PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

283 & 285 McLeod Street

Ottawa, Ontario

Prepared for REZY Properties Inc.

Report PG5489-1 Revision 2 dated December 11, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by REZY Properties Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 283-285 McLeod Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope 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 development at the subject site will consist of a 5-storey structure with 1 basement level and an approximate footprint of 500 m^2. At finished grades, the proposed building will be immediately surrounded by walkways and landscaped areas.

It is expected that the proposed building will be municipally serviced. Construction of the proposed building will require the demolition of the existing buildings located on-site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on September 4, 2020 and consisted of 3 boreholes advanced to a maximum depth of 11.3 m. The borehole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The approximate locations of the boreholes are shown on Drawing PG5489-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. 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 presented in Appendix 1.

A 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, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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

The subsurface conditions observed in the boreholes 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

Standpipe piezometers were installed in all boreholes to permit monitoring of the groundwater levels following the completion of the field investigation. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes, and the ground surface elevation at each borehole location, are presented on Drawing PG5489-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

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 (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. The sample was tested to determine the concentration of sulphate and chloride, and the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of 2 contiguous properties, 285 and 283 McLeod Street, which border each other to the east and west, respectively. The site is bordered by a commercial property to the north, residential properties to the east and west, and McLeod Street to the south.

The southern half of the subject site is currently occupied by a 2 storey residential structure at 283 McLeod Street and a 2 storey commercial structure at 285 McLeod Street. The northern half of the property is currently occupied by an asphalt paved parking lot. The ground surface across the site is relatively level at approximate geodetic elevation 71 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 50 to 100 mm thickness of asphalt underlain by fill which extends to approximate depths of 2.3 to 3.1 m below the existing ground surface. The fill was generally observed to consist of a either a brown silty sand with gravel and brick or a brown silty clay.

A stiff, grey silty clay deposit was observed underlying the fill material in boreholes BH 1 and BH 3. The silty clay in borehole BH 2 was observed to transition from a very stiff brown silty clay crust to a stiff grey silty clay at a depth of 3.8 m below the existing ground surface.

Refusal of the DCPT was encountered at an approximate depth of 28.2 m below the existing ground surface.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of shale of the Billings formation with a drift thickness of 25 to 50 m.

4.3 Groundwater

Groundwater levels were measured in the standpipes on September 11, 2020. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	71.04	7.70	63.34	Sept 11, 2020
BH 2	71.48	10.40	61.08	Sept 11, 2020
BH 3	71.46	Blocked and Dry	-	Sept 11, 2020

Note – The ground surface elevations at the borehole locations are referenced to a geodetic datum.

The long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered samples. Based on these observations, the long-term groundwater levels are expected to range between approximately 8 to 10 m below ground surface.

However, 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 considered suitable for the proposed development. It is recommended that the proposed multi-storey building be founded on one of the following:

❒ A raft foundation bearing on an undisturbed, stiff silty clay bearing surface, or

❒ End-bearing piles extending to the bedrock.

Conventional spread footing may also be utilized to provide foundation support for isolated exterior columns and auxiliary structures.

Due to the presence of a deep silty clay deposit, a permissible grade raise restriction is required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Protection of Subgrade (Raft Foundation)

Since the subgrade material will consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying.

Compacted Granular Fill Working Platform (Pile Foundation)

Should the proposed building be supported on a driven pile foundation, the use of heavy equipment would be required to install the piles (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck.

A typical working platform could consist of 0.6 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the basement slab.

Vibration Considerations

Construction operations could be the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: pile driving rig, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by pile driving (if required) or other construction operations, could be the cause or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are current construction standards.

These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Should the proposed multi-storey building be supported on a pile foundation, a pre-construction survey of the existing structures located in proximity of the pile driving operations should be conducted prior to commencing construction. The extent of the survey should be sufficient to respond to any inquiries/claims related to the blasting operations.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values for Exterior Columns and Auxiliary Structures

Where required, strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface 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.

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a glacial till 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 or engineered fill of the same or higher capacity as the soil.

