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Geotechnical Investigation

Proposed Multi Storey Building
729 Ridgewood Avenue
Ottawa, Ontario

Prepared for

Brigil

June 21, 2021

Report PG5172-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil to conduct a geotechnical investigation for the proposed mid-rise residential building to be located at 729 Ridgewood Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsurface and groundwater conditions by means of boreholes and existing soils information.

- ☐ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. The report contains Paterson's findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

The development is understood to consist of a multi storey residential building with up to 2 levels of underground parking. It is further understood that the proposed building will encompass the majority of the subject site. Associated at-grade access lanes, car parking and landscaped areas are also anticipated. The proposed building is anticipated to be municipally serviced.

The subject property is presently occupied by a two slab on grade commercial buildings. It is expected that the existing buildings within the site will be demolished as part of the proposed project. A former mechanics garage was located on the east portion of the site. The building was recently demolished.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed between June 25 and 26, 2020. At that time, 7 boreholes were advanced to a maximum depth of 9.7 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development taking into consideration existing site features and underground services. The borehole locations are shown on Drawing PG5172-1 - Test Hole Location Plan included in Appendix 2.

Sampling and In-Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sample or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to Paterson's laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are presented as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH6. 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.

Rock samples were recovered from BH7 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

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

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Flexible piezometers were installed in all the boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test hole locations were determined and located in the field by Paterson. All ground surface elevations reference a geodetic datum (NAD83). The locations of the boreholes and the ground surface elevations for each borehole location are presented on Drawing PG5172-1 -Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and the bedrock core were recovered from the subject site and visually examined in Paterson's laboratory to review the field logs.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

One 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 sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject property is presently occupied by two slab on grade commercial buildings. A former mechanical shop located on southeastern portion of the site was recently demolished. The area was backfilled with granular material. A parking lot and pavement structure covers the majority of the site. Some landscaped areas were noted along Ridgewood Avenue.

The ground surface across the subject site is relatively flat and slightly below grade from Ridgewood Avenue and the property to the west. The site is bordered to the west by a residential high rise structure, to the north and east by a residential and institutional development, and Ridgewood Avenue to the south.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the boreholes consist of asphaltic concrete overlying a fill layer consisting of crushed stone and silty sand. The fill layer is underlain by a stiff to hard layer of brown silty clay with sand seams. Glacial till was encountered below the above noted layers consisting of a compact to a very dense silty sand with clay, gravel, cobbles, and boulders. Seams of coarse sand where encountered in the glacial till layer at some test hole locations. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Bedrock was cored at one borehole location to confirm refusal. Limestone bedrock was encountered at a depth of 9.7 m below the existing ground surface at BH6. Refusal was encountered in the other boreholes between a depth of 4.8 to 8.7 m. It should be noted that boulders are to be expected.

Upon review of the core hole sample, the upper first meter of the bedrock was found to be of good quality.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone of the Bobcaygeon Formation. The overburden drift thickness is anticipated to be between 5 to 15 m in depth.

4.3 Groundwater

Flexible piezometers were installed as part of our geotechnical investigation. Groundwater level measurements were recorded at the borehole locations and our findings are presented in Table 1. It should also be noted that the groundwater level is subject to seasonal fluctuations. Therefore, groundwater could vary at the time of construction. It should be further noted that groundwater measurements at monitoring well locations can be influenced by surface water entering the backfilled borehole, which can lead to higher than normal groundwater level readings. Long-term groundwater levels can also be determined based on observations of the recovered soil samples, such as moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is expected at a 5 to 6 m depth.

Table 1 - Groundwater Measurements at Monitoring Well Locations

Test Hole Location	Ground Surface Elevation (m)	GW Level Reading (m)	GW Level Elevation (m)	Date
BH 1	82.55	2.75	79.80	July 7, 2020
BH 2	81.92	3.54	78.38	July 7, 2020
BH 3	82.05	4.72	77.33	July 7, 2020
BH 4	81.35	4.01	77.34	July 7, 2020
BH 5	81.71	3.28	78.43	July 7, 2020
BH 6	82.02	1.88	80.14	July 7, 2020
BH 7	81.61	3.15	78.46	July 7, 2020

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed mid-rise residential building is anticipated to be founded on spread footings placed directly or indirectly by the use of a lean concrete in-filled trench on a clean, surface sounded bedrock bearing surface or compact glacial till bearing surface.

