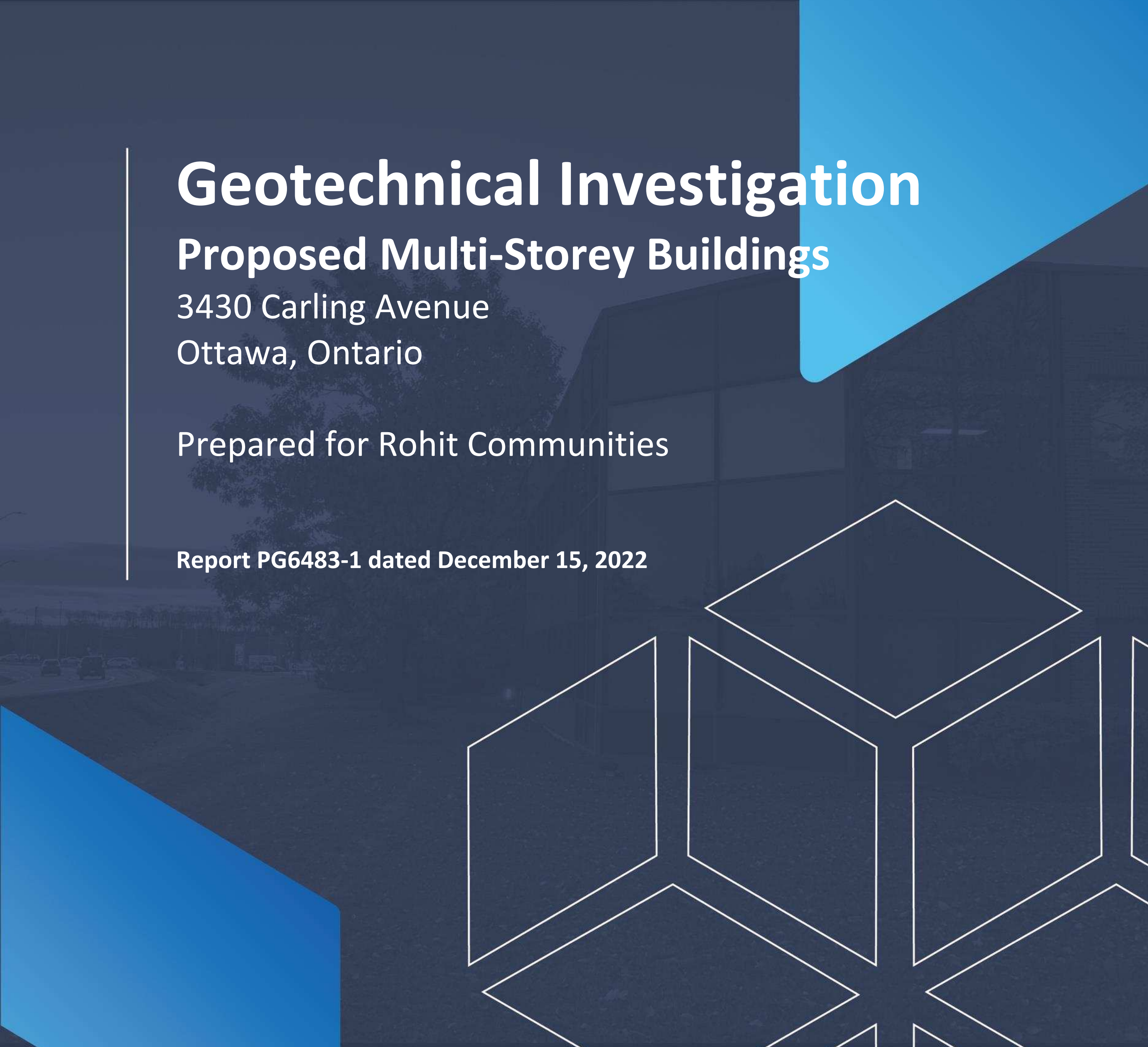
Geotechnical Investigation

Proposed Multi-Storey Buildings

3430 Carling Avenue
Ottawa, Ontario

Prepared for Rohit Communities

Report PG6483-1 dated December 15, 2022



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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Rohit Communities to prepare a geotechnical investigation report for the proposed multi-storey buildings to be located at 3430 Carling Avenue, Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

☐ determine the subsoil and groundwater conditions at the site by means of test holes

☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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 conceptual drawings, it is understood that the proposed development will consist of two mid-rise apartment buildings with one to two underground parking. Associated at-grade parking areas, walkways, and hardscape areas are also anticipated as part of the development. It is expected that the proposed multi-storey buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on March 6 and 7, 2019. At that time, 4 boreholes were advanced to a maximum depth of 10 m below existing ground surface. 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 holes locations are shown on Drawing PG6483-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a track mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test holes procedures consisted of excavating to the required depth at the selected location and sampling 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 subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located on Carling Avenue. The TBM was assigned an elevation of 100.00 m. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6483-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (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 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 subject site is currently occupied by a single-storey commercial building located on the eastern end of the site. The remainder of the site is occupied by asphalt paved access lanes and parking areas with landscaped margins.

