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Geotechnical Investigation

Proposed Multi-Storey Building
100 Argyle Avenue
Ottawa, Ontario

Prepared For

Colonnade BridgePort

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Report PG4458-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade BridgePort Developments 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 was 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 Development

It is understood that the proposed development will consist of a 21 storey mixed use building (first floor commercial use and the remainder will be residential) with up to 3 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, 3 boreholes were advanced to a maximum depth of 39.6 m below ground surface. 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 2 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. 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 groundwater level is between 7 and 9 m below existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1	70.25	7.29	62.96	August 7, 2018
BH 2	70.25	8.60	61.65	August 7, 2018
BH 3	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.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site has a deep silty clay deposit that extends from 34 m to approximately 42 m below the existing grade. For the height of the proposed building, it's expected that the heavier loading on the underlying silty clay deposit will require a deep foundation consisting of end bearing piles driven to bedrock.

Furthermore, to lessen the effects settlement on adjacent structures due to long term dewatering of the silty clay deposit, a tanked system will be required for the foundation portion below the long term groundwater table. A pressure relief chamber will be incorporated into the tanked system to enable a suitable groundwater management system.

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

5.2 Site Grading and Preparation

Stripping Depth

Since the building footprint will occupy the entire area of the site, the site excavation will be removing all the soil to a depth of approximately 11.5 m below the existing grade to accommodate 3 levels of underground parking and a foundation slab.

Fill Placement

Fill used for a granular working mat beneath the building footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type II crushed stone or a well graded 100 mm minus crushed stone. 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 95% of its standard Proctor maximum dry density (SPMDD).

Pressure Relief Chamber

To prevent the long term dewatering of adjacent structures surrounding the site, at the founding level, a pressure relief chamber will be installed along with collection pipes within the granular mat. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated in the lowest section of the P3 level within a utility room in close proximity to the proposed sump pits. Figure 2 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed basement slab can be constructed. The purpose of the pressure relief chamber will be as follows:

❑ manage any water infiltration along the bedrock surface during the excavation program.
❑ manage the water infiltration during the pouring of the basement slab to prevent water flow in the fresh concrete.
❑ manage water infiltration below the basement slab until sufficient load is applied to resist any potential hydrostatic uplift.
❑ regulate the discharge valve to control water infiltration once the basement slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
❑ Once the building is completed, the pressure relief valve will be fully closed to prevent any further dewatering.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of **35 kPa** will be developed over the long term and should be incorporated in the design of the raft foundation and the foundation wall. Realistically, achieving a fully watertight is not always possible due to minor water infiltration and, therefore, a realistic long term hydrostatic pressure will be closer to 20 to 25 kPa.

5.3 Foundation Design

Pile Foundation

A deep foundation system driven to refusal in the bedrock is recommended 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.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted, the piles will generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. A resistance factor of 0.9 is applicable for the ULS weight component.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) ϕ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A Fill (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B Type I Fill (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14.0	40	0.62	0.20	0.33	10.3
Granular Working Surface - Coarse Open-Graded Crushed Stone	19.0	12.0	36	0.55	0.26	0.41	7.5
Site Excavated Silty Sand or Gravelly Sand Fill	18.0	11.2	32	0.48	0.30	0.46	5.6

Notes:
- Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
- The earth pressure coefficients provided are for horizontal backfill profile.
- Passive pressure coefficients incorporate wall friction of 0.5 ϕ'.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

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

It is expected that the finished basement slab will be placed over the structural slab designed to resist uplift from hydrostatic pressure. A layer of clear stone or free draining granular backfill to promote drainage to the sump pit will be placed between the two slabs. It is expected that the basement area will be mostly parking and that a concrete slab will be used. A rigid pavement structure is presented in Subsection 5.8. The thickness of the granular subfloor layer will be dependent on the proposed elevation of the P3 level floor slab. It is also expected that a sump pit will be incorporated in the design of the basement structural slab to drain any water which enters the granular layer via a breach in the basement structural slab or foundation wall waterproofing system.

The final basement floor slab and associated underfloor granular material should only be placed once the pressure relief chamber valve has been fully closed and no significant water infiltration is observed after hydrostatic pressure is applied.

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 Rock Anchor Design

With the proposed structural tanked foundation design, it is expected that the structural engineer will not require any rock anchors. However, should rock anchors be required, the following can be considered:

The geotechnical design of grouted rock anchors in limestone bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock 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 individual anchor load capacity.

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

The centre to centre spacing between bond lengths should be at least 1.2 m or a minimum of four times the anchor hole diameter to ensure the group influence effects are minimized. 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 fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

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

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on bedrock information, a **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Lengths

Parameters used to calculate grouted rock anchor lengths are provided in Table 4.