Bedrock removal may be required to complete the underground level. Hoe ramming is an option where only small quantities of bedrock need to be removed. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Since the building will occupy the entire boundaries of the subject site, it is expected that most of the overburben will be removed to bedrock. Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (15 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of 1,500 kPa.

Vibration Considerations

Construction operations are 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, as much as possible, a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

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

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective 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

Footings placed on an undisturbed, **dense glacial till bearing surface** can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. 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.

Footings placed on the upper levels of the **fractured limestone** bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. Where the design underside of footing is slightly above the bedrock surface, footings can be placed over concrete in-filled (17 MPa). zero entry, near vertical trenches extended to a surface sounded bedrock bearing surface using the same bearing resistance values. The concrete in-filled trenches should extend a minimum 300 mm beyond the footing faces in all directions.

A factored bearing resistance value at ULS of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5 if founded on **clean, surface sounded limestone bedrock** and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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

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

Soil/Bedrock Transition

It is expected that not all footings will be founded on bedrock. Where the building is founded on the glacial till deposit, it is recommended to decrease the soil bearing capacity by 25% for the footing placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that a 2 m transition zone composed of 0.5 m layer of nominally compacted OPSS Granular A or Granular B type II be placed directly on sound bedrock. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition should be placed in the top part of the footing and foundation walls.

Raft Foundation

Alternatively, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional spread footing foundation. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **250 kPa** can be used for design purposes. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal associated with one underground parking level. The factored bearing resistance (contact pressure) at ULS can be taken as **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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.

Base on a single underground parking level or more it is expected that the raft foundation will be installed on the glacial till deposit. The modulus of subgrade reaction was calculated to be **30 MPa/m** for a contact pressure of 250 kPa. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. However, a higher site class (**Class A or B**) can be achieved. The higher site class will require a site specific shear wave velocity test to be completed in confirmation of the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed building, the native soil surface, bedrock or approved engineered fill pad will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. A clear crushed stone fill is recommended for backfilling below the floor slab for limited span slab-on-grade areas, such as front porch or garage footprints. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone below basement floor slabs.

It is expected that the basement area will be mostly parking and a rigid pavement structure designed by a structural engineer will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab 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 98% of its SPMDD.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.01 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, 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. 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.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

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 Conditions

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 Structure

For design purposes, the flexible pavement structure presented in the following table could be used for the design of car only parking areas in the lower level of the parking garage.

Table 4 - Recommended Pavement Structure - 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 silty clay or sand or crushed stone material placed over in situ soil.	

Table 5 - Recommended Pavement Structure - Local Roadways, Access Lanes and Heavy Vehicle Parking

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ silty clay or sand or crushed stone material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for parking areas and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

The proposed pavement structure, where it abuts the existing pavement, should match the existing pavement layers. It is recommended that a 300 mm wide and 50 mm deep stepped joint be provided where the new asphalt layer joins with the existing asphalt layer to provide more resistance to cracking at the joint.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

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

A waterproofing membrane will be required to lessen the effect of water infiltration for the lower P-2 basement level. The waterproofing membrane can be placed and fastened to the shoring system (soldier pile and timber lagging) and should extend to the bottom of the excavation at the founding level of the raft foundation.

It is recommended that the composite drainage system, such as Delta Drain 6000 or equivalent, extend from the exterior finished grade to the founding elevation (underside of raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

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

Underfloor Drainage

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

Adverse Effects of Dewatering on Adjacent Properties

It is understood that up to 2 underground parking levels are planned for the proposed development, with the lower portion of the foundation having a groundwater infiltration control system in place. The existing buildings along the west portion are expected to be founded over bedrock or within the glacial till above the bedrock surface.

Based on field observations and assessment, the groundwater level is anticipated at a 5 to 6 m depth below existing grade. A local groundwater lowering is expected under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal groundwater lowering. It should also be noted that the lower portion of the foundation walls will be waterproofed which will limit groundwater lowering within the subject site and surroundings.

