Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation

Proposed Multi-Storey Building
390 Bank Street
Ottawa, Ontario

Prepared For

Urban Capital Property Group

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September 16, 2019

Report PG4996-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Urban Capital Property Group to conduct a geotechnical investigation for the proposed multi-storey building to be located at 390 Bank Street 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 9-storey mixed-use building with 2 levels of underground parking. Associated at-grade amenity and landscaped areas are also anticipated. It is understood that the first floor will be used for commercial purposes while the remaining storeys will be dedicated to residential use. 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 25, 2019. At that time, two (2) boreholes were advanced to a maximum depth of 17.5 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 taking into consideration existing site features and underground utilities. The borehole locations are shown on Drawing PG4996-1 - Test Hole Location Plan in Appendix 2.

The boreholes were completed with a truck-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.

Rock samples were recovered using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

The subsurface conditions observed in the 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

A 51 mm diameter rigid PVC monitoring well was 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 a manhole cover located at the northwest entrance to the subject site. A geodetic elevation of 71.77 m was provided for the TBM by Stantec Geomatics. The TBM and borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG4996-1 - 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 and a one-storey restaurant building of slab on grade construction with the associated paved and gravel-surfaced parking areas and access lanes. The ground surface across the subject site is flat and at grade with Bank and James Street and the neighboring properties.

The site is bordered by James Street followed by commercial buildings to the north, two three-storey commercial buildings to the west, a three-storey residential building followed by Florence Street to the south, and Bank Street followed by commercial buildings to the east.

4.2 Subsurface Profile

Overburden

Generally, the subsoil profile encountered at the borehole locations consists of a pavement structure or crushed stone fill overlying a brown silty sand fill with construction debris and trace clay. The fill is underlain by a firm to stiff grey-brown silty clay followed by a stiff, grey silty clay deposit. Glacial till was encountered below the silty clay layer consisting of dark brown silty clay with fragmented shale, sand and gravel. Shale bedrock was encountered in all boreholes at depths between 13.77 and 14.35 m below existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in the attachments 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 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on August 8, 2019. 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 coloring of the recovered samples, it is expected that the long-term groundwater level is between 4.5 and 5.5 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	71.79	5.81	65.98	August 8, 2019
BH 2	72.09	4.95	67.14	August 8, 2019
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 71.77 m provided by Stantec Geomatics.				

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. Due to the anticipated building loads either an end bearing pile foundation or a raft foundation are recommended for the subject site.

The proposed shoring system will have to take into consideration support of the existing structure due to the close proximity of the south neighbouring building.

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

5.2 Site Grading and Preparation

Stripping Depth

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

It is expected that the proposed underground parking levels will extend to a depth well within the native soils. Therefore, all surface soils will be removed as part of the excavation for the proposed structure.

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.

Protective Mud Slab

If a raft slab is selected to support the proposed building, it is recommended that a lean concrete mud slab be placed on the undisturbed silty clay subgrade surface to protect it from disturbance due to worker traffic. A 75 mm thick lean concrete mud slab (minimum 15 MPa 28-day compressive strength) is recommended to be poured over the undisturbed silty clay surface once exposed.

5.3 Foundation Design

Raft Foundation

For design purposes, the raft foundation base is assumed to be located at an approximately 7 m depth over an undisturbed, stiff silty clay bearing surface. The bearing medium will consist of a stiff grey silty clay which is susceptible to disturbance under construction traffic. The bearing surface should be protected to prevent disturbance as described above.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **160 kPa** can be used. 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 factored bearing resistance (contact pressure) at ULS can be taken as **250 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 **4.5 MPa/m** for a contact pressure of **160 kPa**. The design of the raft foundation 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 with the above parameters and a potential total and differential settlement of 25 and 20 mm, respectively.

Pile Foundation

If the raft slab bearing resistance values are insufficient for the proposed building, a deep foundation system driven to refusal in the bedrock will be 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.

Spread Footing Foundation (Lightly Loaded, External Structures)

Footings, up to 5 m wide, and strip footings, up to 3 m wide, founded on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed in the dry prior to the placement of concrete for footings.