Table 4 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Hoek and Brown parameters	44 m=0.821 and s=0.00293
Unconfined compressive strength - Shale	50 MPa
Unit weight - Submerged Bedrock	15 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 are provided in Table 5. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 5 - 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	1.0	3.0	450
	2.4	1.2	3.6	600
	2.6	1.5	4.1	750
	3.0	1.8	4.8	900
125	1.8	0.8	2.6	450
	2.0	1.0	3.0	600
	2.2	1.3	3.5	750
	2.4	1.6	4.0	900

Other considerations

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom to top of the anchor holes.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on test procedures can be provided upon request.

5.8 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 6 to 8.

Table 6 - Recommended Rigid Pavement Structure - Parking Garage Lower Areas

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
200	**BASE** - 20 mm clear stone
	SUBGRADE - Concrete transfer slab

Table 7 - 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 8 - 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 and Waterproofing

It is understood that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that the foundation wall will be blind poured against a drainage system and waterproofing system placed against the shoring face.

A waterproofing membrane will be required to lessen the effect of water infiltration for the basement levels starting at 5 m below finished grade. The waterproofing membrane can be placed and fastened to the shoring system (expected to be a soldier pile and timber lagging) and should extend to the bottom of the excavation at the founding level of the structural slab foundation.

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) extend down to the footing level. The purpose of the composite drainage system is to relieve any water infiltration resulting from a breach of the waterproofing membrane. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the structural slab interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Structural Slab Construction Joints

It is expected that the structural slab will be poured in sections. For the construction joint at each pour should incorporate a rubber water stop along with a chemical grout (Xypex or equivalent) applied to the entire vertical joint of the structural slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the structural slab.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, we recommend that 150 mm in diameter perforated pipes be placed along the interior perimeter of the foundation wall and one drainage line within each bay. The spacing of the underfloor drainage system should be confirmed at the time of backfilling the floor completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane system is recommended to lessen the effects of water infiltration. Any long term dewatering of the site will be minimal and should have no adverse effect to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor and an attempt will be made to grout or patch any areas with noticeable water infiltration.

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose.

Pressure Relief Chamber

The purpose of the pressure relief chamber will be to control the groundwater infiltration and hydrostatic pressure created by fully or partially tanking the basement level. To avoid uplift on the structural foundation slab prior to having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during the construction program.

During the construction program, the valve of the pressure relief chamber can be gradually closed as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

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 Temporary Shoring Requirements

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

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system will consist of a combination of soldier pile and lagging system for open areas such as roadways and parking lots and a hybrid system of interlocking steel sheet piling with soldier piles for areas adjacent or in close proximity to existing structures. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method. The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 9 - 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.

6.4 Excavation Side Slopes

For any potential open excavation sections, 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.

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.

6.5 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.

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

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

6.7 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.

6.8 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 final design from a geotechnical perspective.
- ❑ Observation of all pile installations and review of dynamic monitoring results.
- ❑ 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.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept. 17-2018
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

Report Distribution

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

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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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