The site is bordered by Carling Avenue to the north, a commercial property to the east, and residential properties to the south and west. The existing ground surface across the site is relatively level and at grade with the surrounding properties and roads.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphalt pavement and fill overlying a silty clay deposit followed by glacial till. The fill was generally observed to consist of crushed stone and brown silty sand with gravel. The silty clay deposit was observed to consist of a hard to stiff, brown silty clay, becoming a stiff to firm, grey silty clay at approximate depths of 3 to 4 m below the existing ground surface. Interbedded silty sand seams were also observed within the silty clay deposit. A glacial till deposit was encountered underlying the silty clay at approximate depths of 5.3 to 7 m below the existing ground surface. The glacial till generally consisted of a loose to compact, grey sandy silt to silty clay with gravel, cobbles, and boulders.

Practical refusal of the augers or DCPT were encountered at depths of 9.4 to 10 m below the existing ground surface.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of dolomite of the Oxford formation with a drift thickness of 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and standpipes on September 27, 2019. The observed groundwater levels are summarized in Table 1.

Table 1 – Summary of Groundwater Levels Readings

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1*	99.31	4.77	94.54	September 27, 2019
BH 2	99.23	Blocked	-	September 27, 2019
BH 3*	99.17	Inaccessible	-	September 27, 2019
BH 4*	99.43	5.29	94.14	September 27, 2019

Note: - * Denotes borehole instrumented with a 51 mm diameter monitoring well.
- The ground surface elevations at the borehole locations are referenced to a TBM, consisting of the top spindle of the fire hydrant located on Carling Avenue which was assigned an elevation of 100.00 m.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. 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 to 4 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 level 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 multi-storey buildings.

It is understood that the proposed multi-storey buildings will have either one or two shared underground parking levels. Given the subsurface conditions encountered during the geotechnical investigation and the anticipated building loads, 3 options were considered for foundation support of the proposed building, considering one underground parking level only:

- ☐ conventional spread footings bearing on the undisturbed, stiff silty clay deposit.
- ☐ a raft foundation bearing on the undisturbed, stiff silty clay deposit.
- ☐ a deep foundation, such as end-bearing piles, which extend to the bedrock surface.

If two underground parking levels are considered, then the structure can be founded on a raft foundation bearing on the loose to compact glacial till deposit.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding depth, it is anticipated that all topsoil and deleterious fill, such as those containing organic materials, will be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

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. 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 placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Protection of Subgrade (Raft Foundation)

It is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed subgrade shortly after the completion of the excavation. The main purpose of the mudslab 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.

5.3 Foundation Design

Conventional Spread Footings (One Underground Parking Level)

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**.

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 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 firm to stiff silty clay bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

Raft Foundation

Based on the expected loads from the proposed multi-storey buildings, a raft foundation is an alternative solution to found the proposed building. For one level of underground parking, it is expected that the excavation will extend between 4 to 5 m below existing ground surface.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. 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 one level of underground parking.

For the option of one level of underground parking, a bearing resistance value at SLS (contact pressure) of **150 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **225 kPa**. For this case, the modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **150 kPa**.

For the option of two levels of underground parking, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed glacial till. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. For this case, the modulus of subgrade reaction was calculated to be **8 MPa/m** for a contact pressure of **200 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

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

Pile Foundation

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

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

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance
		Factored at ULS (kN)
245	9	1460
245	11	1650
245	13	1760

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

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations constructed at the subject site. A site specific shear wave velocity test may be completed to determine if a seismic Site Class C is applicable for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code 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 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 building founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone backfill under the lower basement floor. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. 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.