Since the neighbouring structures are founded within native glacial till or directly over a bedrock bearing surface based on available soils information. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

Foundation Backfill

Above the bedrock surface, 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 Miradrain G100N or 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 of Footings Against Frost Action

The parking garage 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.

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

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

6.3 Excavation Side Slopes

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority of the excavation to be constructed by open-cut methods (i.e. 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 will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring 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 representative prior to implementation.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 6 - Soil Parameters 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

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Concrete Underpinning

Based on proximity of existing adjacent buildings support in the form of concrete underpinning maybe required during excavation for the proposed building. It is expected that the founding elevations of the existing foundations will be in close proximity to the bedrock surface (less than 1.5 m) and conventional concrete underpinning may be used to support the full width and length of the foundation.

It is expected that the structural engineer along with the geotechnical engineer will review the site conditions at the time of construction and finalize the underpinning program based on their observations at that time.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

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

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.

A temporary Ministry of Environment, Conservation and Parks (MECP) Category 3 Permit to Take Water (PTTW) may be required if more than 400,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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

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 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The analytical testing results indicate that the sulphate content is less tan 0.1%. This results indicates that Type 10 Portland Cement (i.e. normal cement) would be appropriate for this site. The chloride content and pH of the samples indicate that they are not significant factors in creating a corrosive environment, whereas the resistivity is indicative of an aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed foundation drainage design and requirements.

☐ 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.

☐ Field density tests to determine the level of compaction achieved.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A 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, 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 Brigil or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 21, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

- ☐ Brigil
- ☐ Paterson Group

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. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2020

FILE NO. PG5172

HOLE NO. BH 1

GROUND SURFACE

SOIL DESCRIPTION	Depth to base (m)
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.51
Hard to very stiff, brown **SILTY CLAY**	3.05
GLACIAL TILL: Compact to dense, brown silty sand and gravel	5.84

End of Borehole

Practical refusal to augering at 5.84m depth.

(GWL @ 2.75m - July 7, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	67	10
SS	3	67	14
SS	4	100	10
SS	5	100	28
SS	6	33	8
SS	7	33	4
SS	8	43	50+

DEPTH (m)	ELEV. (m)
0	82.55
1	81.55
2	80.55
3	79.55
4	78.55
5	77.55

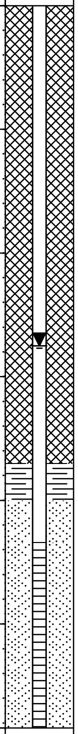
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

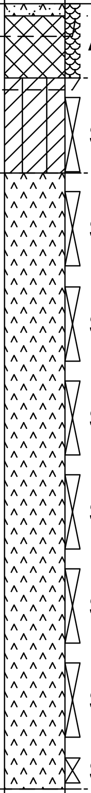
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2020