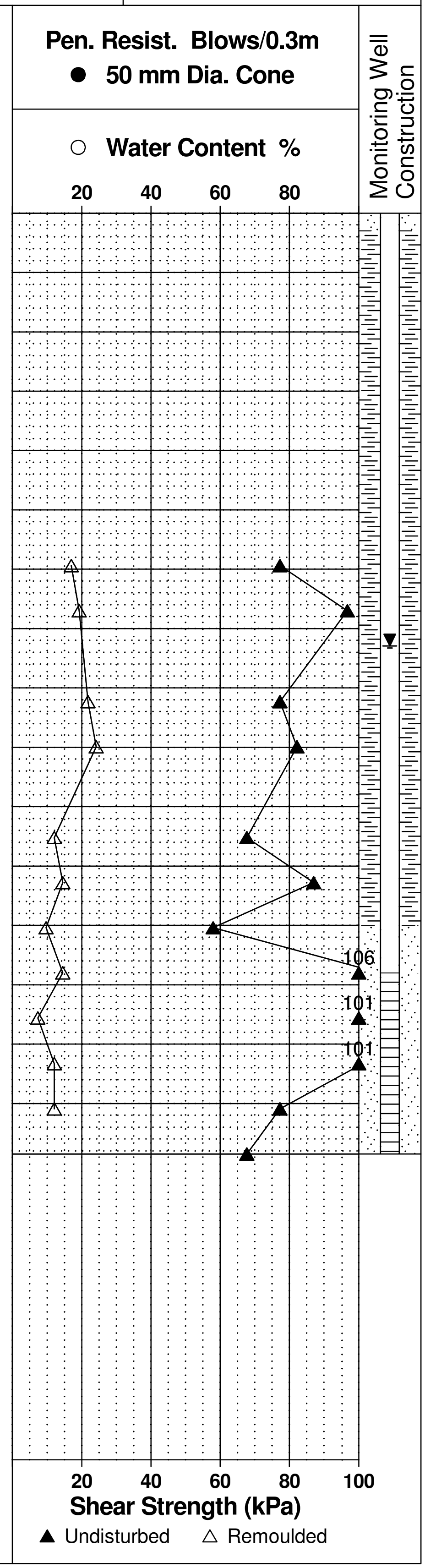
HOLE NO. BH1-18

SOIL DESCRIPTION (GROUND SURFACE)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Asphaltic concrete 0.08	AU	1		
FILL: Brown silty sand, some gravel, trace asphalt 1.62	SS	2	33	4
Very stiff to stiff, brown **CLAYEY SILT,** trace sand 4.57	SS	3	46	8
	SS	4	67	7
	SS	5	62	9
	SS	6	96	4
Stiff, grey **SILTY CLAY** 15.85	SS	7		2
	SS	8	96	1
	SS	9	96	W

End of Borehole

(GWL @ 7.29m - Aug. 7, 2018)

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



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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

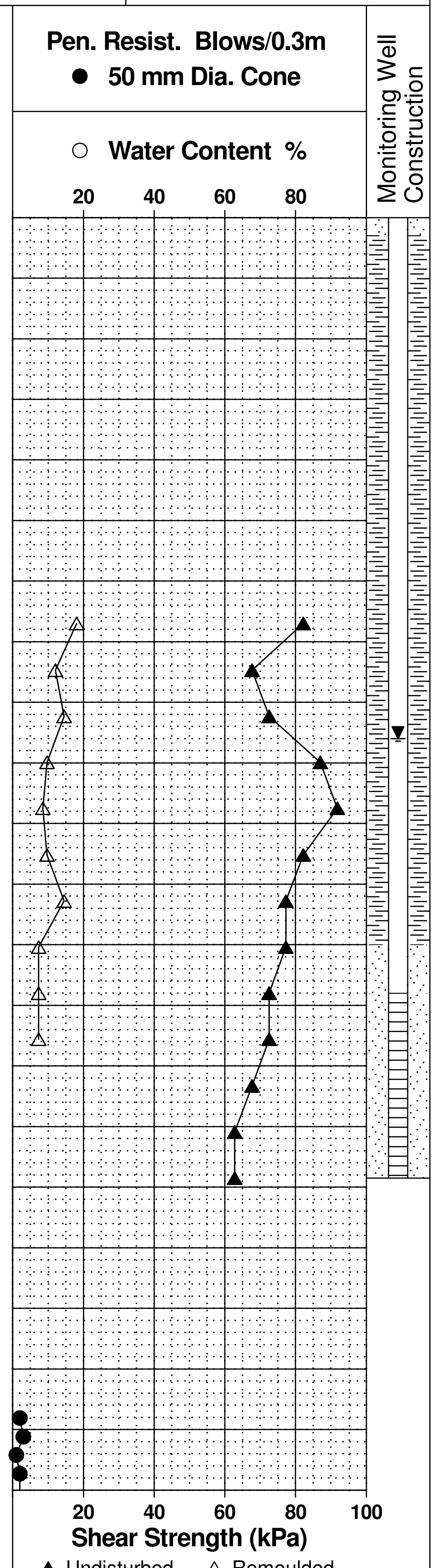
DATE July 26, 2018

FILE NO. PG4458

HOLE NO. BH2-18

SOIL DESCRIPTION	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



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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