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/m^3, 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 0.281 g according to OBC 2012 (Amendment 2019). 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 and heavy traffic access areas are expected at this site. The proposed pavement structures are presented in Tables 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 3 - Recommended Pavement Structure – Car Only Parking Areas

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

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

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
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, bedrock or concrete 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 I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Depending on the final number of underground levels, a portion of the proposed building foundation walls may 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 also recommended that 150 mm diameter sleeves at 3-6 m centers 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 is 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 Backfilling

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.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or 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. Given the proximity of the underground parking level to 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 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 may be 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 rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the 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 and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

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

It is anticipated that groundwater infiltration into the excavation 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 the shallow excavation. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' 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/building) 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 Neighboring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. 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.

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 sever to very aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Based on our review of the subsurface profile below the subject site, the silty clay crust within the upper 3 to 3.5 m is relatively dry and designated as a hard to stiff silty clay. Therefore, the proposed development is located within an area of low sensitive silty clay deposits for tree planting.

Based on the above discussion, and due to the founding depth of the proposed building with 1 level of underground parking, it is recommended that trees placed within 4.5 m of the foundation wall consist of street trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 4.5 m from the foundation wall may consist of moderate water demanding trees with roots extending to a maximum 2 m depth. It should be noted that shrubs and other small plantings are permitted within the 5 m setback area.

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 of the grading plan from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavating program, prior to construction.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Complete a full inspection program of the installation of the groundwater infiltration control system during construction.

☐ 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 Rohit Communities or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Maha K. Saleh, P.Eng.

LICENSED PROFESSIONAL ENGINEER
December 15, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Rohit Communities (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 3430 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site, along Carling Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 6

FILE NO. PG4836

HOLE NO. BH 1

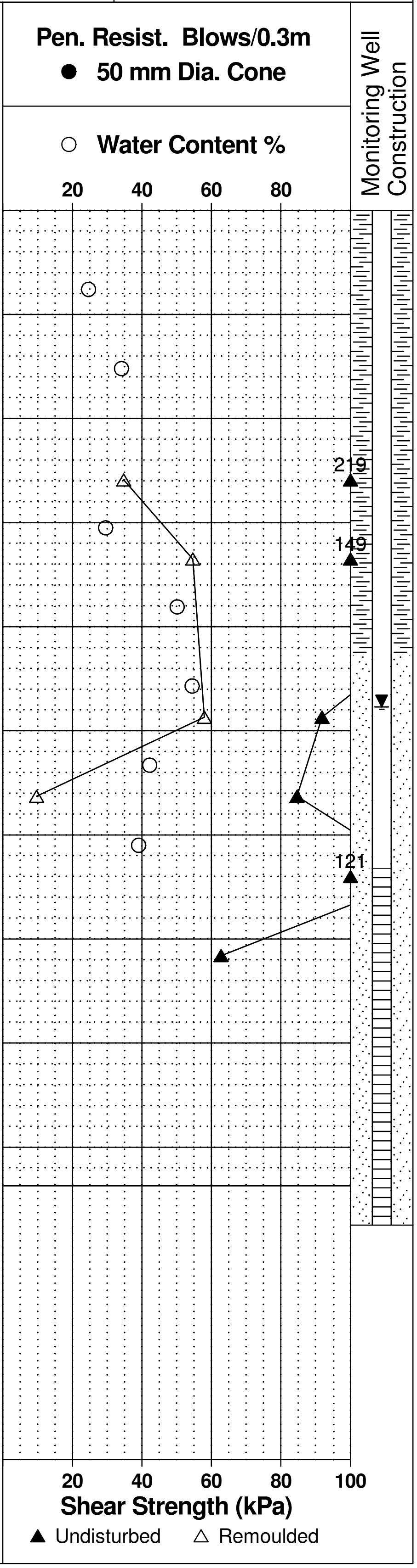
SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08	AU	1		
FILL: Brown silty sand with gravel	0.60				
Hard to very stiff, brown **SILTY CLAY**	3.00	SS SS	2 3	100 100	25 10
Very stiff, brown **CLAYEY SILT,** trace sand	3.91	SS	4	8	P
Stiff, grey **SILTY CLAY** with interbedded silty sand seams	6.96	SS SS SS SS	5 6 7 8	100 100 100 8	2 P P P
GLACIAL TILL: Grey silty clay with gravel, cobbles and boulders	7.62	SS	9	88	P
GLACIAL TILL: Grey sandy silt, some clay, trace gravel, cobbles and boulders	9.37	SS SS SS	10 11 12	100 0 78	4 3 50+

End of Borehole

Practical refusal to augering at 9.37m depth

(GWL @ 4.77m - Sept. 27, 2019)

DEPTH (m)	ELEV. (m)
0	99.31
1	98.31
2	97.31
3	96.31
4	95.31
5	94.31
6	93.31
7	92.31
8	91.31
9	90.31



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 3430 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site, along Carling Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 6

