fill material was generally observed to consist of a brown silty sand with gravel or gravelly sand with silt. A stiff to very stiff silty clay deposit was encountered underlying the fill. Practical refusal to the DCPT was encountered at 34.4 m depth in BH 2. A DCPT was carried out to 39.6 m depth in BH 3 with no refusal encountered. A DCPT was carried out to refusal at 40.6 m in BH 3A during the course of the supplemental investigation. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale of the Billings formation with an anticipated overburden drift thickness of 25 to 50 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on August 7, 2018. The measured groundwater level (GWL) readings are presented in Table 1 below. Based on our field observations, experience with the local area, moisture levels and the colouring of the recovered samples, it is expected that the long-term groundwater level is between 6 and 7 m below existing grade.

Table 1 - Summary of Groundwater Levels				
Borehole Number	**Ground Surface Elev. (m)**	**Measured Groundwater Level**		**Recording Date**
		Depth (m)	**Elevation (m)**	
BH 1-18	70.25	7.29	62.96	August 7, 2018
BH 2-18	70.25	8.60	61.65	August 7, 2018
BH 3-18	70.17	8.83	61.34	August 7, 2018
Note: Ground surface elevations at the test hole locations were referenced to a TBM consisting of the top spindle of a fire hydrant with geodetic elevation of 71.09 m provided by Annis, O'Sullivan, Vollebekk Ltd.				

It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed building. It is understood that the proposed building will have two underground parking levels. Therefore, the proposed structure is expected to be founded over a raft foundation placed over an undisturbed, stiff silty clay bearing surface. Consideration can be given to a piled foundation if the building loads exceed the bearing resistance values provided.

To complete the construction of the underground levels, temporary shoring will be needed. The design of the temporary shoring system needs to adequately support the existing building along the west side of the site, which is in close proximity with the proposed excavation.

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

5.2 Site Grading and Preparation

Stripping Depth and Subgrade Preparation

Topsoil and fill, containing deleterious, organic or otherwise unsuitable materials, should be stripped from under the proposed structure. A minimum 75 mm thick layer of lean concrete (min. 17 MPa) mud slab should be placed across the exposed subgrade to limit disturbance due to worker traffic.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values - Raft Foundation

For a raft foundation placed over undisturbed stiff silty clay bearing surface, the bearing resistance value at SLS (contact pressure) of **250 kPa** can be used for design purposes. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **375 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **8.0 MPa/m** for a contact pressure of **250 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Pile Foundation

Consideration can be given to a piled foundation if the building loads exceed the bearing resistance values given above. A deep foundation system driven to refusal in the bedrock can be used for foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1495	25	40
245	11	1750	24	48.5
245	13	2000	25	56

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the in-situ fill and for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculations for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure (i.e. footing) pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil.

Should additional passive resistance be required, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for pile foundations bearing on the bedrock surface. The soils underlying the subject site are not susceptible to liquefaction.

Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The basement slab will be placed over a granular fill layer placed immediately over the raft foundation. OPSS Granular B Type II is recommended for backfilling below the basement slab. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compacted to at least 98% of the material's SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. 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 dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by 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 wall height should be incorporated to the 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 calculated 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 stay at least 0.3 m away 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}) could 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. The vertical seismic coefficient is assumed to be zero.

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

Minimum Pavement Structure Recommendations

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 3 and 4.

Table 3 - 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 4 - 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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the information provided, it is expected that a portion of the proposed building foundation walls located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which breaches the waterproofing layer.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level and the following is suggested for preliminary design purposes:

- ❏ Place a suitable waterproofing membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum of 600 mm below the footing at underside of footing level.
- ❏ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane, as a secondary system. The composite drainage layer should extend from finished grade to underside of footing level.
- ❏ Pour the foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the top of the raft foundation. It is recommended that 150 mm diameter sleeves at 3-6 m centres be cast at the foundation wall/top of raft slab interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration for the proposed structure. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Due to low permeability of the subsoils profile, any minor dewatering will be considered temporary and limited to the local area of the proposed building during the construction period. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to any groundwater lowering.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

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

6.3 Excavation Side Slopes

The side slopes of excavations at the site should be cut back at acceptable slopes from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the excavation to be undertaken by open-cut methods.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1.5H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

A temporary Ministry of 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, 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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 5.1. Any groundwater which encounters the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 15,000 L/day with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on the existing groundwater level and low permeability of the adjacent soils, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the successful implementation of the water suppression system and the minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that 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.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) is appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to aggressive corrosive environment.

7.0 Recommendations

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

- ❑ Review the geotechnical aspects of the excavating program, prior to construction.
- ❑ 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 drawings and specifications are completed.

