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Geotechnical Investigation

Proposed Multi-Storey Building
275 King Edward Avenue
Ottawa, Ontario

Prepared For

165177 Canada Inc

April 26, 2021

Report: PG5721-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 165177 Canada Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located on 275 King Edward Avenue in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.
- Provide geotechnical recommendations pertaining to 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed project will consist of a multi-storey building having up to 8 stories and three underground parking levels. Associated at-grade parking and landscaped areas are also anticipated as part of the development. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on April 8, 2021 and consisted of advancing a total of 2 boreholes to a maximum depth of 7.0 m below existing ground surface. A previous investigation was also completed by others. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5721-1 - Test Hole Location Plan included in Appendix 2.

The test holes were put down using a track mounted 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.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. 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.

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

Undrained shear strength testing was conducted in cohesive soils using a field vane apparatus.

The thickness of the silty clay layer was evaluated during the course of the current investigation by dynamic cone penetration testing (DCPT) completed at BH 1-21. The DCPT consisted of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. the number of blows required to drive the cone tip into the soil is recorded for each 300 mm increments.

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

Groundwater

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5721-1 - 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. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures, one of which was collected from test hole BH 2-21. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The property was formerly occupied by several residential buildings with either slab-on-grade construction or single basement levels. The ground surface across the site and adjacent properties is level.

The site is bordered to the north partially by Murray Street and an existing residential dwelling, to the east by low rise residential buildings, to the south by Clarence Street and to the west by King Edward Street.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil and fill overlying very stiff to stiff brown silty clay deposit, becoming grey below ~4.0m. A compact to dense, grey glacial till deposit was inferred beneath the silty clay deposit from the DCPT completed in BH 1-21. Practical refusal to DCPT was encountered in BH 1-21 at a depth of 11m below existing ground surface.

Based on the RQD values from the recovered rock core from the previous investigation, a limestone bedrock of excellent quality was encountered below the glacial till. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, and refusal to DCPT, the bedrock in the subject area consists of limestone and shale of the Verulam formation, with an overburden drift thickness of 5 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured within the installed piezometers during the current investigation on April 14, 2021. Also, the groundwater level was reported during the previous investigation by others.

Based on the groundwater readings, the groundwater levels recorded ranged from 2.6 to 5.3 m below existing ground surface. The measured groundwater levels are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels				
Test Hole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level / Groundwater Infiltration for Test Holes**		**Dated Recorded**
		Depth (m)	**Elevation (m)**	
Current Investigation				
BH 1-21	58.10	5.27	52.83	April 14, 2021
BH 2-21	58.23	2.56	55.67	
Previous Investigation (2007)				
BH 1	58.33	3.4	54.93	March 7, 2007
BH 2	58.54	-	-	
Note: The ground surface elevation at each borehole location during the current investigation was surveyed using a handheld GPS using a geodetic datum.				

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3.0 to 4.0 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is anticipated that the proposed building will have three levels of underground parking. Therefore, it is recommended that the proposed building be supported on conventional spread footings bearing on a clean, bedrock bearing surface or lean concrete in-filled trench extended to the bedrock surface for areas where the bedrock surface extends below the design underside of footing level for the proposed building.

Alternatively, the building can be founded on a raft foundation placed on undisturbed compact to dense glacial till bearing surface.

To complete the construction of the underground levels, temporary shoring will be needed to support the neighboring buildings and roads. The design of the temporary shoring system needs to adequately support the existing low-rise buildings along the east side of the site, which are in close proximity with the proposed excavation.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Based on the anticipated excavation depth, all topsoil and fill materials will be removed from within the perimeter of the proposed building.

Fill Placement

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

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

5.3 Foundation Design

Bearing Resistance Values

Bedrock Bearing Surface

Footings placed on a clean, surface sounded bedrock surface, or on lean concrete in-filled trenches extended to a clean, surface sounded bedrock surface can be designed using a bearing resistance value at ultimate limits states (ULS) of **3,000 kPa**. A geotechnical factor of 0.5 was applied to the above noted bearing resistance value.

Where the bedrock extends below the underside of footings elevation, the subgrade should be sub-excavating below the underside of footings down to a clean surface sounded bedrock. The side walls of the zero entry, vertical trenches can be used as the forms for the concrete to be poured. The subgrade should be reviewed and approved by Paterson personnel at the time of excavation, prior to pouring the concrete. The vertical trenches can be in-filled with a minimum 15 MPa lean concrete (28 day strength) up to the proposed underside of footing elevation.