FILE NO. PG4836

HOLE NO. BH 2

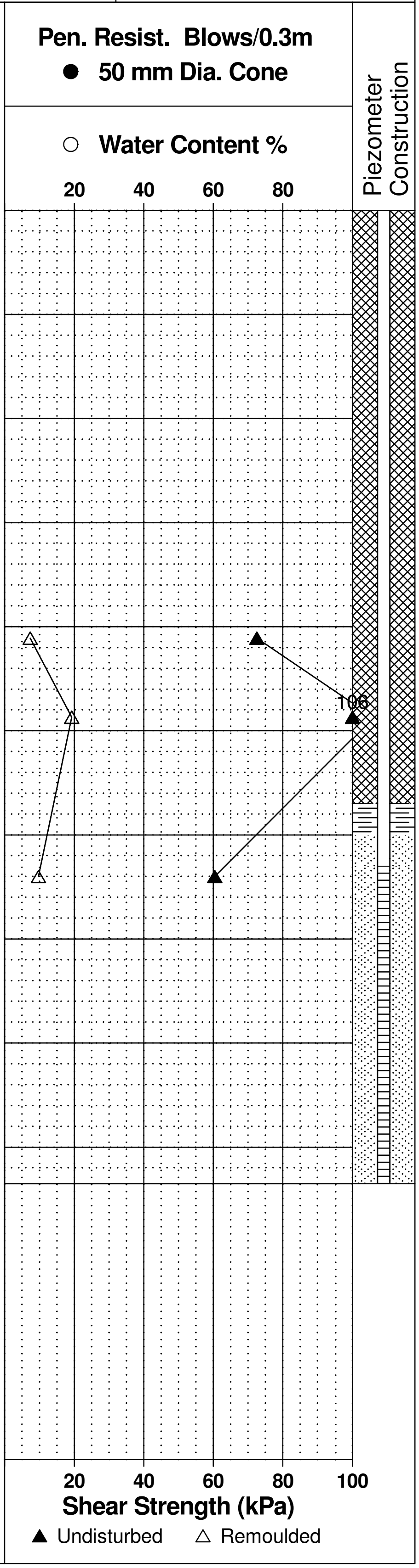
SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand with gravel	0.61	AU	1		
Hard, brown **SILTY CLAY**		SS	2	75	20
	2.44	SS	3	27	10
Stiff, brown **CLAYEY SILT,** trace sand	3.00	SS	4	38	6
Stiff, grey **SILTY CLAY** with interbedded silty sand seams		SS	5	100	1
		SS	6	100	P
		SS	7	100	P
	6.70	SS	8	100	P
GLACIAL TILL: Grey silt, some clay, trace sand, gravel, cobbles and boulders	7.47	SS	9	100	2
GLACIAL TILL: Grey clayey silt with sand, gravel, cobbles and boulders		SS	10	46	17
		SS	11	29	5
	9.35	SS	12	88	50+

End of Borehole

Practical refusal to augering at 9.35m depth.

(Piezometer blocked - Sept. 27, 2019)

DEPTH (m)	ELEV. (m)
0	99.23
1	98.23
2	97.23
3	96.23
4	95.23
5	94.23
6	93.23
7	92.23
8	91.23
9	90.23



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 3430 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site, along Carling Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** 2019 March 7