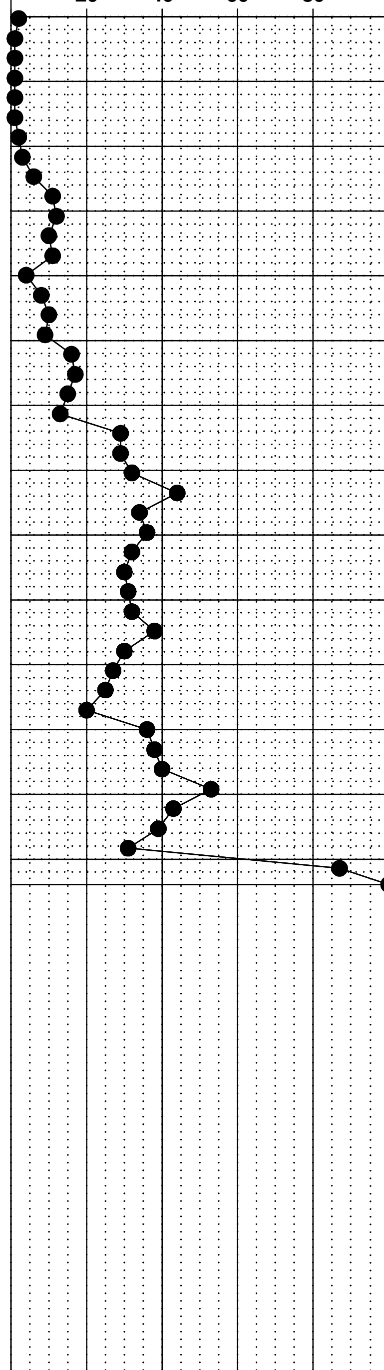
BORINGS BY CME 55 Power Auger

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 %	Monitoring Well Construction
GROUND SURFACE								20 40 60 80	
						21	49.25		
						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		
34.39						34	36.25		
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

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 27, 2018

FILE NO. PG4458

HOLE NO. BH3-18

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete (0.08)	AU	1		
FILL: Brown gravelly sand with silt	SS	2	46	26
	SS	3	62	60
	SS	4	75	40
	SS	5	67	20
	SS	6	46	13
	SS	7	38	11
- some clay by 5.3m depth	SS	8	29	3
(6.20)	SS	9		1
Stiff, grey **SILTY CLAY**	SS	10	42	W
	SS	11	42	W
(15.85)				
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 18.3m depth.				

DEPTH (m)	ELEV. (m)
0	70.17
1	69.17
2	68.17
3	67.17
4	66.17
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
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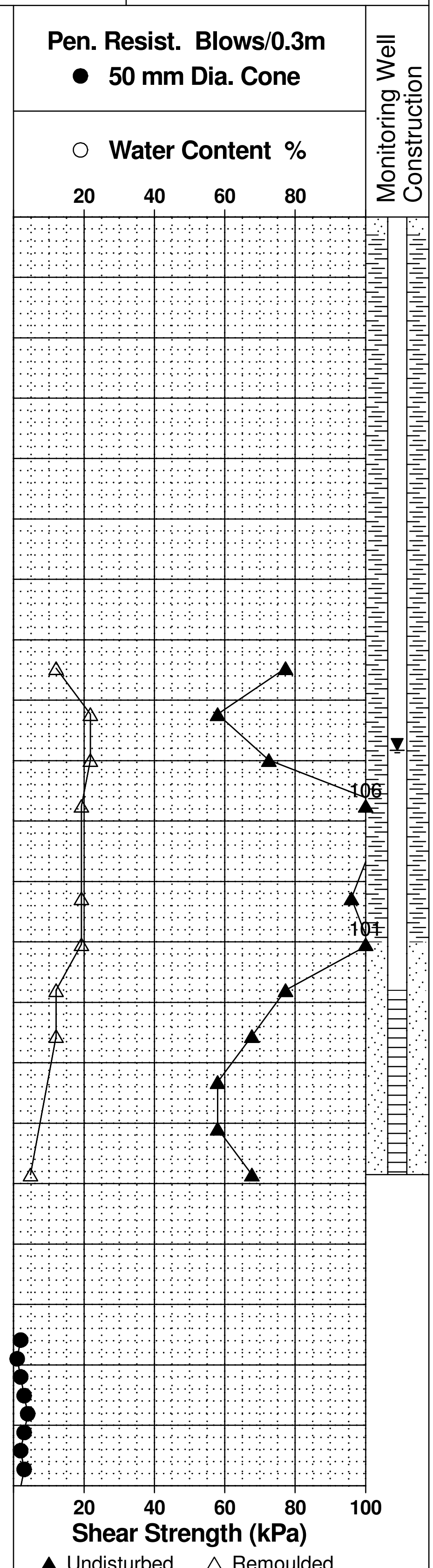
19	51.17
20	50.17
21	49.17



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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