The bearing resistance value at SLS for conventional style footings will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to provide adequate lateral support with respect to the excavations and different founding levels. Adequate lateral support is provided to a silty clay bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise

Based on the existing borehole coverage and results of the undrained shear strength testing completed within the underlying cohesive soils, a permissible grade raise restriction of **2.0 m** is provided for design purposes for the subject site.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations considered at this site. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, the native soil surface, approved by the Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill or a mud mat. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

A sub-floor drainage system, consisting of a series of perforated drainage pipes connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

A 75 mm thick layer of concrete mud slab should be placed to protect the native soil from construction equipment and traffic if the building will be founded on spread footings or a raft slab. A granular working mat up to 600 mm thick may be required for the support of pile driving equipment.

5.6 Basement Wall

There are several applicable combinations of backfill materials and retaining soils for the basement walls of the subject structure. However, the conditions should be designed by assuming the retaining 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 total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

The static horizontal earth pressure (P_o) should be calculated by a triangular earth pressure distribution equal to $K_o \gamma H$ where:

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) should be calculated using the earth pressure distribution equal to $0.375 a_c \gamma H^2/g$ where:

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

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

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of the pavement structure for the parking garage and flexible pavement structure for the access lanes.

Table 3 - Recommended Rigid Pavement Structure - Parking Garage

Thickness (mm)	Material Description
125	Reinforced Concrete Slab
300	BASE - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill or the raft foundation	

Table 4 - Recommended Pavement Structure - Access Lanes

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
400	**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 100% 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 preliminary information provided, it is expected that a portion of the proposed building foundation walls will be 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 primary groundwater infiltration control system.

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

- ❑ Place a suitable 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 beyond 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.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration for the proposed structure. For design purposes, Paterson recommends 150 mm diameter perforated pipes. 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 the 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

Footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through a stiff silty clay. Where excavation is above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used. The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

It is expected that deep service trenches in excess of 3 m will be completed using a temporary shoring system designed by a structural engineer, such as stacked trench boxes in conjunction with steel plates. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave (if required).

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

Temporary Shoring

The excavation support may consist of socketed soldier pile and lagging, or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be anchored or braced, if required. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 5.

Table 5 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.3
Passive Earth Pressure Coefficient (K_p)	3.3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	18
Submerged Unit Weight(γ'), kN/m^3	13

The total unit weight should be used above the waterproofing level while the submerged or effective unit weight should be used below the waterproofing level. The hydrostatic groundwater pressure should be added to the earth pressure distribution below the waterproofing level. Conventional braced excavation pressure envelopes can also be used by the shoring designer, as applicable.

Generally, it is anticipated that the shoring systems will be driven to refusal and provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the grey silty clay or below the long term groundwater level, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the 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) Category 3 may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the 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.

Impacts on Neighboring Structures

Based on our observations, the long term groundwater level is expected at a depth ranging from 4.5 to 5.5 m below the existing grade and within the silty clay deposit. Based on our observations, no groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighboring structures are expected to be founded within the silty clay deposit. Issues are not 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 subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

The trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

The foundation of adjacent buildings should be protected. The proposed excavation could reduce the existing earth frost cover to an unacceptable thickness. For situations where this could occur, such as shoring, it is recommended to reduce frost penetration and potential unexpected settlement.

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:

❑ 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 Urban Capital Property Group 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.

Faisal I. Abou-Seido, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

- ❏ Urban Capital Property Group (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
Proposed Multi-Storey Building - 390 Bank Street
Ottawa, Ontario

DATUM TBM - Top of catch basin near the northwest corner of property, along James Street. Geodetic elevation = 71.77m, as provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 July 25