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

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

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

Paterson Group Inc.

Maha K. Saleh, P.Eng. (Provisional)

David J. Gilbert, P.Eng.

Report Distribution

- ❏ Colonnade BridgePort (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** July 26, 2018

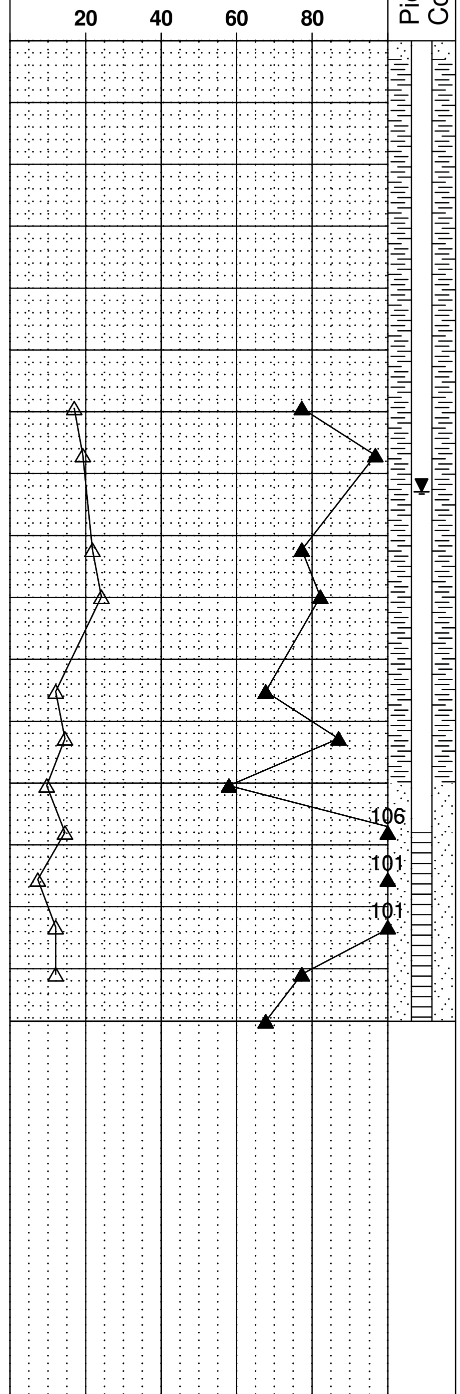
FILE NO. PG4458

HOLE NO. BH1-18

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	70.25
Asphaltic concrete (0.08)	AU	1				
FILL: Brown silty sand, some gravel, trace asphalt (1.62)	SS	2	33	4	1	69.25
Very stiff to stiff, brown **CLAYEY SILT,** trace sand (4.57)	SS	3	46	8	2	68.25
	SS	4	67	7	3	67.25
	SS	5	62	9		
	SS	6	96	4	4	66.25
Stiff, grey **SILTY CLAY** (15.85)	SS	7		2	5	65.25
					6	64.25
	SS	8	96	1	7	63.25
					8	62.25
	SS	9	96	W	9	61.25
					10	60.25
					11	59.25
					12	58.25
					13	57.25
					14	56.25
					15	55.25
End of Borehole						
(GWL @ 7.29m - Aug. 7, 2018)						