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

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Overburden Bearing Surface

If the founding depth is less than 11m, then the proposed building can be founded on a raft foundation placed over the undisturbed compact to dense, glacial till bearing surface. In this case, the bearing resistance value at SLS (contact pressure) of **300 kPa** can be used for design purposes. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **500 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 **10 MPa/m** for a contact pressure of **300 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to native soil when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium. In unfractured bedrock, a plane with a slope of 1H:6V can be used.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are attached to the present letter report.

Field Program

The seismic array testing location was placed within the east portion of the subject site in a north-south direction as presented in Drawing PG5721-1 - Test Hole Location Plan attached to the present letter report. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between five (5) to ten (10) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3, 4.5 and 10 m away from the first and last geophone and at the centre of the seismic array.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

It is anticipated that the proposed building will be founded directly on bedrock. Based on our testing results, the bedrock shear wave velocity is 2,072 m/s.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2,072m/s}\right)}$$

$$V_{s30} = 2,072m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for foundations placed on bedrock is 2,072 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building founded on bedrock, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native soils and bedrock surface will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

If a raft slab is considered, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For a floor slab bearing on bedrock, it is recommended that a minimum 300 mm thick layer of 19mm crushed stone compacted to at least 95% of its SPMDD be present between the floor slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

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 bulk (drained) unit weight of 20 kN/m3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.28 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Design

Car only parking areas, access lanes, heavy truck parking areas and rigid pavement are expected at this site. The subgrade material will consist of fill and native soil. The proposed pavement structures are presented in Tables 2, 3 and 4.

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 I or II material.

Table 2 – Recommended Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil.	

Table 3 – Recommended Rigid Pavement Structure

Thickness (mm)	Material Description
125	**Wear Course –** Concrete Slab
200	**BASE –** OPSS Granular A Crushed Stone
Subgrade – Concrete Transfer Slab	

Table 4 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
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.	

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

The pavement granulars (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 compaction 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 located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which breaches the primary ground infiltration control system.

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

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

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

It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Underfloor Drainage

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

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

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

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

The footings located along parking garage entrance may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. Insufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Therefore, a temporary shoring system will be required to complete the proposed excavation.

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site 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.

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.

Temporary Shoring

Temporary shoring is required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of 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 temporary shoring system may be calculated with the following parameters.

Table 5 – Soils Parameter for Shoring System Design	
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
Unit Weight (γ), kN/m^3	20
Submerged 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 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 upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of 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.

Permit to Take Water

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

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

Long-term Groundwater Control

Any groundwater encountered along the building perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

The subsoil conditions at this site 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 and 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.

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. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be 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

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Observation of all bearing surfaces prior to the placement of concrete.
- Inspection of all below grade waterproofing and foundation drainage systems
- Sampling and testing of the concrete and fill materials.
- 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 the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 165177 Canada Inc or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 26, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

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

David J. Gilbert, P.Eng

Report Distribution:

❑ 165177 Canada Inc (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
Prop. Multi-Storey Building - 275 King Edward Avenue
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5721**