Raft Foundation

As noted above, it is expected that a raft foundation will be required to support the proposed multi-storey building. For our design calculations, one level of underground parking was assumed which would extend approximately 3 to 3.5 m below existing ground surface. The maximum SLS contact pressure is **150 kPa** for a raft foundation bearing on the undisturbed, stiff silty clay. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. 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 **225 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 **6 MPa/m** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, glacial till is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

Paterson should review the final raft bearing pressure distribution diagram for conformance with the recommendations provided above.

Pile Foundation

Should the maximum SLS contact pressure provided above for a raft foundation be insufficient for support of the proposed multi-storey building, a deep foundation system driven to refusal in the bedrock would be required to provide adequate foundation support. 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 below. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

Table 2 – Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance Factored at ULS (kN)	Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
245	9	1090	10	28.5
245	10	1260	10	34.2
245	13	1500	10	40.7

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

The minimum recommended 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.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.5 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

For foundations constructed at the subject site, the site class for seismic site response can be taken as **Class D**, according to the Ontario Building Code (OBC) 2012. The soils underlying the subject site are not susceptible to liquefaction.

Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

If a raft slab is considered, a granular layer of OPSS Granular A is recommended 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 building founded on piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone, which is placed over an undisturbed, stiff silty clay subgrade.

An underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab. This is discussed further in Subsection 6.1.

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/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, 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

Should a flexible pavement be required for the project, the recommended flexible pavement structures shown in Tables 3 and 4 would be applicable.

Table 3 – Pavement Structure – Car Only Parking Areas and Driveways

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

Table 4 – Pavement Structure – Local Residential Roadways with Bus Traffic

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 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system extending to a series of drainage sleeve inlets through the building foundation wall. The drainage sleeves should be at lease 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the sub-slab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower garage area.

Foundation Raft Slab Construction Joints

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

Underslab Drainage

Underslab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximately 6 m centers. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available, 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 composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

However, the foundations are expected to have sufficient frost protection due to the founding depth. Unheated structures such as the access ramp may require insulation against the deleterious effect of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given that the proposed building is anticipated to extend near the property lines, it is expected that a temporary shoring will be required to support the excavation.

Unsupported Excavations

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

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils due to the proximity of the basement level to the property lines. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events. The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 – Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
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.

Underpinning of Adjacent Structures

As the excavation for the proposed basement level will be located near the existing building located in the southwest corner of the site, and the adjacent building to the northwest of the subject site, underpinning would be required if the proposed building foundations extend within the lateral support zone of the existing building foundations.

Conventional timber lagged pits and concrete underpinning piers are considered to be suitable for this project. The depth of the underpinning, should it be required, will be dependent on the depth of the adjacent foundations relative to the foundation depths of the proposed addition at the subject site.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. However, the bedding thickness should be increased to 300 mm for areas over a grey silty clay subgrade. 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 50 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density. It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, at 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 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR).

A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

Impacts on Neighbouring Properties

Based on the existing groundwater level encountered during the geotechnical investigation, the proposed building construction will not extend below the groundwater level. Therefore, groundwater lowering is not anticipated during or after construction, and accordingly, the proposed development will not negatively impact the neighbouring structures.

6.6 Winter Construction

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

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

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) 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 non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Setbacks

Tree planting shall follow the City of Ottawa's "Tree Planting in Sensitive Marine Clay Soils - 2017 Guidelines". As Atterberg limits testing was not completed as part of the geotechnical investigation program, it is recommended that tree planting setbacks be a minimum of **7.5 m** for trees with a mature height smaller than or equal to 7.5 m, and for trees with a mature height greater than 7.5 m, the tree planting setback should be equal to the mature height of tree. It should be noted that shrubs and other small planting with mature root depths less than 2 m are permitted within the tree planting setbacks.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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.

- ❑ Review the final grading plan, from a geotechnical perspective.
- ❑ Review the contractor's design of the temporary shoring system.
- ❑ Observe test pits to determine requirements for underpinning of adjacent structures.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable
- ❑ 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.

All excess excavated soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and to review our recommendations when the drawings and specifications are complete.

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 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 its 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 REZY Properties Inc. 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.