FILE NO. PG4836

HOLE NO. BH 3

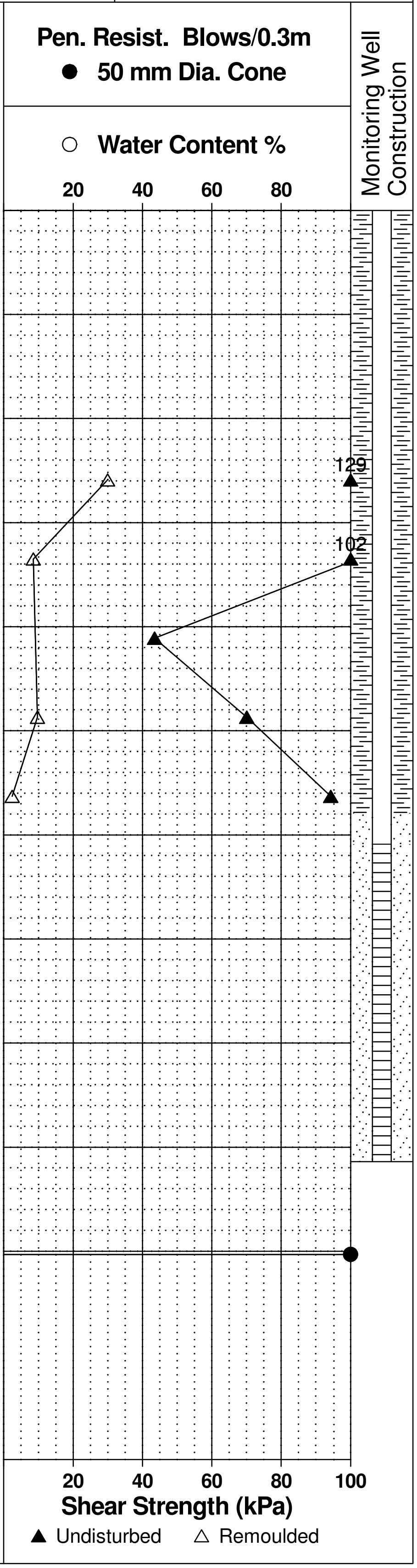
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone with silty sand	0.29
Very stiff, brown **SILTY CLAY**	2.21
Very stiff, brown **CLAYEY SILT,** trace sand	2.97
Very stiff to firm, grey **SILTY CLAY,** trace sand	5.33
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles, boulders	6.02
GLACIAL TILL: Grey silt, some clay, trace sand	6.78
GLACIAL TILL: Grey clayey silt, trace sand and gravel	10.03

Dynamic Cone Penetration Test commenced at 9.75m depth. Practical DCPT refusal at 10.03m depth.

(Monitoring well inaccessible - Sept. 27, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	20
SS	3	100	6
SS	4	100	P
SS	5	100	P
SS	6	67	P
SS	7	100	P
SS	8	0	P
SS	9	75	2
SS	10	79	2
SS	11	33	4
SS	12	100	4
SS	13	92	3

DEPTH (m)	ELEV. (m)
0	99.17
1	98.17
2	97.17
3	96.17
4	95.17
5	94.17
6	93.17
7	92.17
8	91.17
9	90.17
10	89.17



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 3430 Carling Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site, along Carling Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 March 6

FILE NO. PG4836

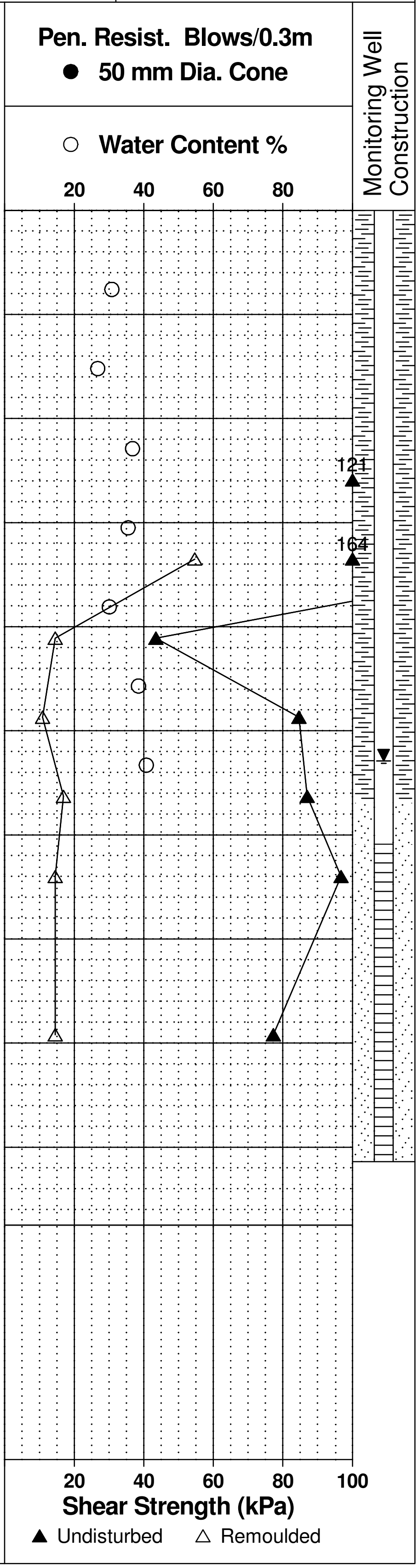
HOLE NO. BH 4

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with gravel	0.60
Very stiff to stiff, brown **SILTY CLAY**	3.66
Brown **SILT,** some clay	4.27
Firm to stiff, grey **SILTY CLAY** with interbedded silty sand seams	6.55
GLACIAL TILL: Grey clayey silt with sand, gravel, cobbles and boulders	9.75
End of Borehole	
(GWL @ 5.29m - Sept. 27, 2019)	