FILE NO. PG5172

HOLE NO. BH 2

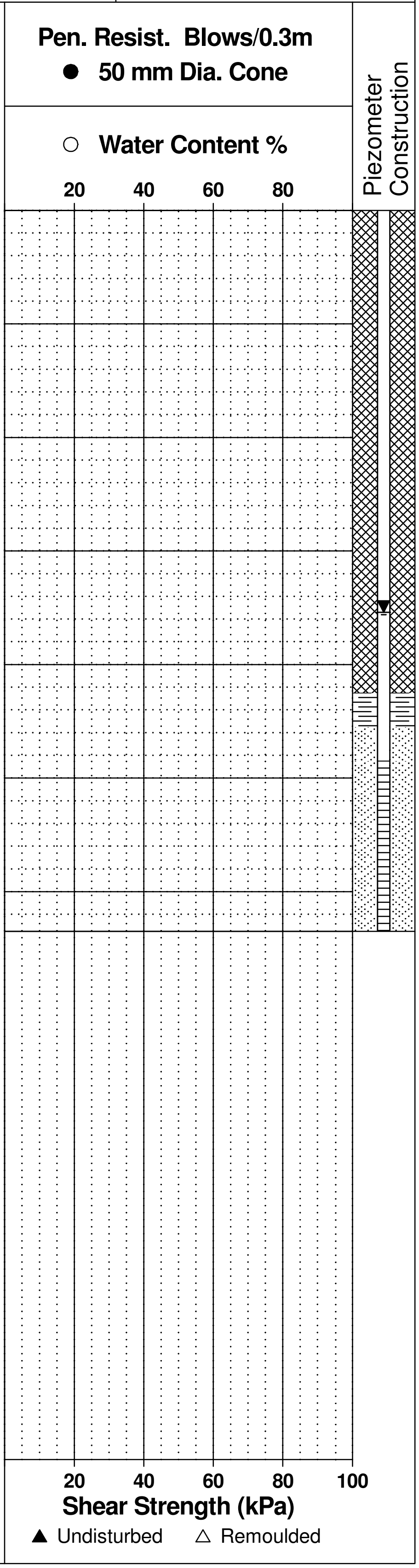


SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.92
Asphaltic concrete 0.10						
FILL: Brown silty sand with crushed stone 0.60	AU	1				
Very stiff, brown **SILTY CLAY,** trace sand 1.37	SS	2	29	11	1	80.92
GLACIAL TILL: Compact to dense, brown silty sand with clay and gravel, some cobbles	SS	3	67	18	2	79.92
	SS	4	33	19	3	78.92
	SS	5	33	25	4	77.92
	SS	6	25	16	5	76.92
	SS	7	33	19	6	75.92
	SS	8	83	4		
6.35	SS	9	50	50+		

End of Borehole

Practical refusal to augering at 6.35m depth.

(GWL @ 3.54m - July 7, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5172

REMARKS

HOLE NO. BH 3

BORINGS BY CME-55 Low Clearance Drill

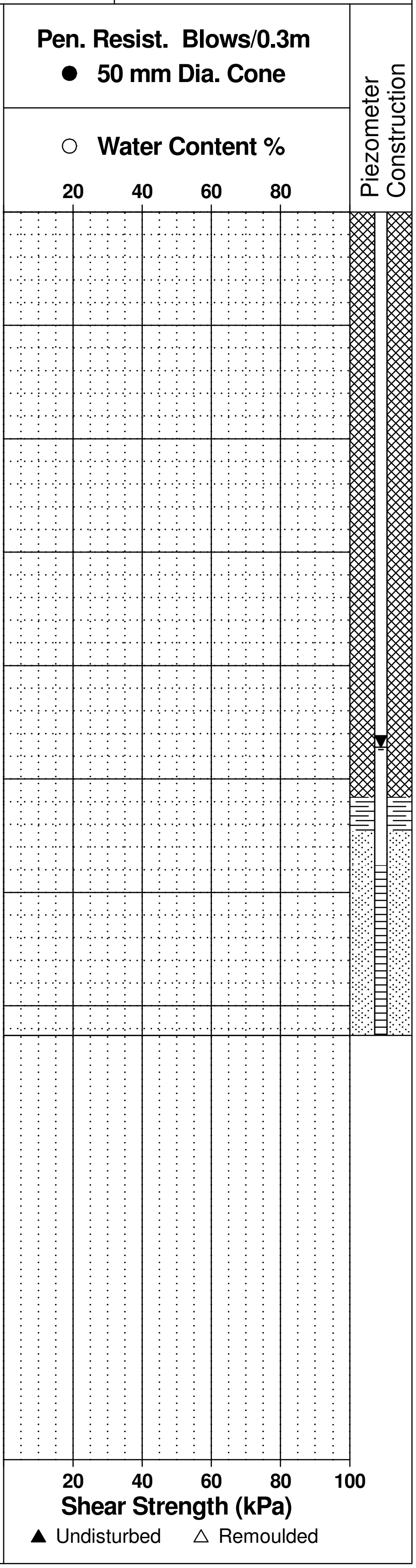
DATE June 25, 2020

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	82.05
Asphaltic concrete 0.10						
FILL: Brown silty sand with crushed stone 0.60	AU	1				
Very stiff, brown **SILTY CLAY,** trace sand	SS	2	75	12	1	81.05
	SS	3	100	12	2	80.05
2.90	SS	4	100	11		
GLACIAL TILL: Comapct to dense, brown silty sand, some clay and gravel, trace cobbles	SS	5	50	21	3	79.05
	SS	6	58	21	4	78.05
	SS	7	58	8	5	77.05
	SS	8	17	44	6	76.05
	SS	9	8	8		
7.26	SS	10	25	50+	7	75.05

End of Borehole

Practical refusal to augering at 7.26m depth.