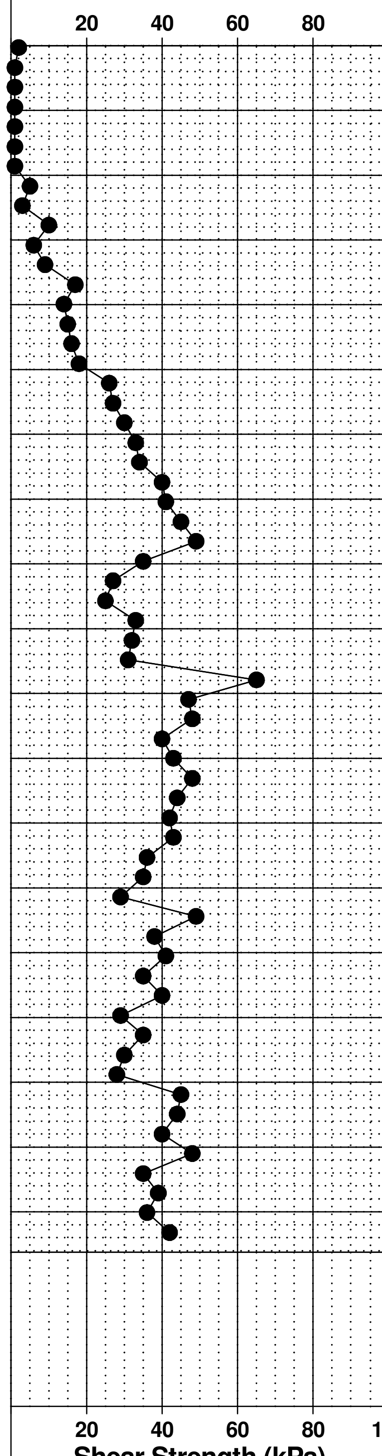
BORINGS BY CME 55 Power Auger

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 %	Monitoring Well Construction
GROUND SURFACE						21	49.17		
						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)									



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
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

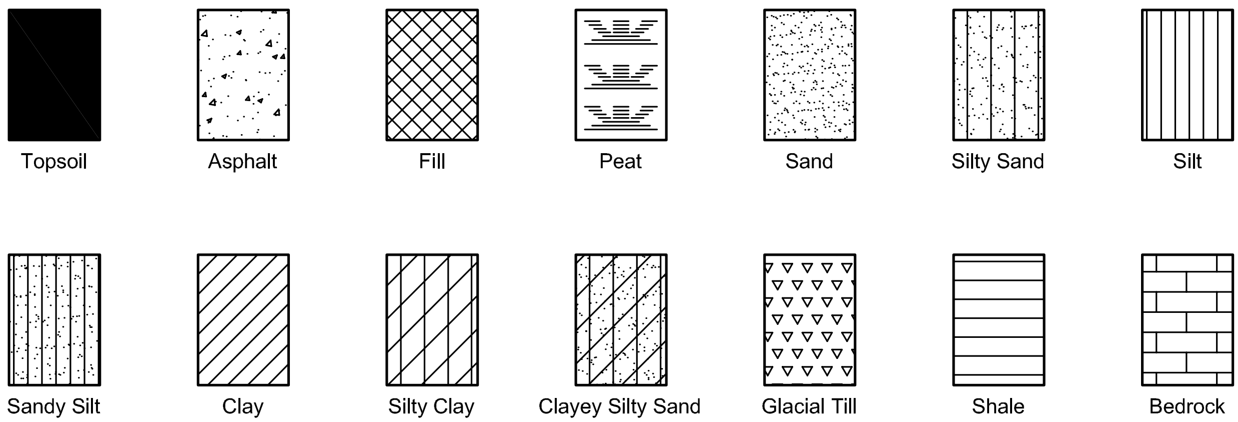
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

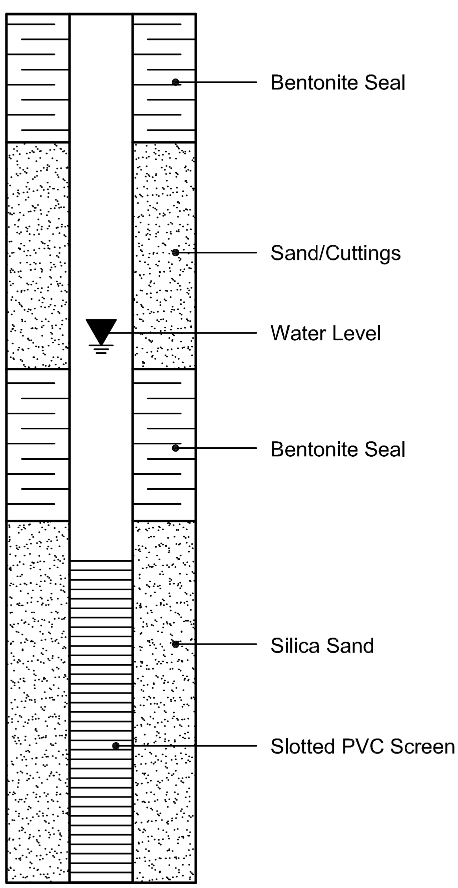
SYMBOLS AND TERMS (continued)