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

BORINGS BY CME 55 Power Auger

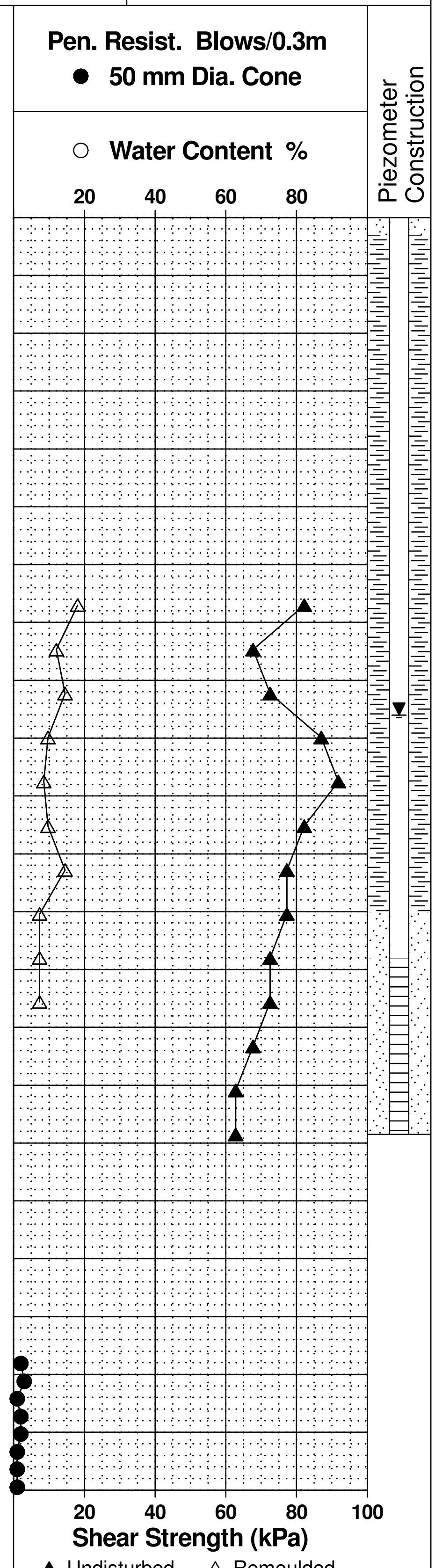
DATE July 26, 2018

FILE NO. PG4458

HOLE NO. BH2-18

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06	AU	1		
FILL: Brown silty sand with gravel	2.29	SS	2	33	15
		SS	3	8	4
FILL: Brown gravelly sand, some silt	3.81	SS	4	67	34
		SS	5	100	8
Very stiff, brown **CLAYEY SILT,** some sand, trace gravel	5.33	SS	6	96	8
		SS	7	96	5
Very stiff, brown **SILTY CLAY**	6.10	SS	8	96	3
Stiff, grey **SILTY CLAY**	15.85	SS	9	96	W
		SS	10	96	W
		SS	11	96	W
		SS	12	96	W
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 19.2m depth.					

DEPTH (m)	ELEV. (m)
0	70.25
1	69.25
2	68.25
3	67.25
4	66.25
5	65.25
6	64.25
7	63.25
8	62.25
9	61.25
10	60.25
11	59.25
12	58.25
13	57.25
14	56.25
15	55.25
16	54.25
17	53.25
18	52.25
19	51.25
20	50.25
21	49.25
22	48.25



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

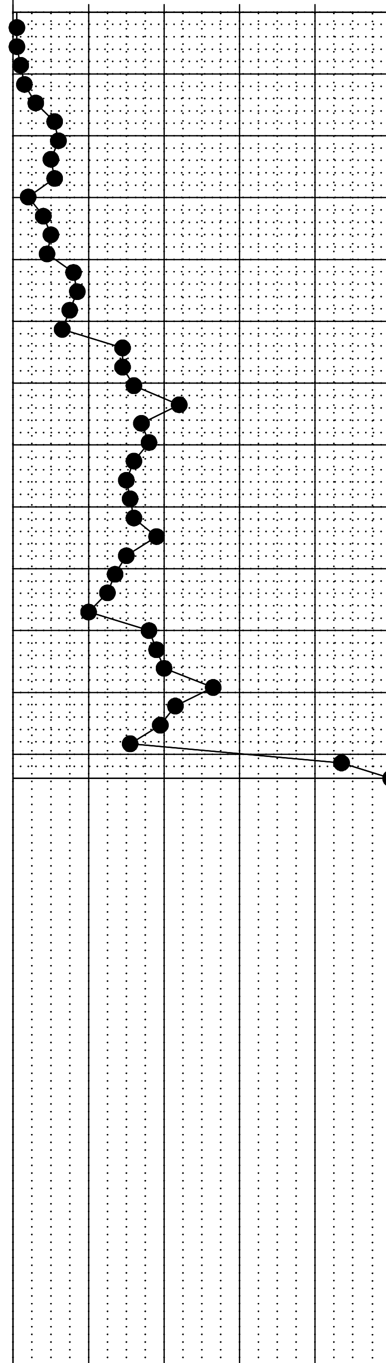
BORINGS BY CME 55 Power Auger

DATE July 26, 2018

FILE NO. PG4458

HOLE NO. BH2-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						22	48.25		
						23	47.25		
						24	46.25		
						25	45.25		
						26	44.25		
						27	43.25		
						28	42.25		
						29	41.25		
						30	40.25		
						31	39.25		
						32	38.25		
						33	37.25		
						34	36.25		
34.39									
End of Borehole Practical DCPT refusal at 34.39m depth (GWL @ 8.60m - Aug. 7, 2018)									



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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