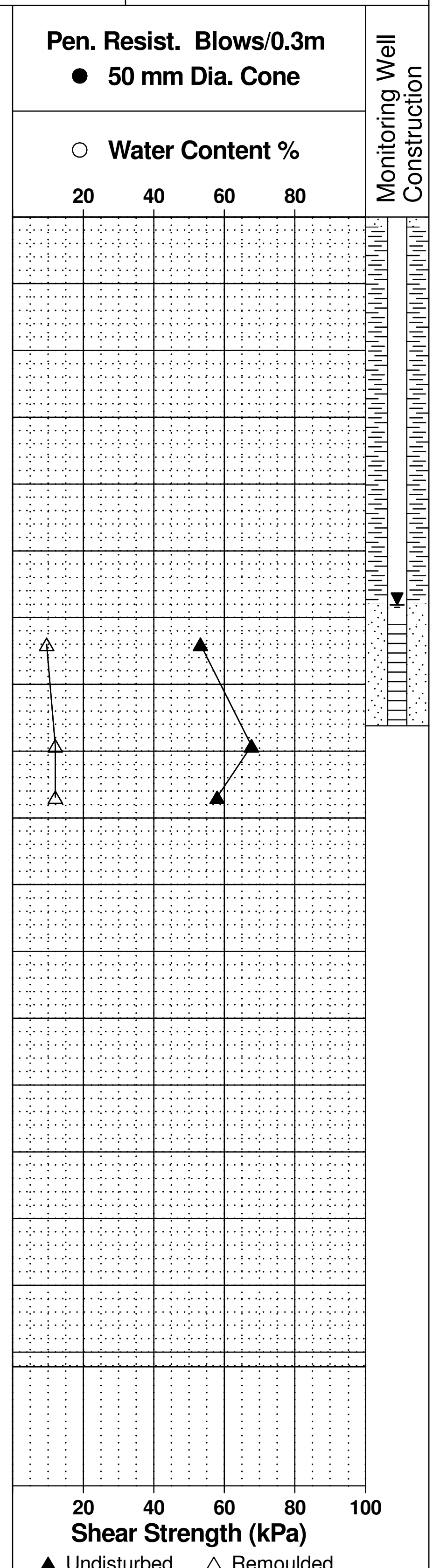
FILE NO. PG4996

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel	0.91	AU	1		
FILL: Dark brown silty sand, trace concrete and construction debris	1.42				
Firm to stiff, grey-brown **SILTY CLAY,** trace sand - grey by 4.3m depth	10.67	SS	2	58	6
		SS	3	100	2
		SS	4	100	2
		SS	5	67	5
		SS	6	21	12
GLACIAL TILL: Dark brown silty clay with fragmented shale, sand and gravel	13.77	SS	7	58	17
		SS	8	54	15
		SS	9	75	33
		SS	10	79	27
		SS	11	100	50+
BEDROCK: Very poor quality, black shale	17.22	RC	1	100	36
		RC	2	100	19
		RC	3	100	11
End of Borehole					

(GWL @ 5.81m - August 8, 2019)

DEPTH (m)	ELEV. (m)
0	71.79
1	70.79
2	69.79
3	68.79
4	67.79
5	66.79
6	65.79
7	64.79
8	63.79
9	62.79
10	61.79
11	60.79
12	59.79
13	58.79
14	57.79
15	56.79
16	55.79
17	54.79



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 390 Bank Street
Ottawa, Ontario

DATUM TBM - Top of catch basin near the northwest corner of property, along James Street. Geodetic elevation = 71.77m, as provided by Stantec Geomatics Ltd.