LICENSED PROFESSIONAL ENGINEER
Dec. 11, 2023
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Otillia McLaughlin B.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ REZY Properties Inc. (e-mail copy)
- ❑ 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 - 283-285 McLeod Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

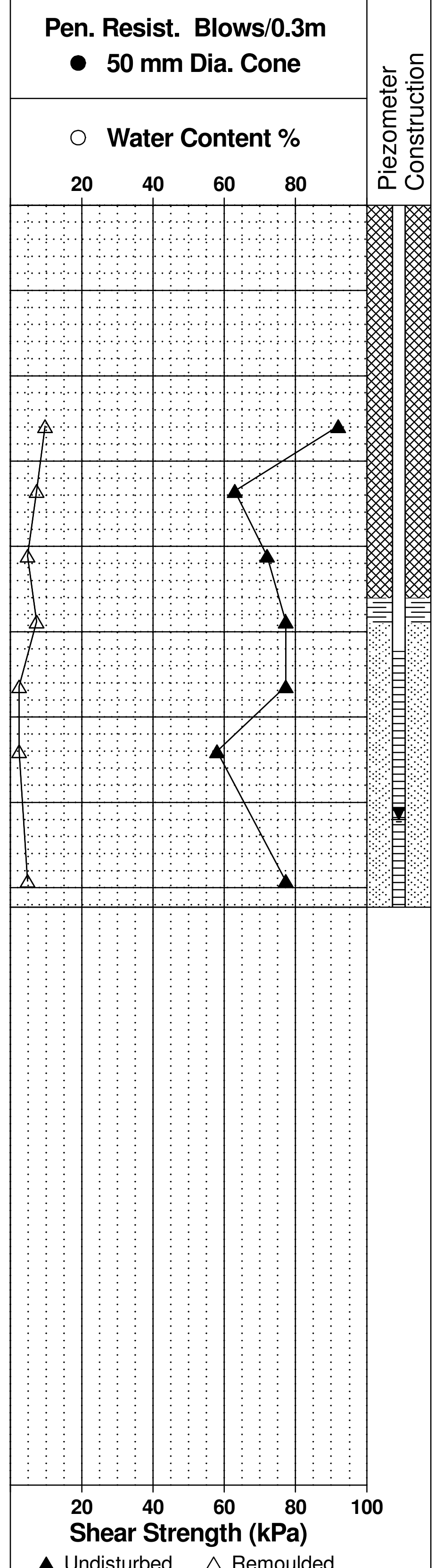
BORINGS BY CME-55 Low Clearance Drill

DATE September 4, 2020

FILE NO. PG5489

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.04
Asphaltic concrete 0.05		AU	1				
FILL: Brown silty sand with crushed stone, trace clay 0.63							
FILL: Brown silty sand 1.52		SS	2	4	8	1	70.04
FILL: Brown silty clay 3.05		SS	3	33	5	2	69.04
						3	68.04
Stiff, grey SILTY CLAY 8.23						4	67.04
						5	66.04
						6	65.04
						7	64.04
						8	63.04
End of Borehole							
(GWL @ 7.20m - Sept. 11, 2020)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 283-285 McLeod Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