STRATA PLOT

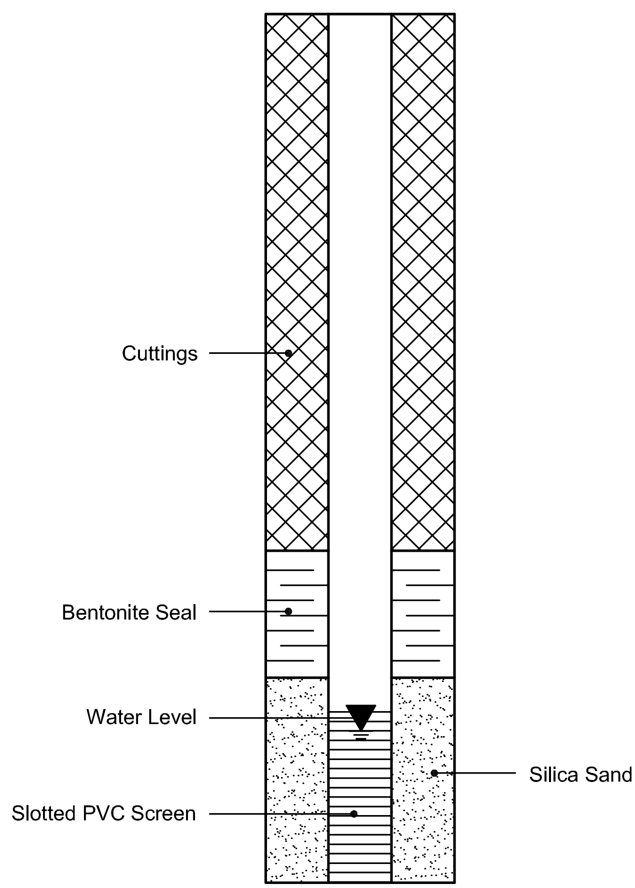


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1831094

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

FIGURE 2 - TYPICAL PRESSURE RELIEF CHAMBER

DRAWING PG4458-1 - TEST HOLE LOCATION PLAN