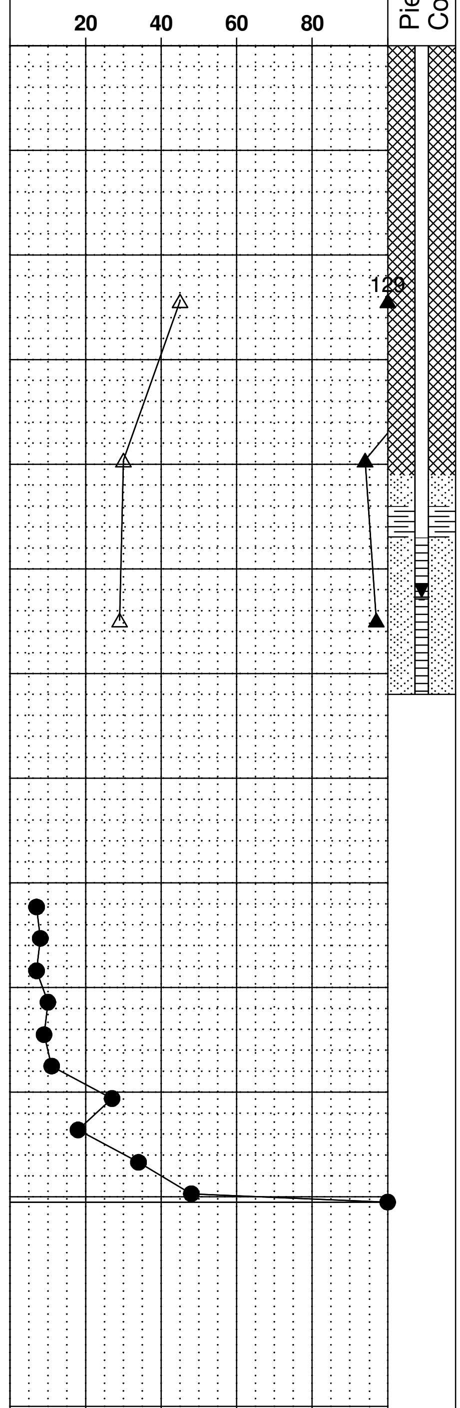
REMARKS

HOLE NO. **BH 1-21**

BORINGS BY Track-Mount Power Auger **DATE** April 8, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.10
TOPSOIL 0.25	AU	1				
FILL: Brown silty sand, some gravel and topsoil 1.37	AU	2				
	SS	3	25	8	1	57.10
Very stiff to stiff, brown **SILTY CLAY**	SS	4	79	8	2	56.10
	SS	5	100	1	3	55.10
- grey by 3.8m depth					4	54.10
	SS	6	100	2	5	53.10
6.15					6	52.10
Dynamic Cone Penetration Test commenced at 6.15m depth. Cone pushed to 8.1m depth.					7	51.10
Inferred **SILTY CLAY**					8	50.10
					9	49.10
9.75					10	48.10
Inferred **GLACIAL TILL** 11.05					11	47.10
End of Borehole						
Practical DCPT refusal at 11.05m depth						
(GWL @ 5.27m - April 14, 2021)						

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



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 275 King Edward Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 8, 2021

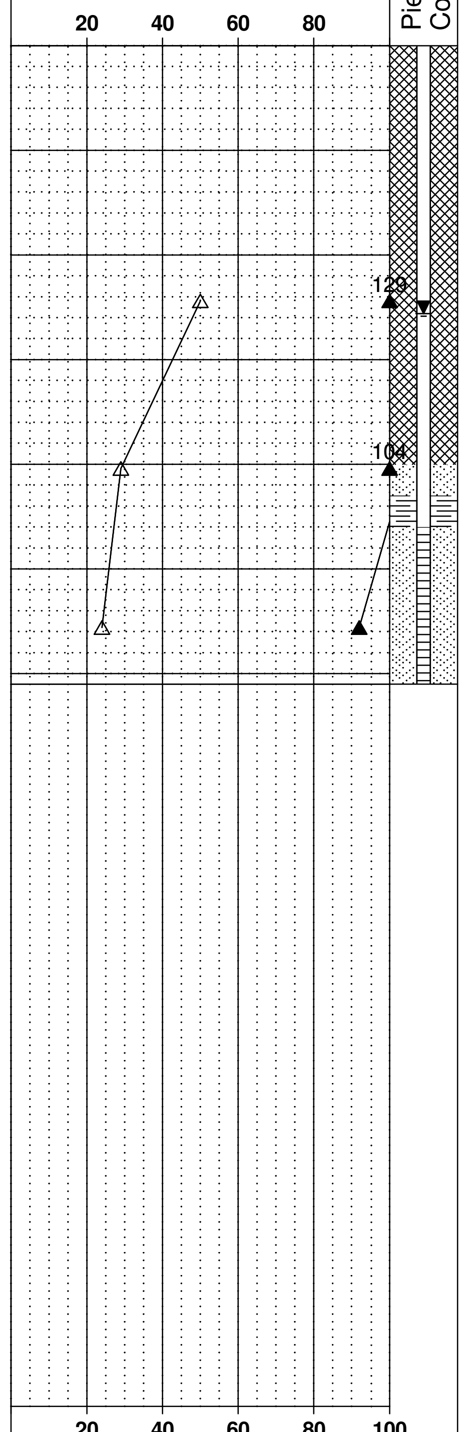
FILE NO. PG5721

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.23
TOPSOIL 0.08		AU	1				
FILL: Brown silty sand, some gravel, crushed stone, brick and concrete fragments		AU	2				
		SS	3	25	11	1	57.23
1.37							
Very stiff to stiff, brown **SILTY CLAY**		SS	4	79	14	2	56.23
						3	55.23
		SS	5	83	2		
- grey by 4.3m depth						4	54.23
		SS	6	83	2	5	53.23
6.10						6	52.23
End of Borehole							
(GWL @ 2.56m - April 14, 2021)							