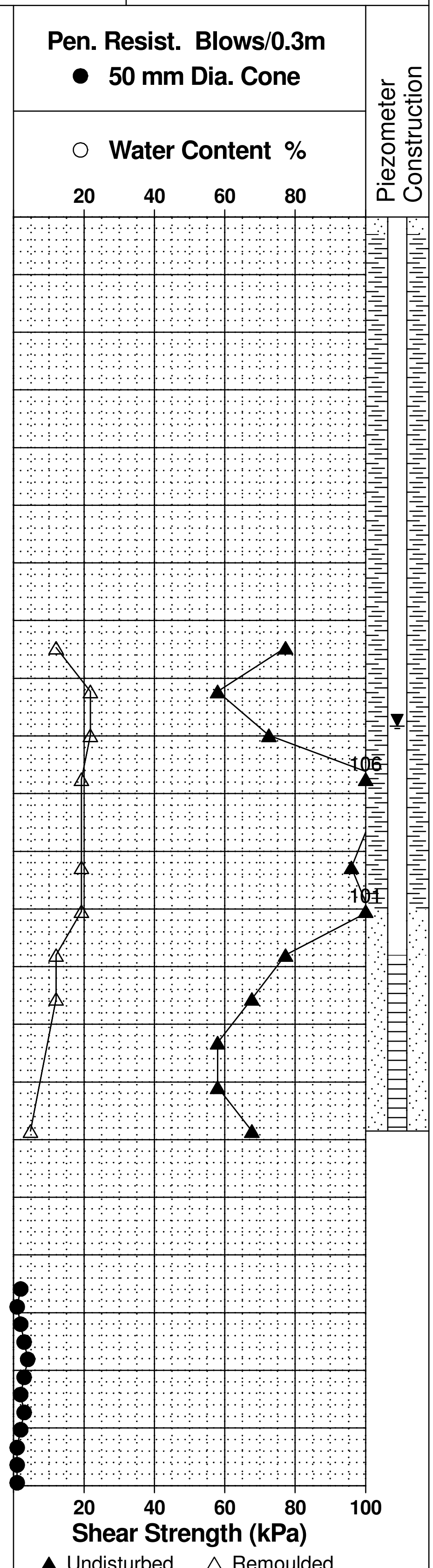
BORINGS BY CME 55 Power Auger

DATE July 27, 2018

FILE NO. PG4458

HOLE NO. BH3-18

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	70.17
Asphaltic concrete 0.08	AU	1				
FILL: Brown gravelly sand with silt	SS	2	46	26	1	69.17
	SS	3	62	60	2	68.17
	SS	4	75	40	3	67.17
	SS	5	67	20	4	66.17
	SS	6	46	13	5	65.17
	SS	7	38	11		
- some clay by 5.3m depth 6.20	SS	8	29	3	6	64.17
	SS	9		1	7	63.17
Stiff, grey **SILTY CLAY**	SS	10	42	W	8	62.17
	SS	11	42	W	9	61.17
					10	60.17
					11	59.17
					12	58.17
					13	57.17
					14	56.17
15.85					15	55.17
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 18.3m depth.					16	54.17
					17	53.17
					18	52.17
					19	51.17
					20	50.17
					21	49.17
					22	48.17



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

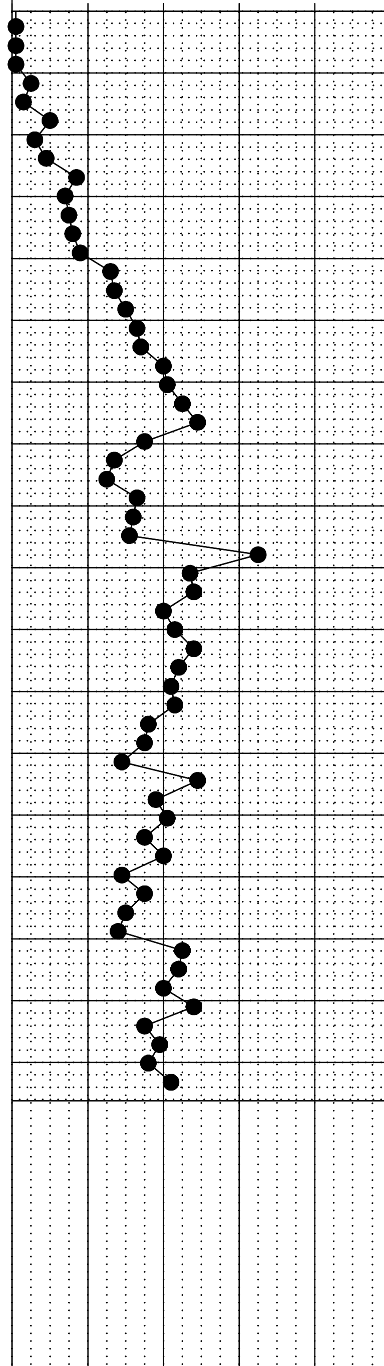
BORINGS BY CME 55 Power Auger

DATE July 27, 2018

FILE NO. PG4458

HOLE NO. BH3-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						22	48.17		
						23	47.17		
						24	46.17		
						25	45.17		
						26	44.17		
						27	43.17		
						28	42.17		
						29	41.17		
						30	40.17		
						31	39.17		
						32	38.17		
						33	37.17		
						34	36.17		
						35	35.17		
						36	34.17		
						37	33.17		
						38	32.17		
						39	31.17		
39.62									
End of Borehole (GWL @ 8.83m - Aug. 7, 2018)									