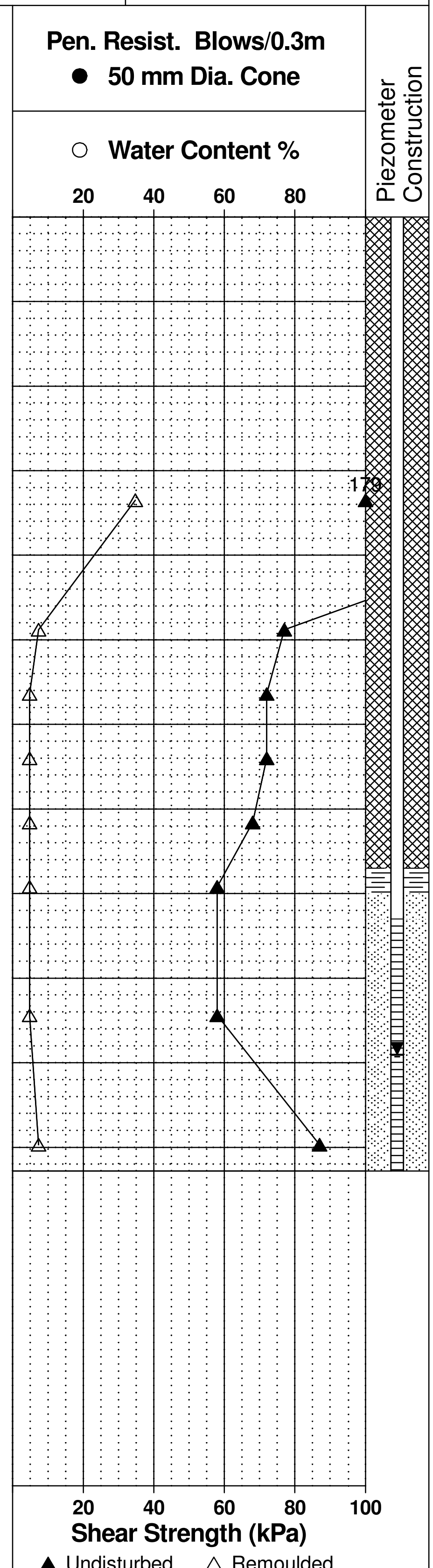
BORINGS BY CME-55 Low Clearance Drill

DATE September 4, 2020

FILE NO. PG5489

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.48
Asphaltic concrete 0.10		AU	1				
FILL: Brown silty sand with crushed stone 0.59							
FILL: Brown silty sand, trace gravel and brick		SS	2	46	5	1	70.48
2.29		SS	3	33	4	2	69.48
Very stiff, brown **SILTY CLAY**		SS	4	75	9	3	68.48
- stiff and grey by 3.8m depth		SS	5	100	2	4	67.48
						5	66.48
						6	65.48
						7	64.48
						8	63.48
						9	62.48
						10	61.48
11.28						11	60.48
End of Borehole							
(GWL @ 9.90m - Sept. 11, 2020)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 283-285 McLeod Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

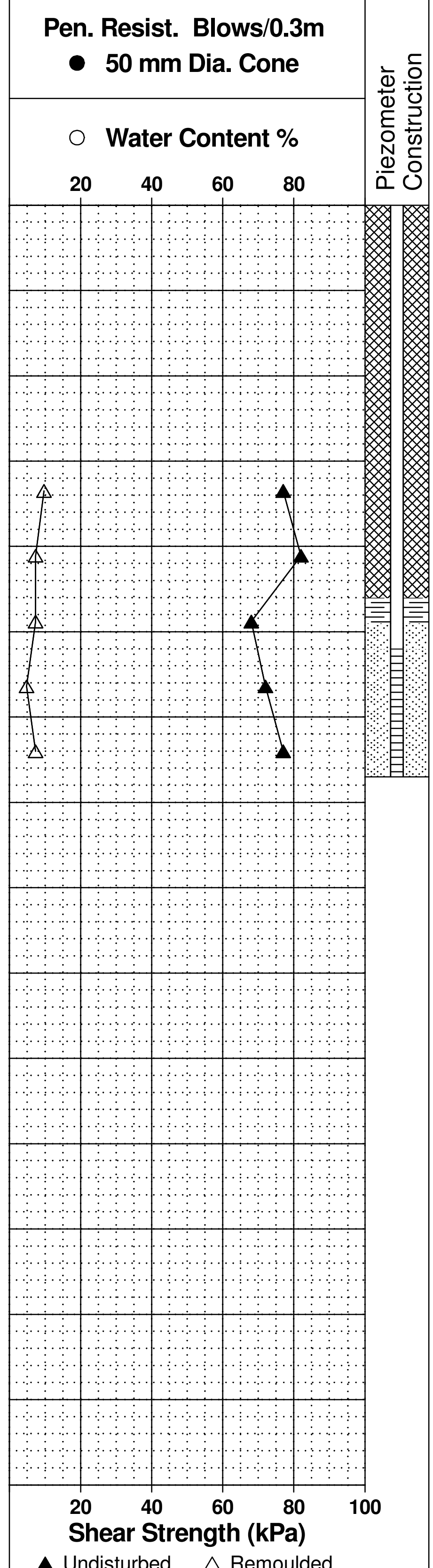
DATE September 4, 2020

FILE NO. PG5489

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with crushed stone	0.56	SS	2	0	50+
FILL: Brown silty sand, trace gravel and brick	1.52				
FILL: Brown silty sand with gravel	2.29	SS	3	33	12
Stiff, grey **SILTY CLAY**	6.70	SS	4	83	9
Dynamic Cone Penetration Test commenced @ 6.70m depth. Cone pushed to 22.25m depth.					