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

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 x D60)
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

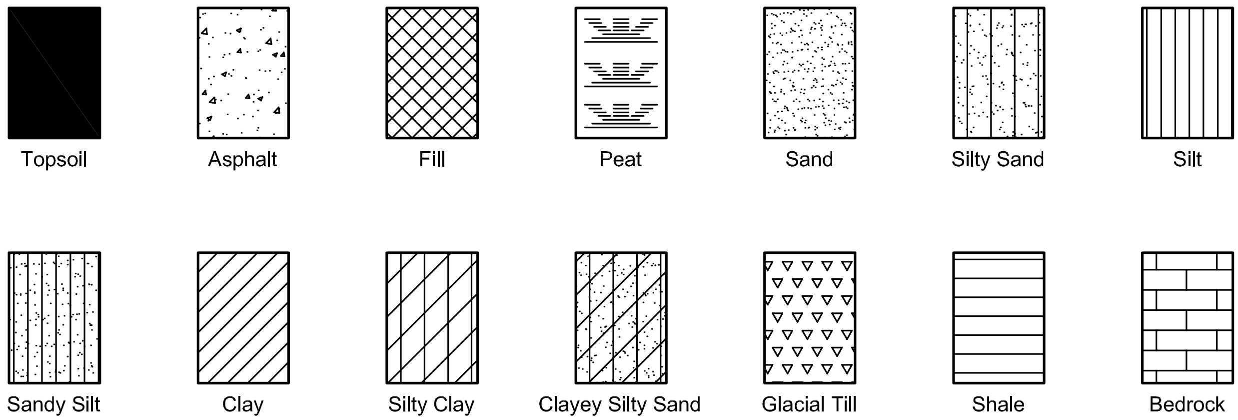
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

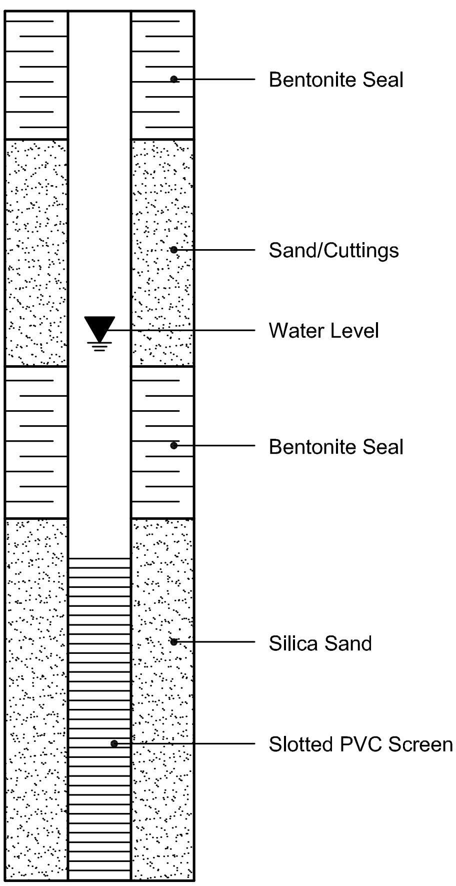
SYMBOLS AND TERMS (continued)