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 30, 2018

FILE NO. PG4458

HOLE NO. BH3A-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	70.17		
						1	69.17		
OVERBURDEN						2	68.17		
						3	67.17		
4.57						4	66.17		
Dynamic Cone Penetration Test (DCPT) commenced at 4.57m depth. Cone pushed to 31.85m depth.						5	65.17		
						6	64.17		
						7	63.17		
						8	62.17		
						9	61.17		
						10	60.17		
						11	59.17		
						12	58.17		
						13	57.17		
						14	56.17		
						15	55.17		
						16	54.17		
						17	53.17		
						18	52.17		
						19	51.17		
						20	50.17		
						21	49.17		
						22	48.17		

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
100 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located across from subject site, north side of Argyle Avenue. Geodetic elevation = 71.09m.

REMARKS

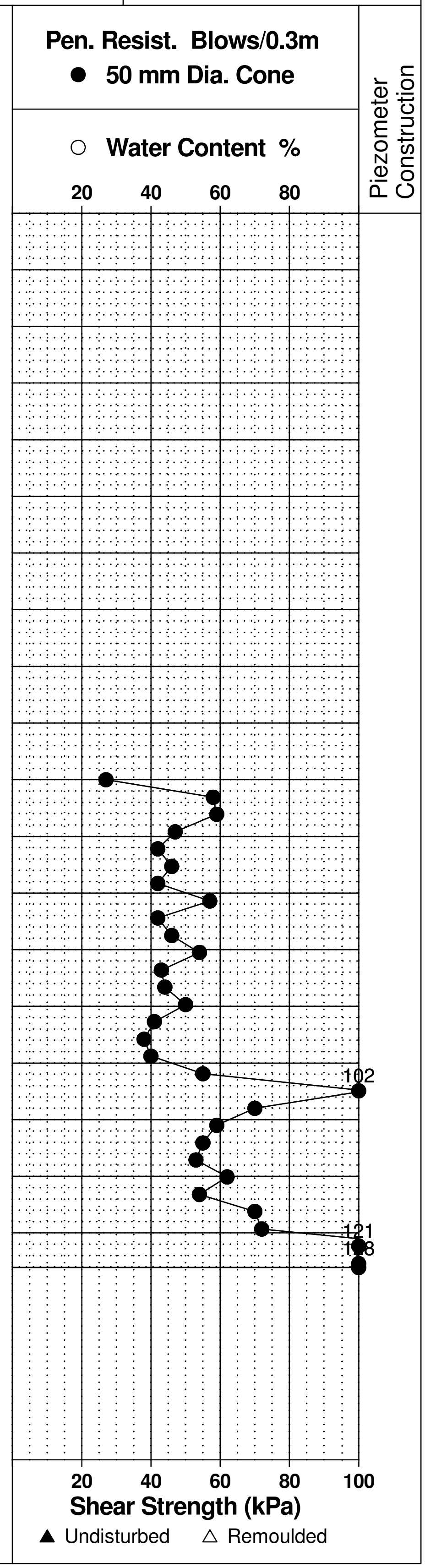
BORINGS BY CME 55 Power Auger

DATE November 30, 2018

FILE NO. PG4458

HOLE NO. BH3A-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						22	48.17
						23	47.17
						24	46.17
						25	45.17
						26	44.17
						27	43.17
						28	42.17
						29	41.17
						30	40.17
						31	39.17
						32	38.17
						33	37.17
						34	36.17
						35	35.17
						36	34.17
						37	33.17
						38	32.17
						39	31.17
						40	30.17
40.61							
End of Borehole Practical DCPT refusal at 40.61m depth.							