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	24
SS	3	46	14
SS	4	38	P
SS	5	38	P
SS	6	75	P
SS	7	100	P
SS	8	83	P
SS	9	100	P
SS	10	100	2
SS	11	100	P
SS	12	100	7

DEPTH (m)	ELEV. (m)
0	99.43
1	98.43
2	97.43
3	96.43
4	95.43
5	94.43
6	93.43
7	92.43
8	91.43
9	90.43



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

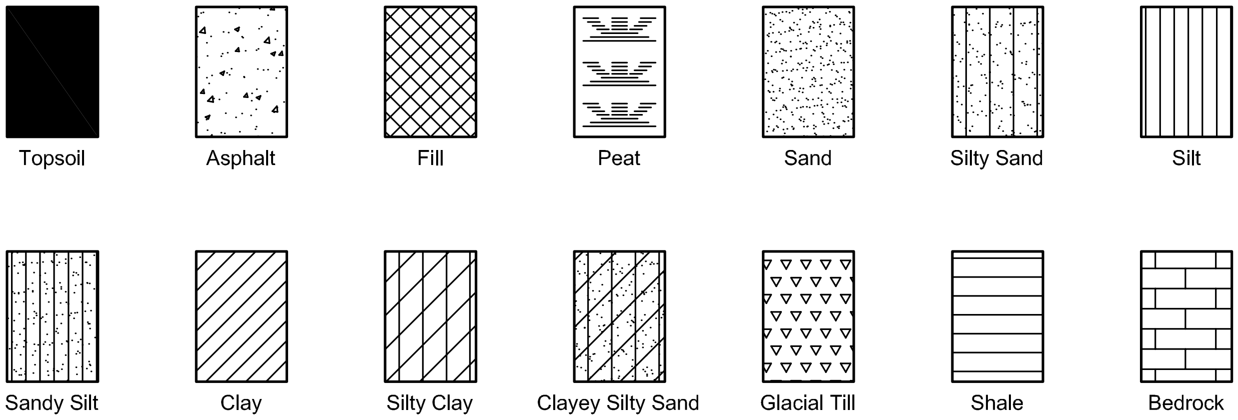
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

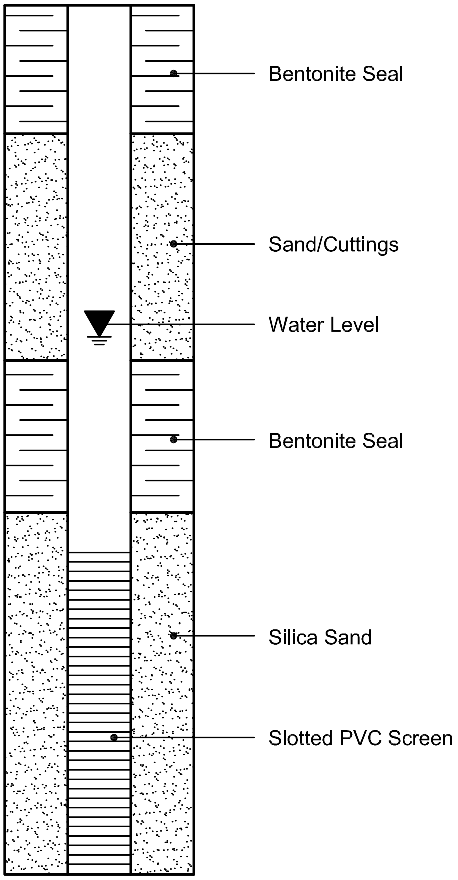
SYMBOLS AND TERMS (continued)