REMARKS

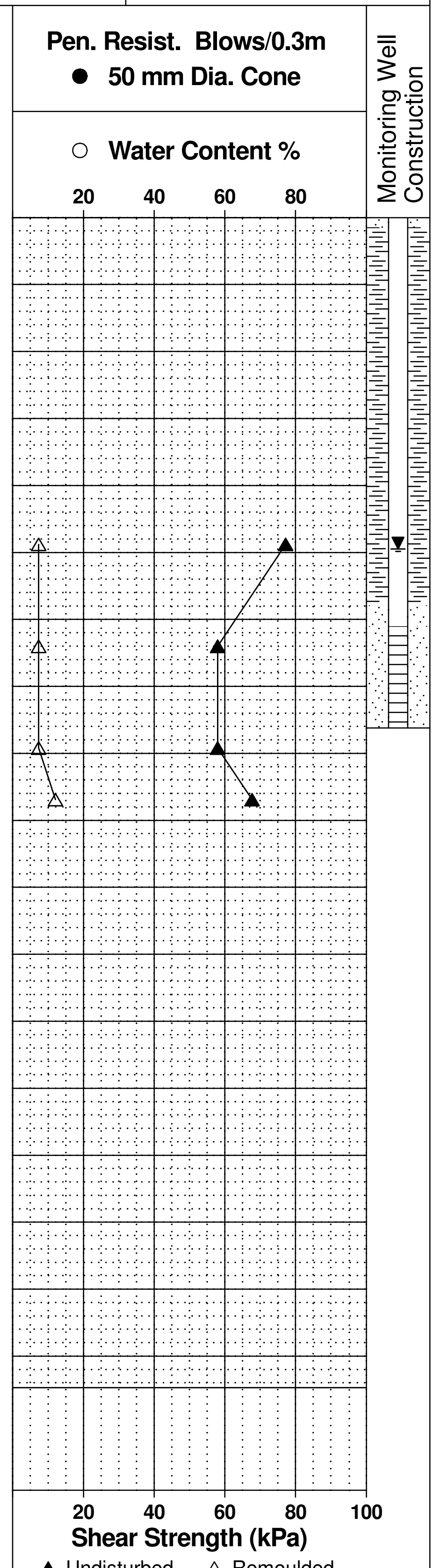
BORINGS BY CME 55 Power Auger

DATE 2019 July 25

FILE NO. PG4996

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.09
Asphaltic concrete	0.04	AU	1				
FILL: Brown silty sand with gravel	0.91					1	71.09
FILL: Brown silty sand, trace clay	2.44	SS	2	33	7	2	70.09
Stiff, grey-brown **SILTY CLAY** - grey by 4.0m depth	9.45	SS	3	100	2	3	69.09
						4	68.09
						5	67.09
						6	66.09
						7	65.09
						8	64.09
						9	63.09
GLACIAL TILL: Dark brown silty clay with fragmented shale, trace sand and gravel	14.35	SS	4	33	5		
		SS	5	62	3	10	62.09
		SS	6	46	23	11	61.09
		SS	7	54	22		
		SS	8	58	20	12	60.09
		SS	9	50	34	13	59.09
		SS	10	75	39	14	58.09
BEDROCK: Poor quality, black shale	17.47	RC	1	100	31	15	57.09
		RC	2	100	33	16	56.09
		RC	3	96	36	17	55.09
End of Borehole (GWL @ 4.95m - August 8, 2019)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

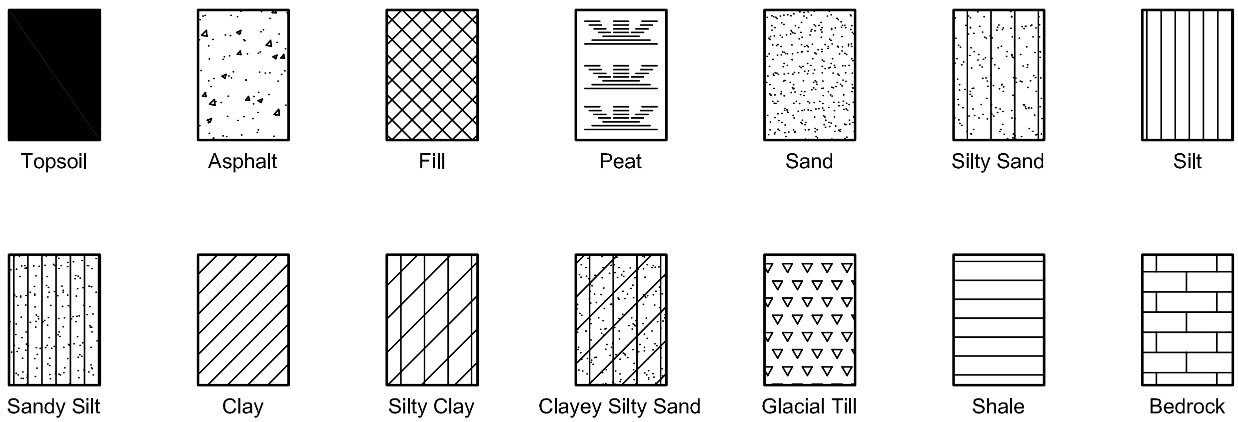
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