DEPTH (m)	ELEV. (m)
0	71.46
1	70.46
2	69.46
3	68.46
4	67.46
5	66.46
6	65.46
7	64.46
8	63.46
9	62.46
10	61.46
11	60.46
12	59.46
13	58.46
14	57.46
15	56.46



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 283-285 McLeod Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 4, 2020

FILE NO. PG5489

HOLE NO. BH 3

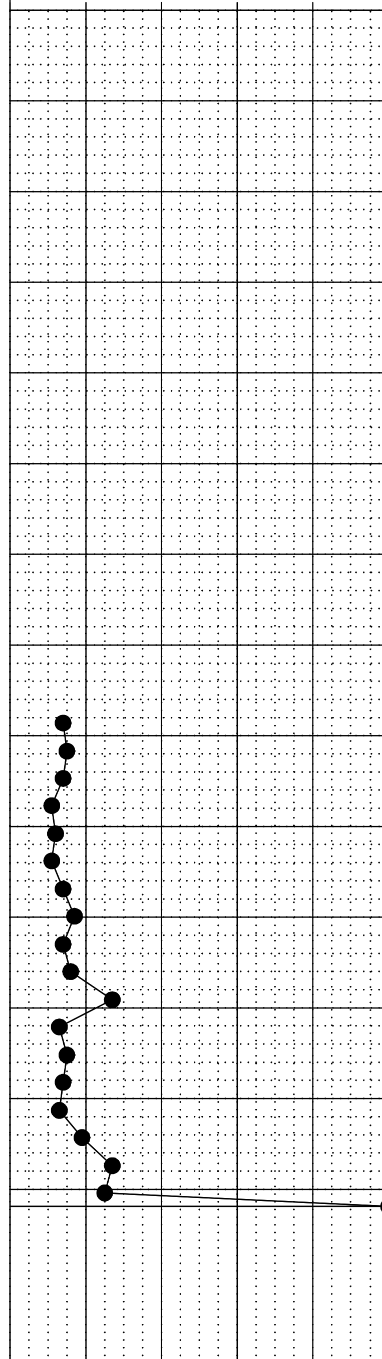
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						15	56.46
						16	55.46
						17	54.46
						18	53.46
						19	52.46
						20	51.46
						21	50.46
						22	49.46
						23	48.46
						24	47.46
						25	46.46
						26	45.46
						27	44.46
						28	43.46
28.19							
End of Borehole Practical DCPT refusal @ 28.19m depth. (Piezometer blocked and dry at 2.4m depth - Sept. 11, 2020)							

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

○ Water Content %

20 40 60 80

Piezometer Construction



20 40 60 80 100
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

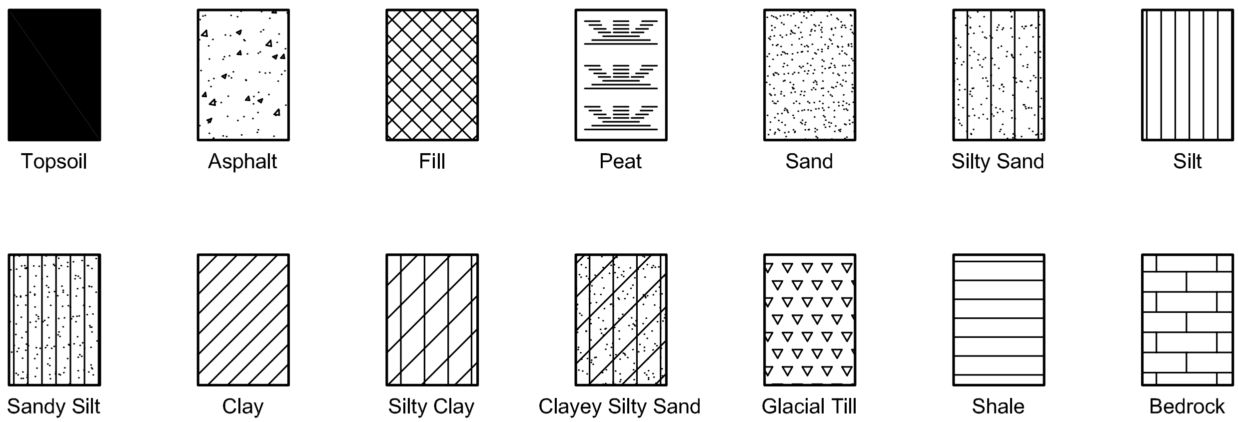
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