STRATA PLOT

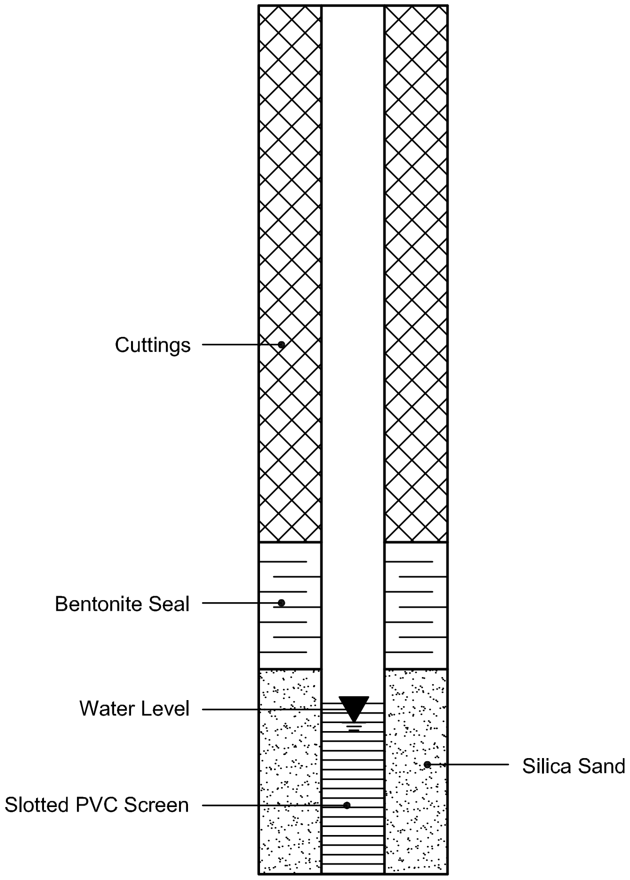


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



	Client ID:	BH2 SS6 12.5'-14.5'	-	-	-
	Sample Date:	03/06/2019 11:00	-	-	-
	Sample ID:	1910469-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.83	-	-	-
Resistivity	0.10 Ohm.m	15.4	-	-	-
Anions					
Chloride	5 ug/g dry	220	-	-	-
Sulphate	5 ug/g dry	144	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6483-1 – TEST HOLE LOCATION PLAN