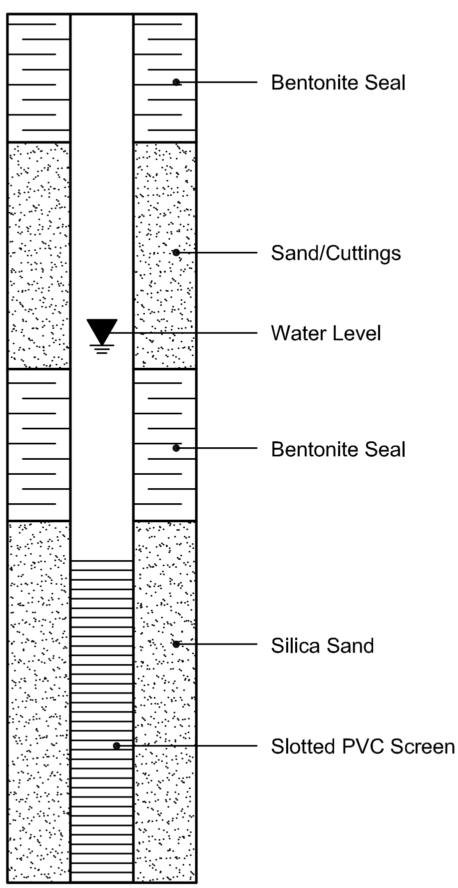
SYMBOLS AND TERMS (continued)

STRATA PLOT

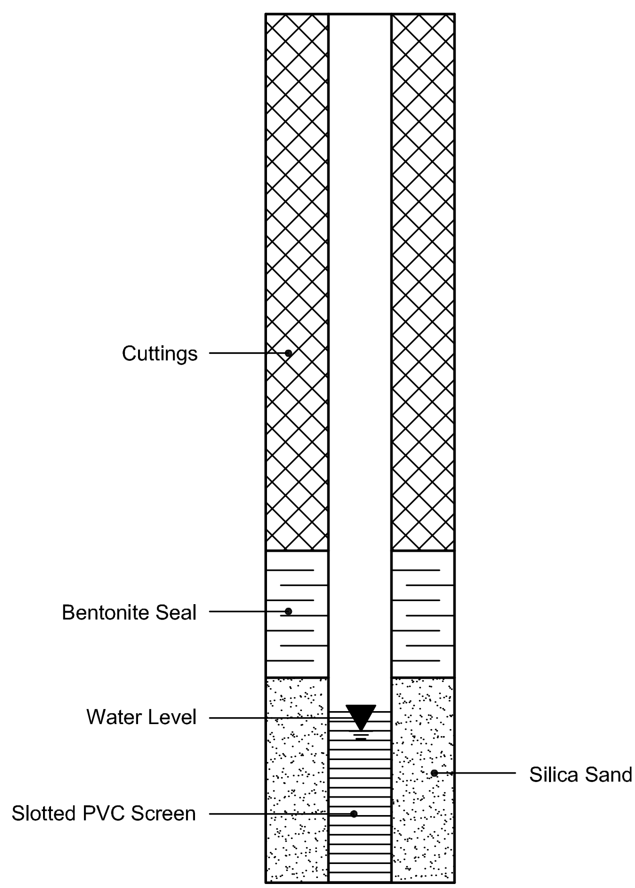


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 25646

Report Date: 15-Aug-2019
Order Date: 9-Aug-2019
Project Description: PG4996

	MDL/Units				
Client ID:		BH1-SS4-15'-17'	-	-	-
Sample Date:		25-Jul-19 13:00	-	-	-
Sample ID:		1932532-01	-	-	-
		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	58.9	-	-	-
General Inorganics					
pH	0.05 pH Units	8.26	-	-	-
Resistivity	0.10 Ohm.m	7.44	-	-	-
Anions					
Chloride	5 ug/g dry	647	-	-	-
Sulphate	5 ug/g dry	244	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4996-1 - TEST HOLE LOCATION PLAN