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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

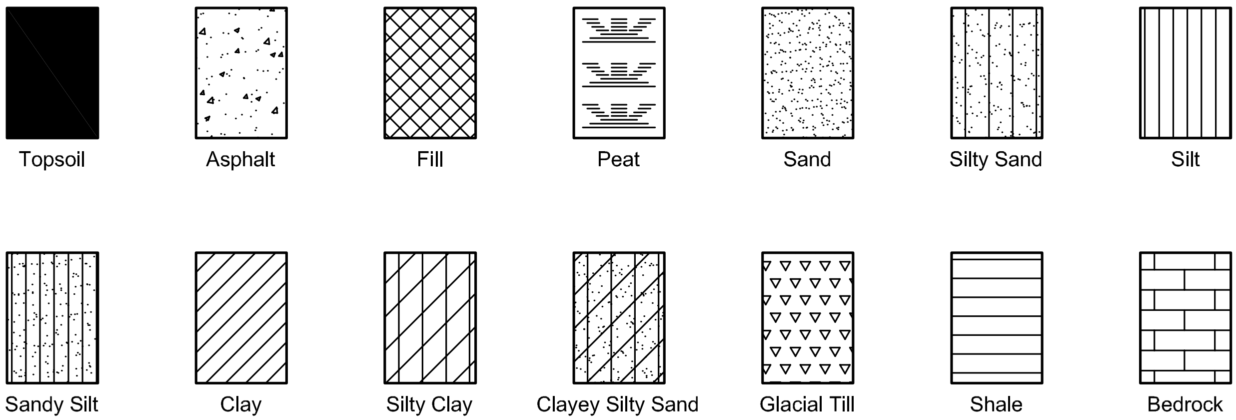
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

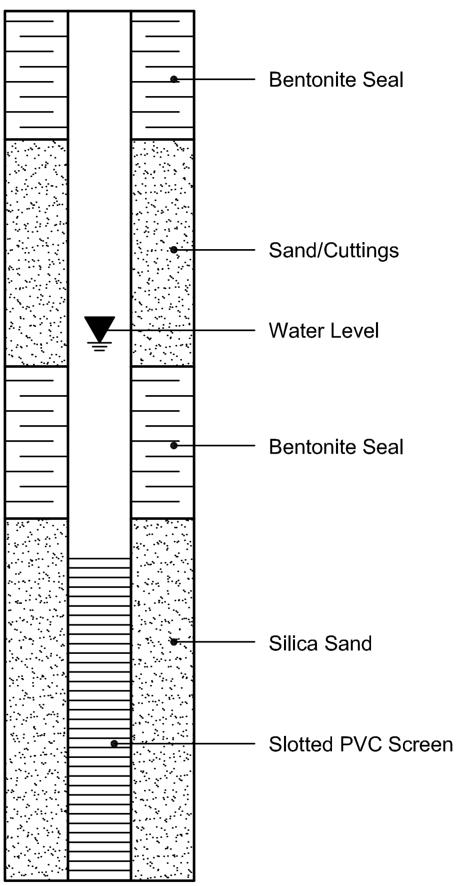
SYMBOLS AND TERMS (continued)

STRATA PLOT

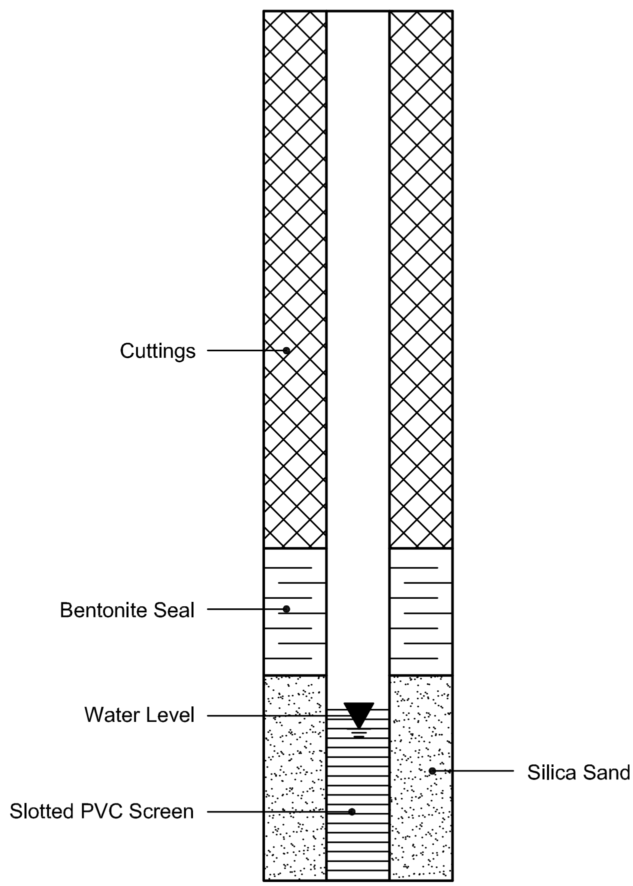


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24811

Report Date: 01-Aug-2018

Order Date: 30-Jul-2018

Project Description: PG4458

	MDL/Units				
Client ID:		BH3 SS4	-	-	-
Sample Date:		07/27/2018 10:50	-	-	-
Sample ID:		1831094-01	-	-	-
		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	95.3	-	-	-
General Inorganics					
pH	0.05 pH Units	8.02	-	-	-
Resistivity	0.10 Ohm.m	3.14	-	-	-
Anions					
Chloride	5 ug/g dry	368	-	-	-
Sulphate	5 ug/g dry	14800	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4458-1 - TEST HOLE LOCATION PLAN