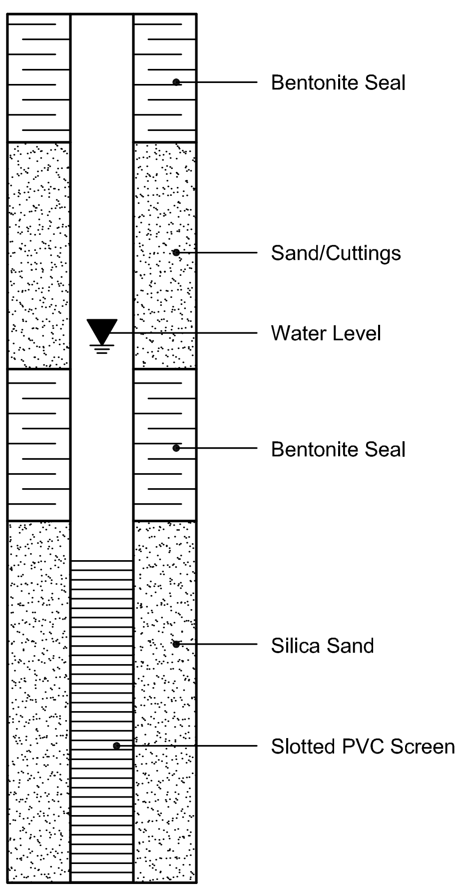
SYMBOLS AND TERMS (continued)

STRATA PLOT

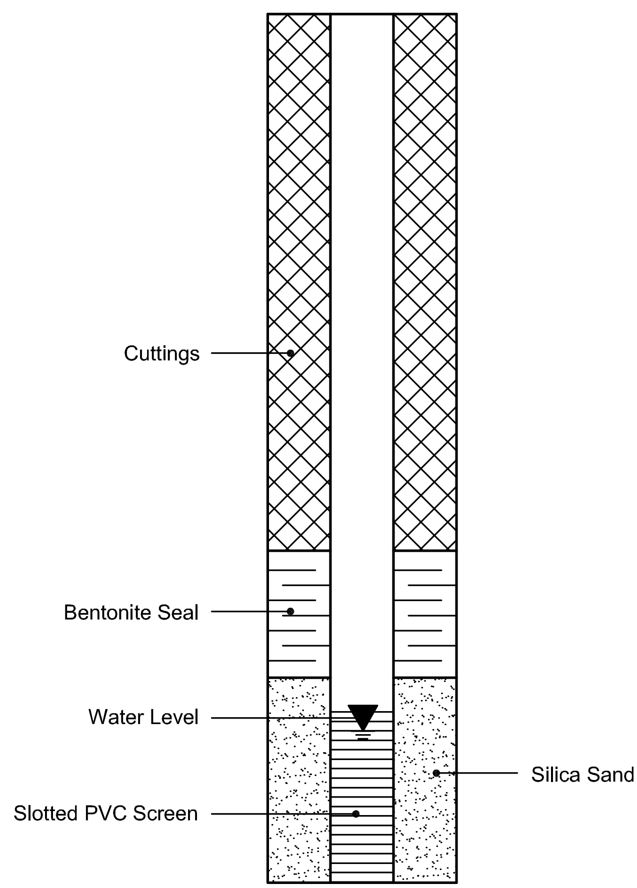


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2037133

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30820

Report Date: 14-Sep-2020

Order Date: 8-Sep-2020

Project Description: PG5489

	Client ID:	BH2-SS4	-	-	-
	Sample Date:	04-Sep-20 08:50	-	-	-
	Sample ID:	2037133-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	69.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.43	-	-	-
Resistivity	0.10 Ohm.m	4.63	-	-	-
Anions					
Chloride	5 ug/g dry	1420	-	-	-
Sulphate	5 ug/g dry	79	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5489-1 - TEST HOLE LOCATION PLAN