STRATA PLOT

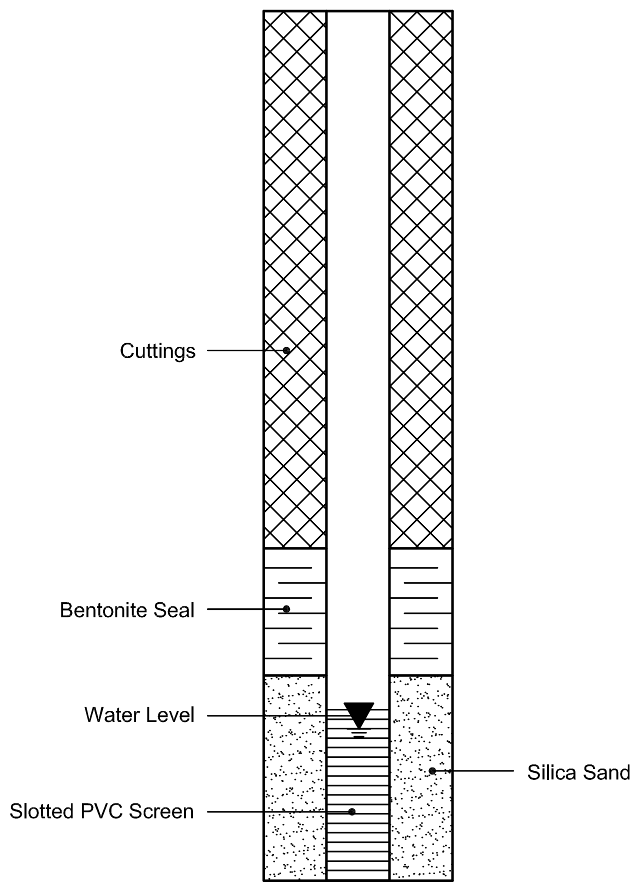


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



REFERENCE No.: T020191-A1

ENCLOSURE No.: 1

INSPEC•SOL

BOREHOLE No.: BH1

ELEVATION: 58.3 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Claude Lauzon

PROJECT: Proposed Five Storey Apt Building

LOCATION: 260 Murray St, Ottawa, Ontario

DESCRIBED BY: M. Patterson CHECKED BY: C.Kamichaitis

DATE (START): February 6, 2007 DATE (FINISH): February 6, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (metres)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Unit Weight (kg/m³)	Penetration Index / RQD (N)
	58.3	GROUND SURFACE				
		Granular foundation, crushed stone (20mm), brown, frozen				
1.0		Fill - Sand, some silt, loose, brown, moist	SS1	50		6
		Silty clay, grey, moist and stiff	SS2	50		5
2.0		Silt, some clay and sand, loose, grey, very moist	SS3	63		6
3.0	WL3.4	Becoming wet	SS4	88		4
4.0			SS5	100		4
5.0		Becoming saturated	SS6	100		4
			SS7	50		4
6.0			SS8	100		2
7.0		Becoming sandy, trace gravel	SS9	100		4
8.0			SS10	100		2
			SS11	100		4
9.0		Sandy Silt, some clay and gravel, compact, grey, saturated	SS12	54		20
10.0			SS13	17		22
11.0			SS14	67		20
		End of borehole with auger refusal at 11.3m				
12.0						
13.0						

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

REFERENCE No.: T020191-A1

ENCLOSURE No.: 2

INSPEC-SOL

BOREHOLE No.: BH2
ELEVATION: 58.5 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Claude Lauzon
PROJECT: Proposed Five Storey Apt Building
LOCATION: 260 Murray St, Ottawa, Ontario
DESCRIBED BY: M. Patterson   CHECKED BY: C.Kamichaitis
DATE (START): February 6, 2007   DATE (FINISH): February 6, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (metres)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Unit Weight (kg/m³)	Penetration Index / RQD (N)
	58.5	GROUND SURFACE				
		Granular foundation, crushed stone (20mm), brown, frozen				
1.0		Some silt, very loose, topsoil inclusions, brown, moist	SS1	25		3
		Silty clay, some sand, some silt and brick debris, brown, moist	SS2	42		6
2.0			SS3	100		10
		Silt, some clay and sand, loose, grey, very moist	SS4	100		6
3.0		Becoming wet	SS5	100		4
4.0			SS6	100		4
5.0		Becoming saturated	SS7	100		4
6.0			SS8	100		5
			SS9	100		4
7.0			SS10	100		4
8.0		Becoming sandy, trace gravel	SS11	100		4
9.0			SS12	100		4
		Sandy silt, some clay and gravel, compact, grey, saturated	SS13	88		13
10.0			SS14	67		23
11.0			SS15	25		24
12.0		Limestone with interbedded shale layers, grey, excellent quality	RC16	91		91
13.0		Borehole terminated at 13.1m				