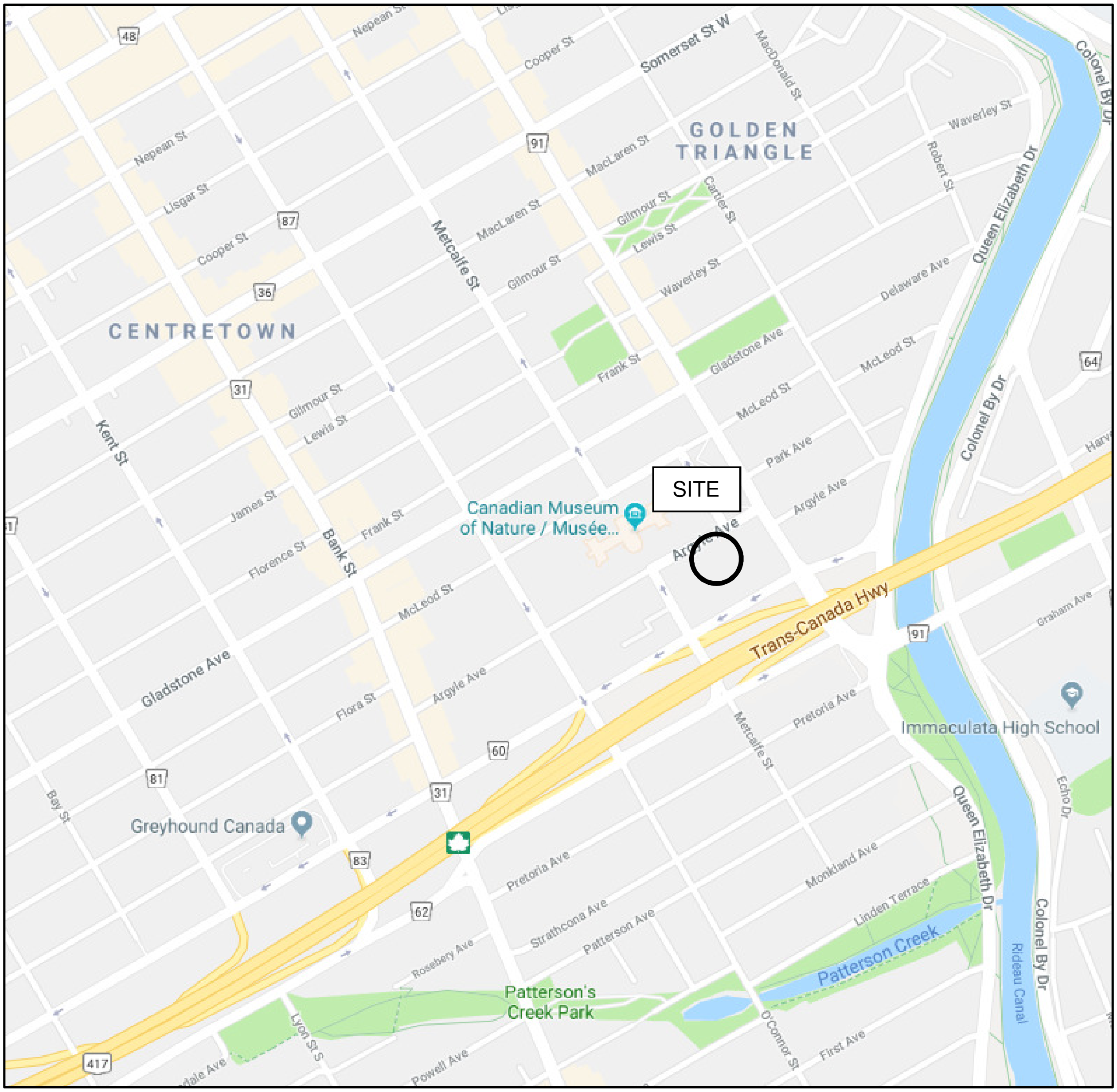
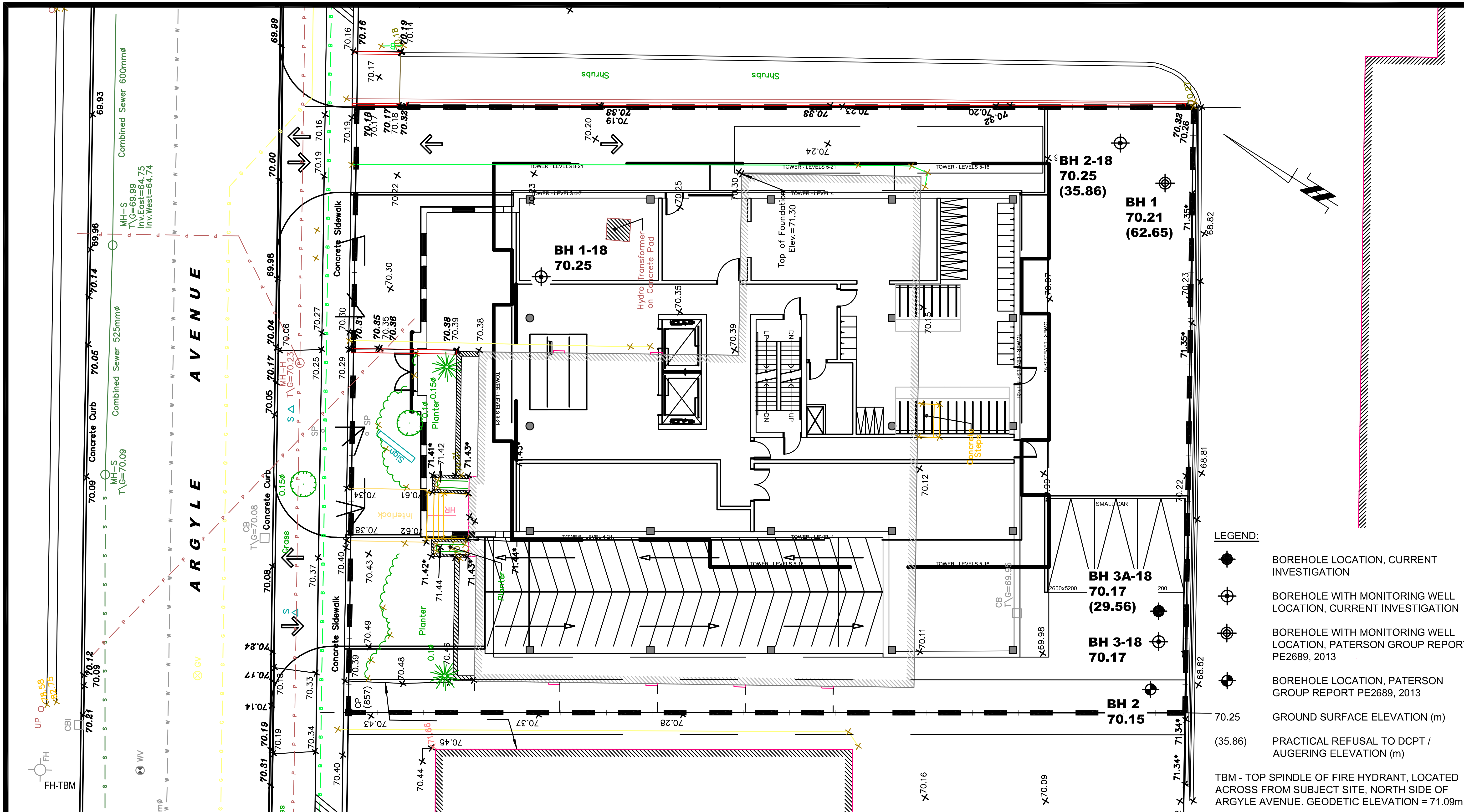


FIGURE 1

KEY PLAN



LEGEND:

- BOREHOLE LOCATION, CURRENT INVESTIGATION
- BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
- BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE2689, 2013
- BOREHOLE LOCATION, PATERSON GROUP REPORT PE2689, 2013
- 70.25 GROUND SURFACE ELEVATION (m)
- (35.86) PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT, LOCATED ACROSS FROM SUBJECT SITE, NORTH SIDE OF ARGYLE AVENUE. GEODETIC ELEVATION = 71.09m.

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consulting engineers

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NO.	REVISIONS	DATE	INITIAL
1	BH 3A-18 ADDED	4/12/2018	NC

COLONNADE BRIDGEPORT
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 100 ARGYLE AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 08/2018
Drawn by: MPG	Report No.: PG4458-1
Checked by: NC	Dwg. No.: PG4458-1
Approved by: DJG	Revision No.: 1

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