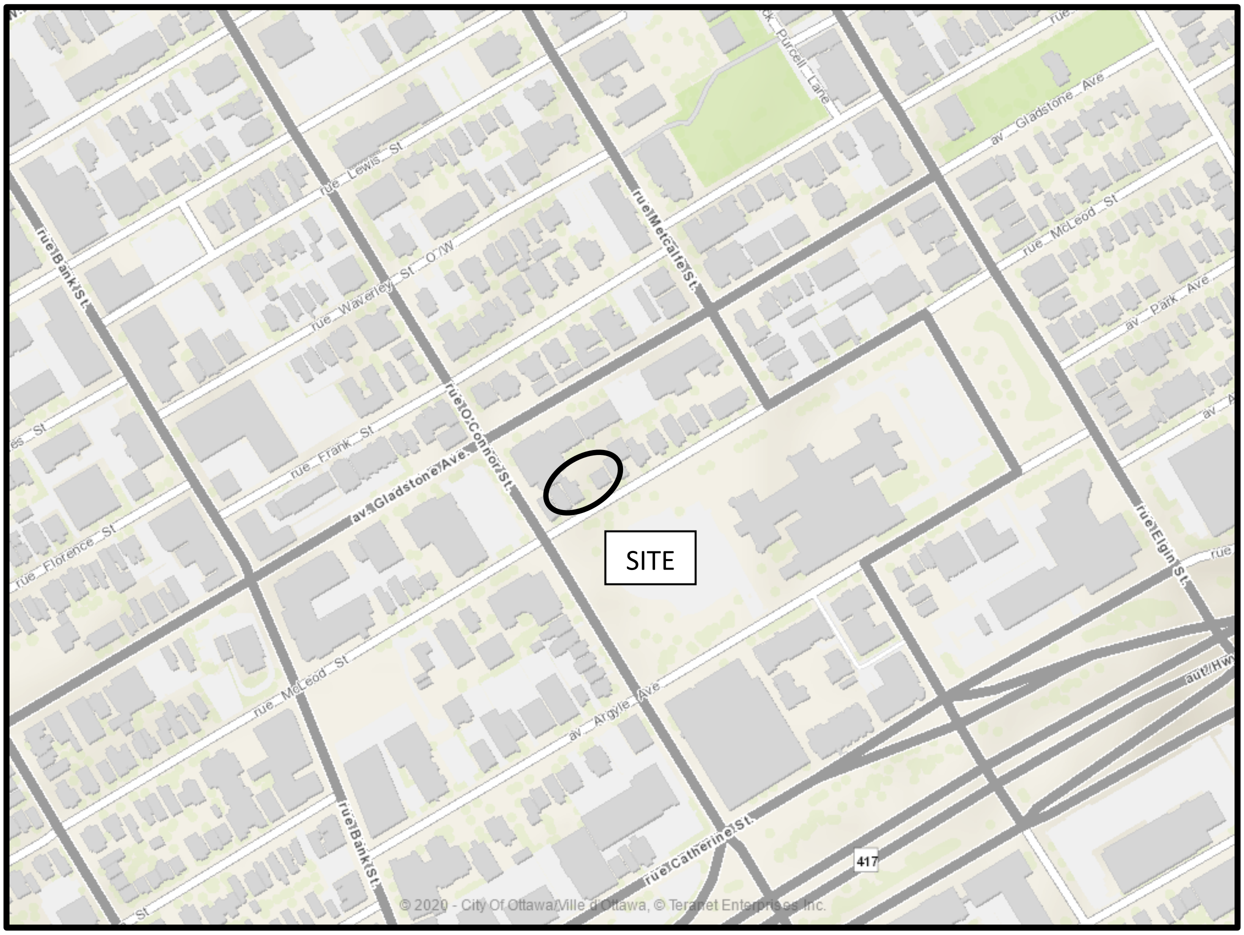
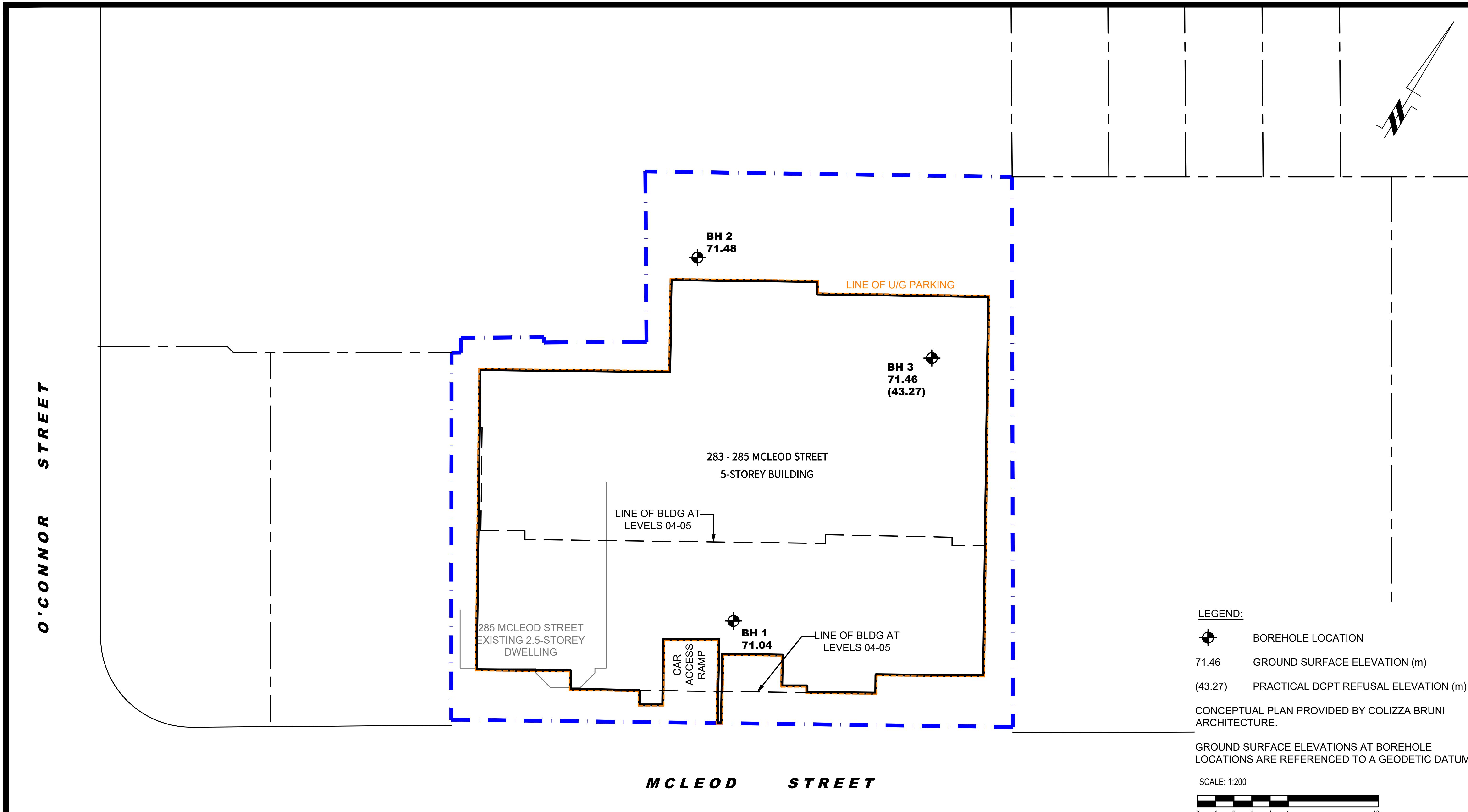


FIGURE 1

KEY PLAN



LEGEND:

BOREHOLE LOCATION

71.46 GROUND SURFACE ELEVATION (m)

(43.27) PRACTICAL DCPT REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY COLIZZA BRUNI ARCHITECTURE.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO NEW CONCEPTUAL PLAN	08/12/2023	OM
1	UPDATED TO NEW CONCEPTUAL PLAN	13/07/2021	SD

ZYER DEVELOPMENTS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 283 AND 285 MCLEOD STREET
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 09/2020
Drawn by: NFRV	Report No.: PG5489-1
Checked by: OM	Dwg. No.: PG5489-1
Approved by: SD	Revision No.: 2

p:\autocad drawings\geotechnical\pg54xx\pg5489\pg5489-1-test hole location plan (rev.02).dwg