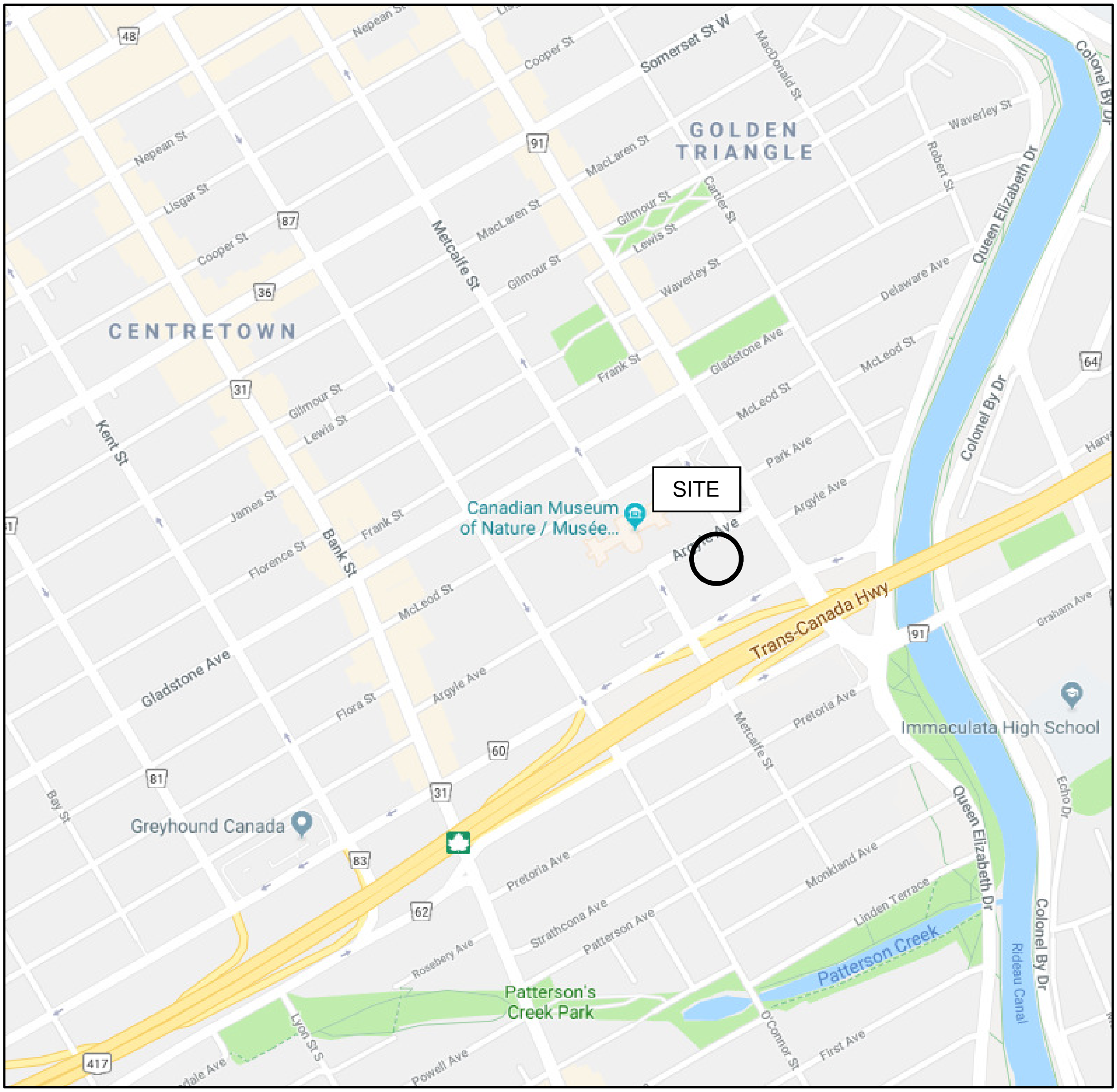


FIGURE 1

KEY PLAN

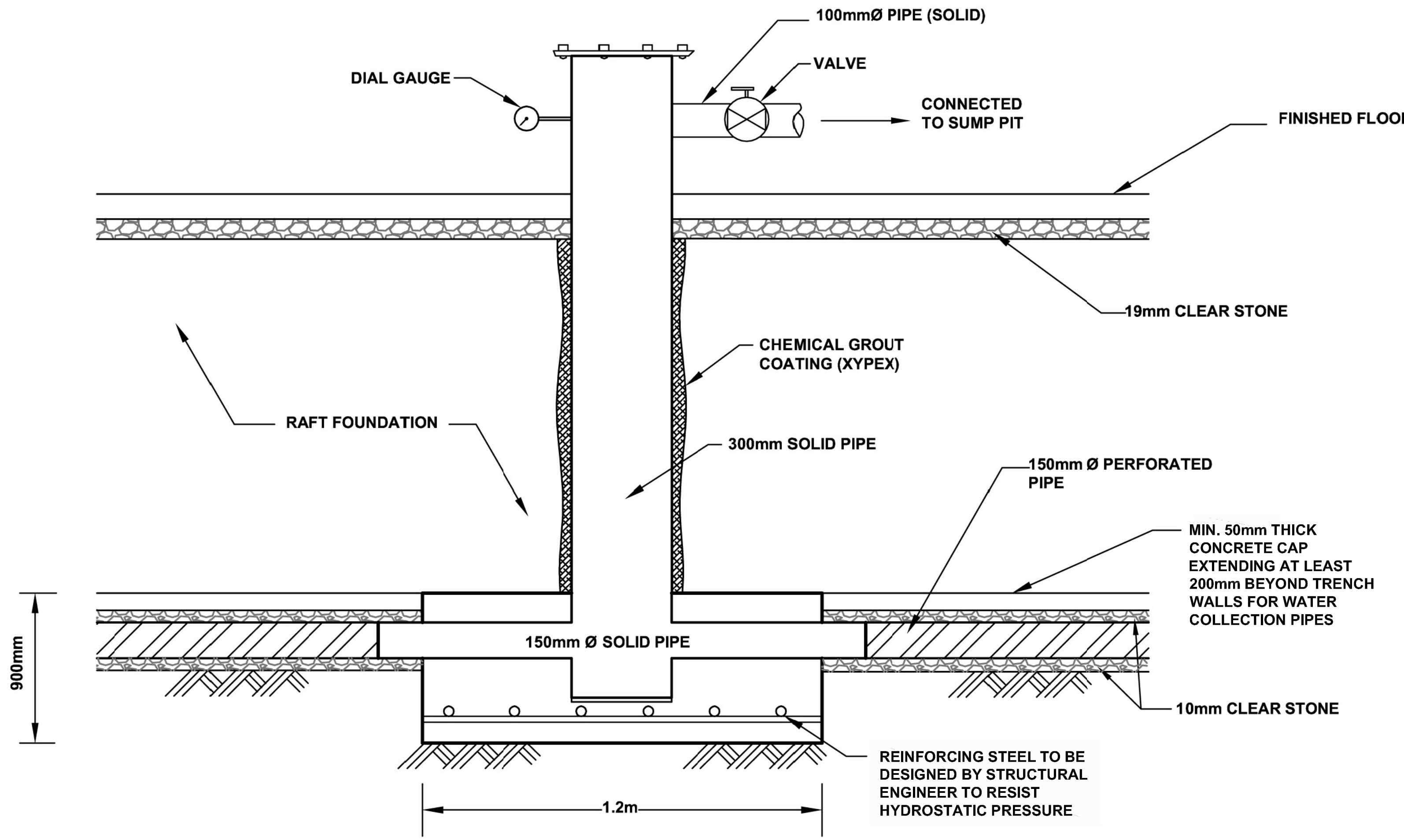


FIGURE 2 - PRESSURE RELIEF CHAMBER

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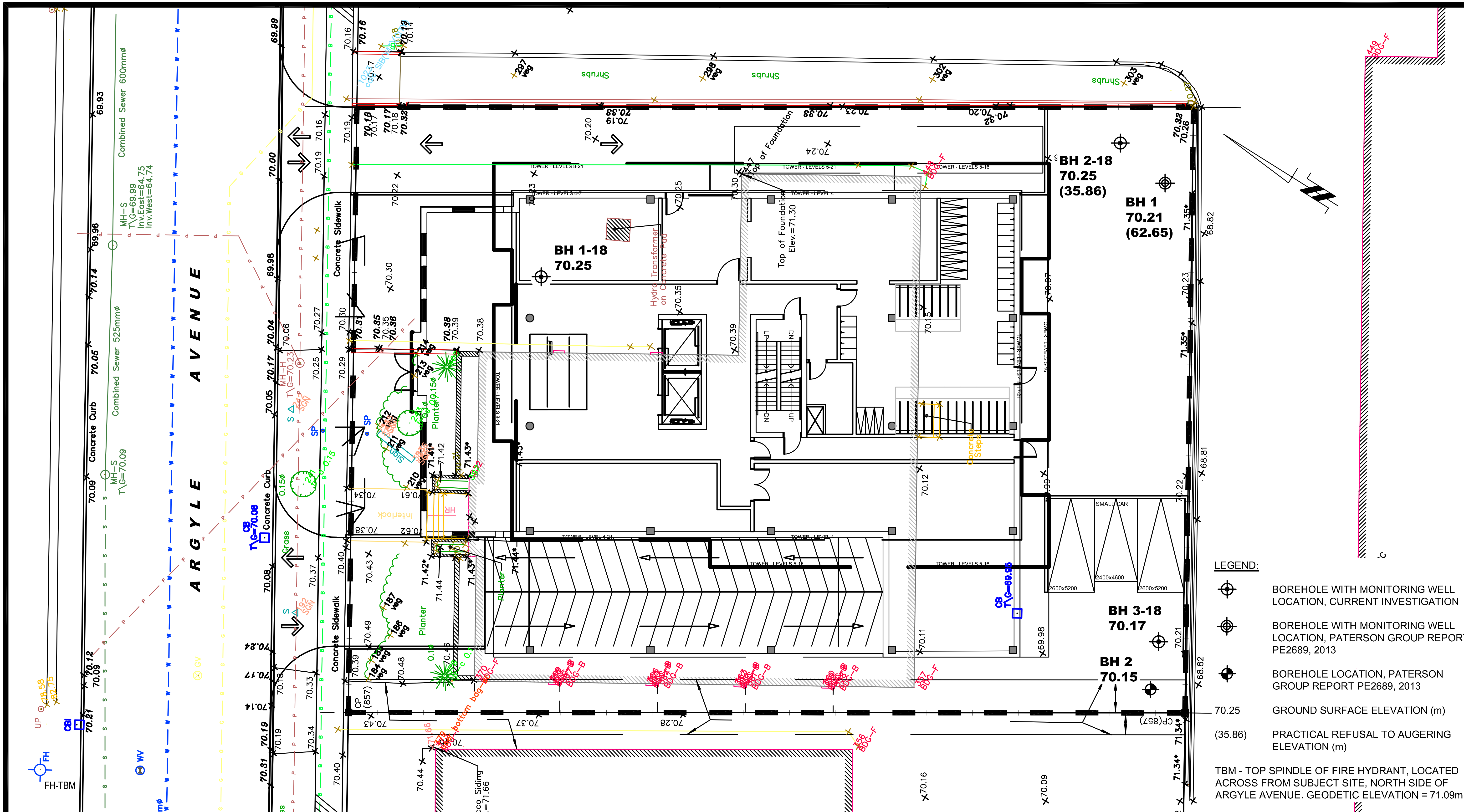

LEGEND:

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

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

NO.	REVISIONS	DATE	INITIAL
0			

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.: 0

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