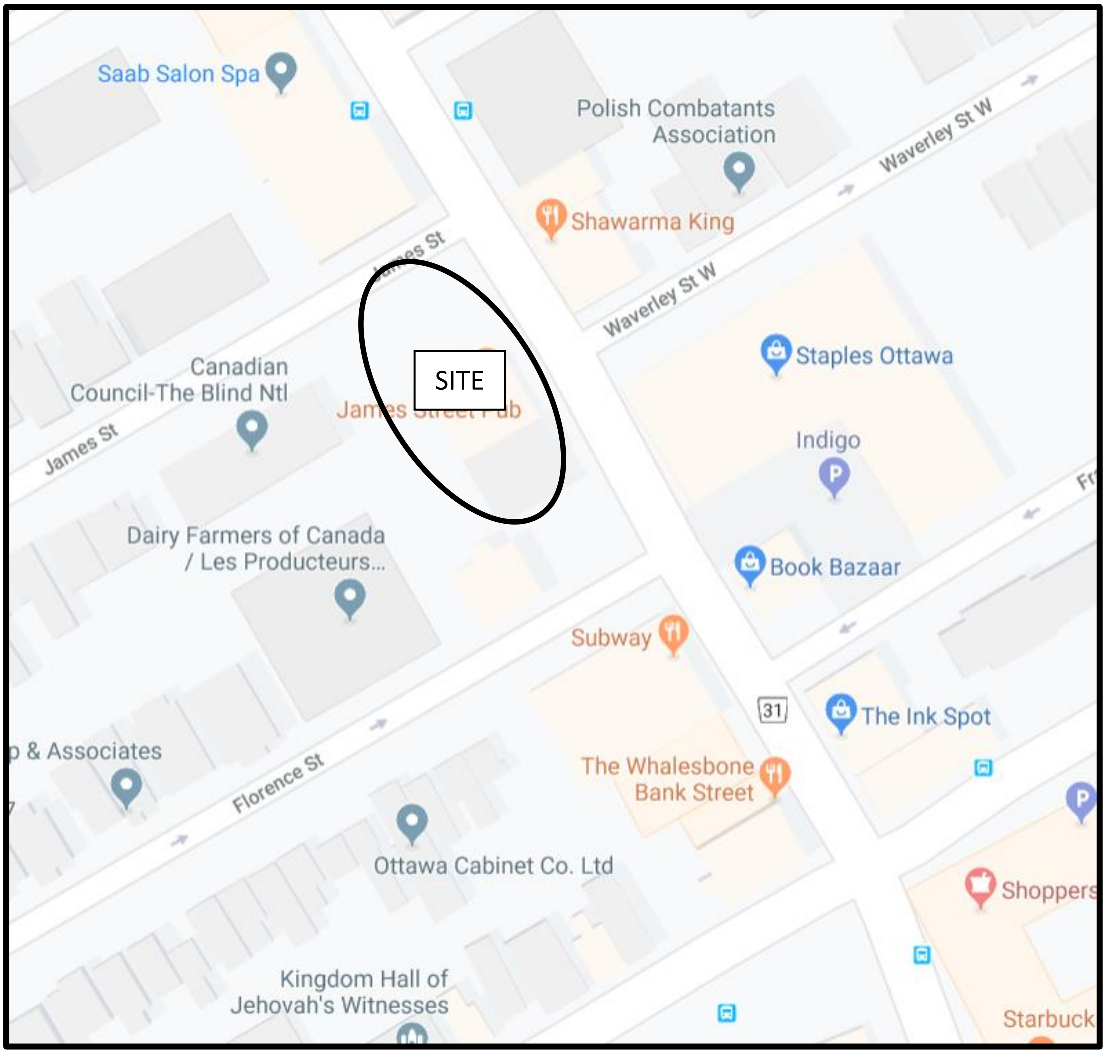
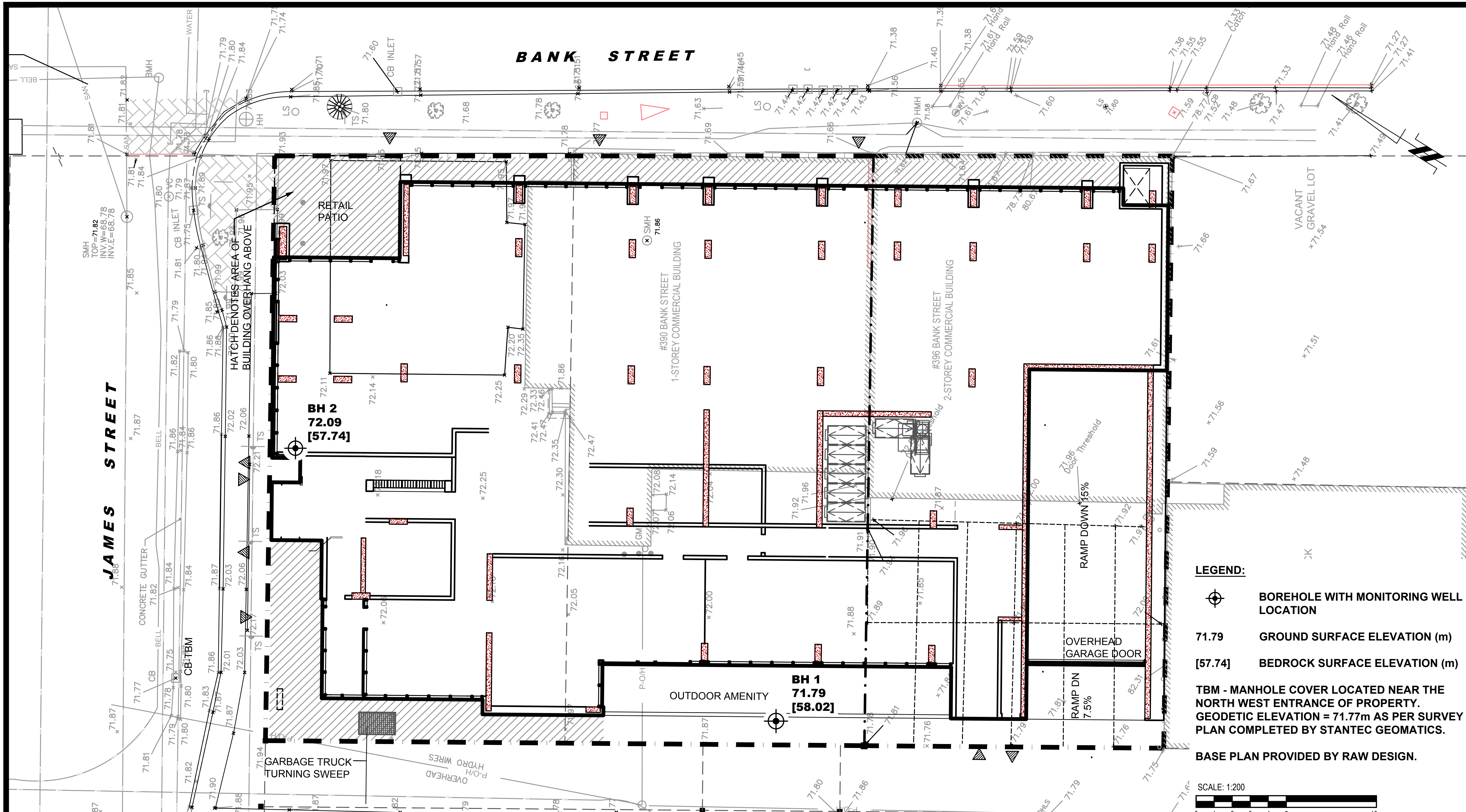


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

- BOREHOLE WITH MONITORING WELL LOCATION
- 71.79 GROUND SURFACE ELEVATION (m)
- [57.74] BEDROCK SURFACE ELEVATION (m)

TBM - MANHOLE COVER LOCATED NEAR THE NORTH WEST ENTRANCE OF PROPERTY. GEODETIC ELEVATION = 71.77m AS PER SURVEY PLAN COMPLETED BY STANTEC GEOMATICS.

BASE PLAN PROVIDED BY RAW DESIGN.

SCALE: 1:200

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

URBAN CAPITAL PROPERTY GROUP

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 390 BANK STREET

OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale: 1:200	Date: 08/2019
Drawn by: MPG	Report No.: PG4996-1
Checked by: FA	PG4996-1
Approved by: FA	Revision No.:

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