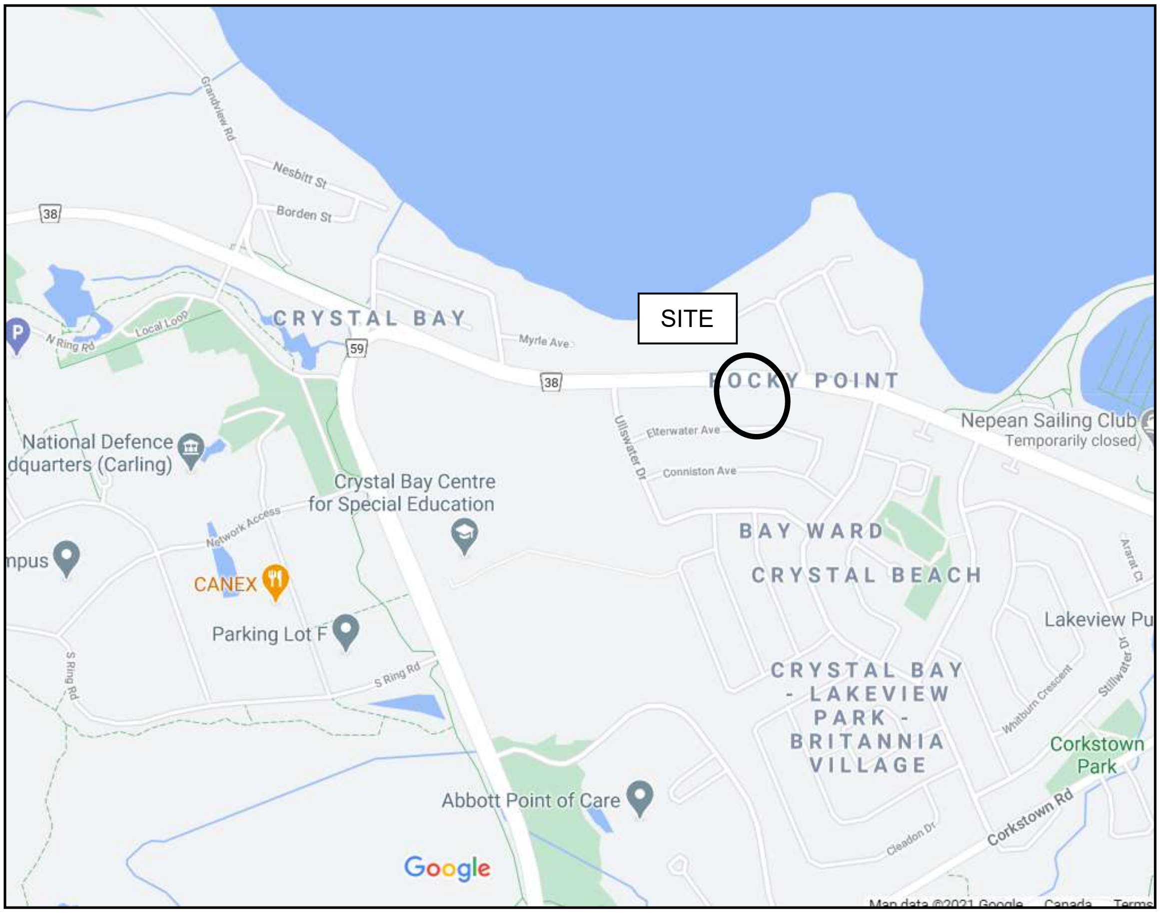
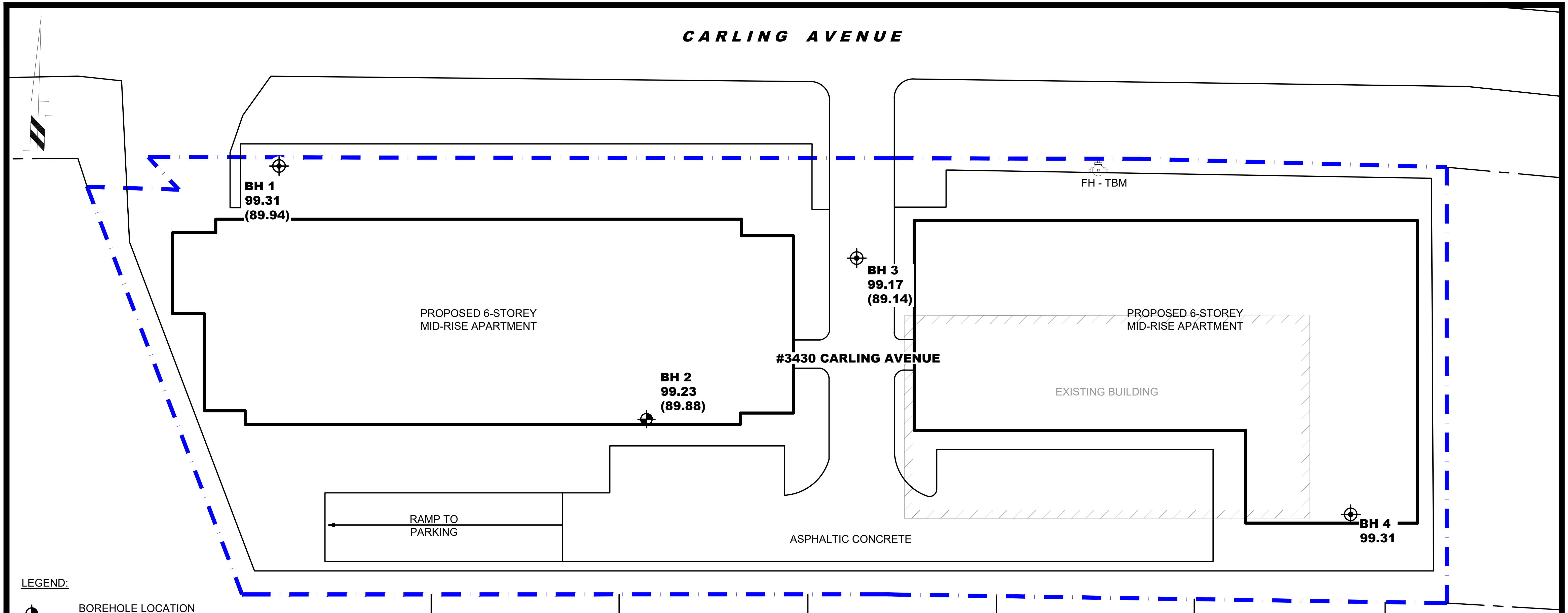


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION (PREVIOUS INVESTIGATION, REPORT No. PG4836)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PREVIOUS INVESTIGATION, REPORT No. PG4836)
99.17	GROUND SURFACE ELEVATION (m)
(89.14)	PRACTICAL REFUSAL TO DCPT/AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY PROJECT 1 STUDIO

TBM- TOP SPINDLE OF FIRE HYDRANT LOCATED ON CARLING AVENUE IN FRONT OF SUBJECT SITE. AN ASSUMED ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

SCALE: 1:400

0 5 10 15 20 25m

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

NO.	REVISIONS	DATE	INITIAL

ROHIT COMMUNITIES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
3430 CARLING AVENUE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Drawn by: YA
Checked by: ZM
Approved by: MS
Date: 12/2022
Report No.: PG6483-1
Dwg. No.: PG6483-1
Revision No.:

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