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

BOREHOLE LOG T020191-A1.GPJ INSPEC_SOL.GDT 3/20/07

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 32981

Report Date: 14-Apr-2021

Order Date: 9-Apr-2021

Project Description: PG5721

	Client ID:	BH2-21 SS4	-	-	-
	Sample Date:	08-Apr-21 09:00	-	-	-
	Sample ID:	2115641-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.84	-	-	-
Resistivity	0.10 Ohm.m	32.4	-	-	-
Anions					
Chloride	5 ug/g dry	10	-	-	-
Sulphate	5 ug/g dry	100	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5721-1 – TEST HOLE LOCATION PLAN

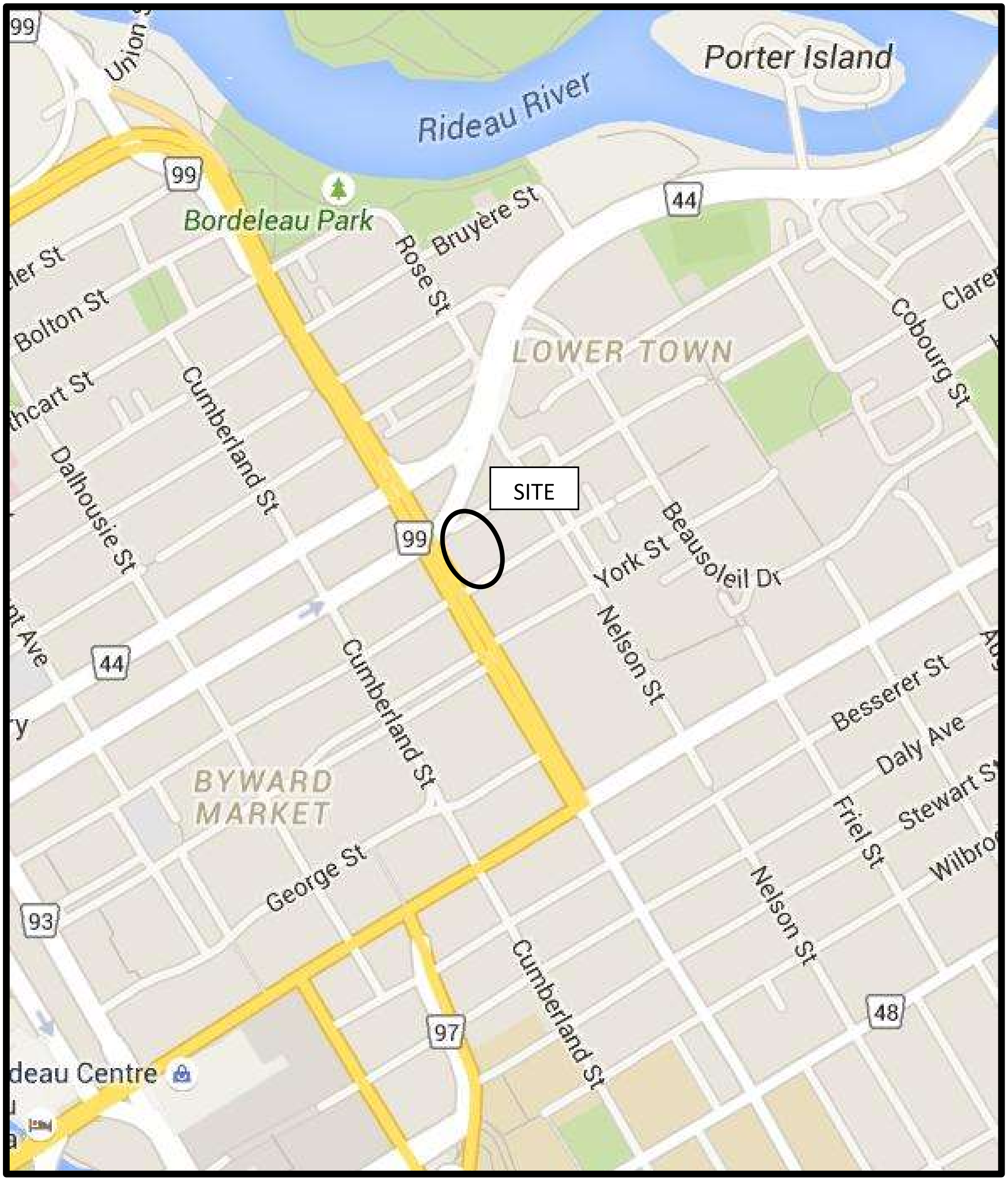
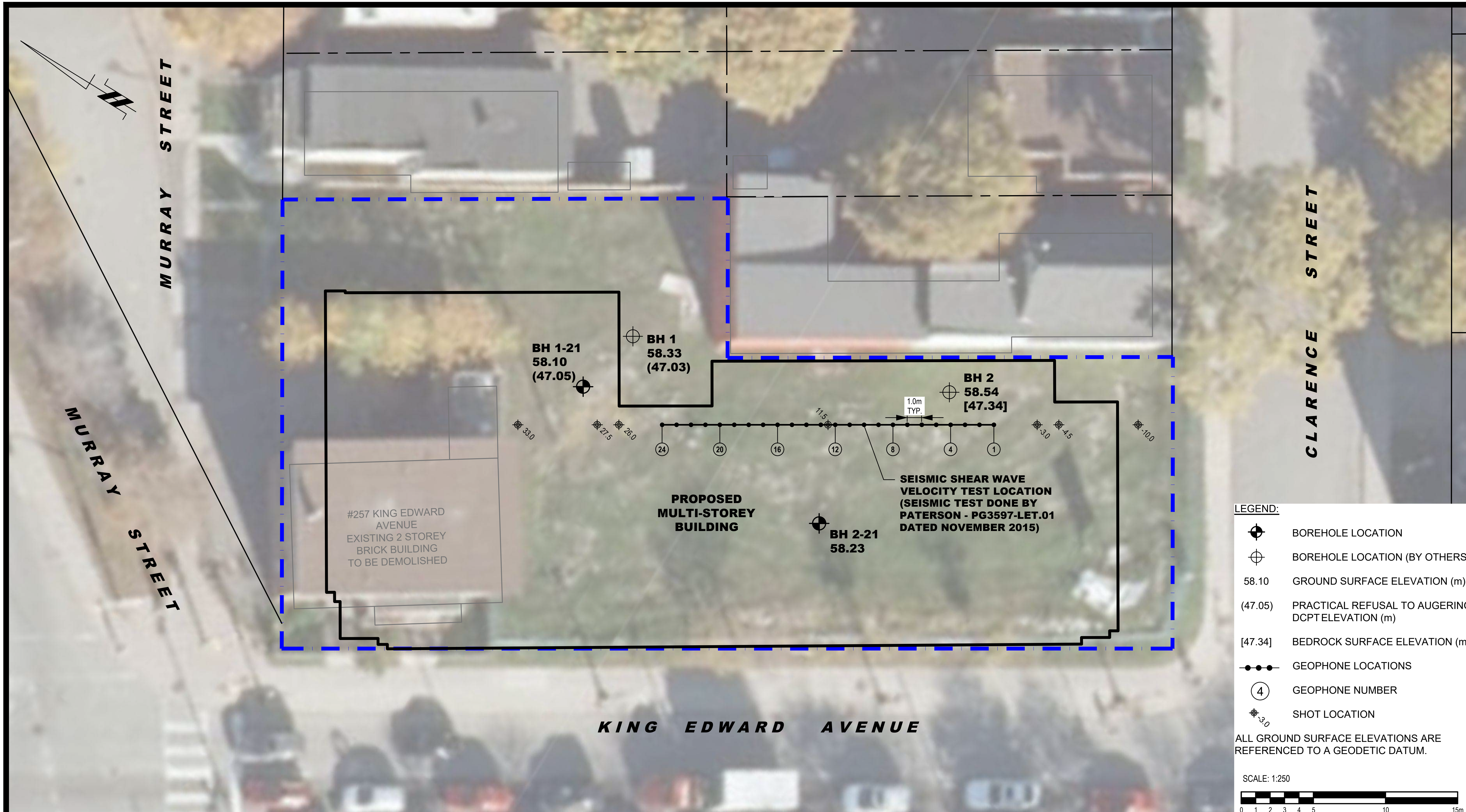


FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL

165177 ONTARIO INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
275 KING EDWARD AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 04/2021
Drawn by: NFRV	Report No.: PG5721-1
Checked by: MS	Dwg. No.: PG5721-1
Approved by: DJG	Revision No.:

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