(GWL @ 4.72m - July 7, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2020

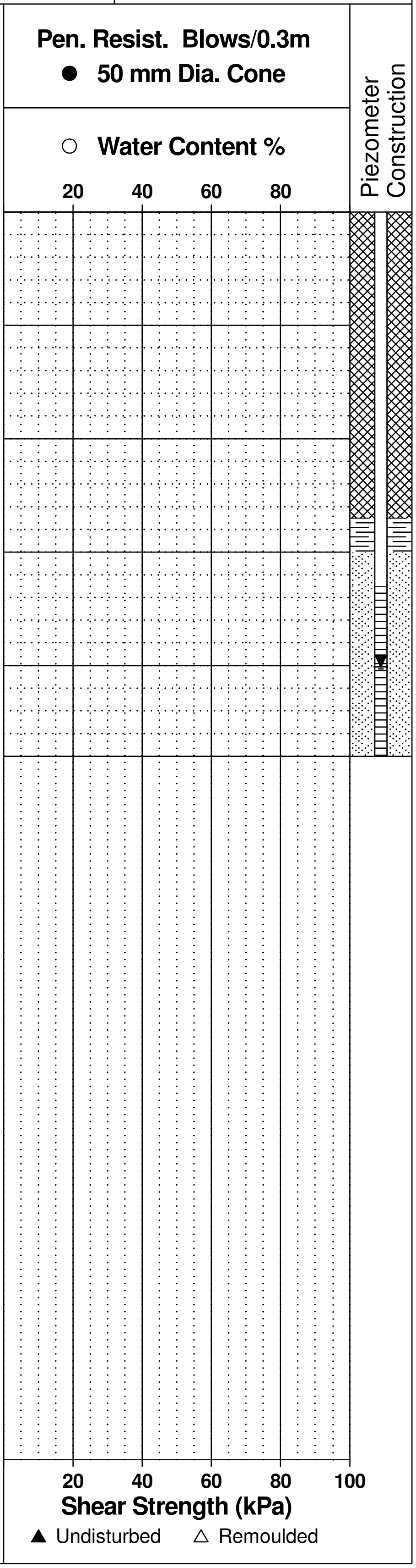
FILE NO. PG5172

HOLE NO. BH 4

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.35
Asphaltic concrete	0.10						
FILL: Brown silty sand with crushed stone, trace cobbles		AU	1				
		SS	2	67	47	1	80.35
	2.29	SS	3	8	8	2	79.35
Very stiff, brown **SILTY CLAY,** trace sand - sand seam at 3.4m depth		SS	4	100	11	3	78.35
	3.81	SS	5		11		
GLACIAL TILL: Dense, brown silty sand, some gravel, trace clay, cobbles and boulders		SS	6		41	4	77.35
	4.80	SS	7		50+		
End of Borehole							

Practical refusal to augering at 4.80m depth.

(GWL @ 4.01m - July 7, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2020

FILE NO. PG5172

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.71
Asphaltic concrete 0.10						
FILL: Brown slty sand with crushed stone 0.60	AU	1				
Hard to very stiff, brown **SILTY CLAY**	SS	2	100	8	1	80.71
	SS	3	100	13	2	79.71
2.90	SS	4	100	9		
GLACIAL TILL: Loose to compact, brown silty sand, some gravel, trace clay and cobbles	SS	5	67	8	3	78.71
	SS	6	42	15	4	77.71
- grey by 4.9m depth	SS	7	67	12	5	76.71
5.89	SS	8	50	13		

End of Borehole

Practical refusal to augering at 5.89m depth.

(GWL @ 3.28m - July 7, 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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 26, 2020

FILE NO. PG5172

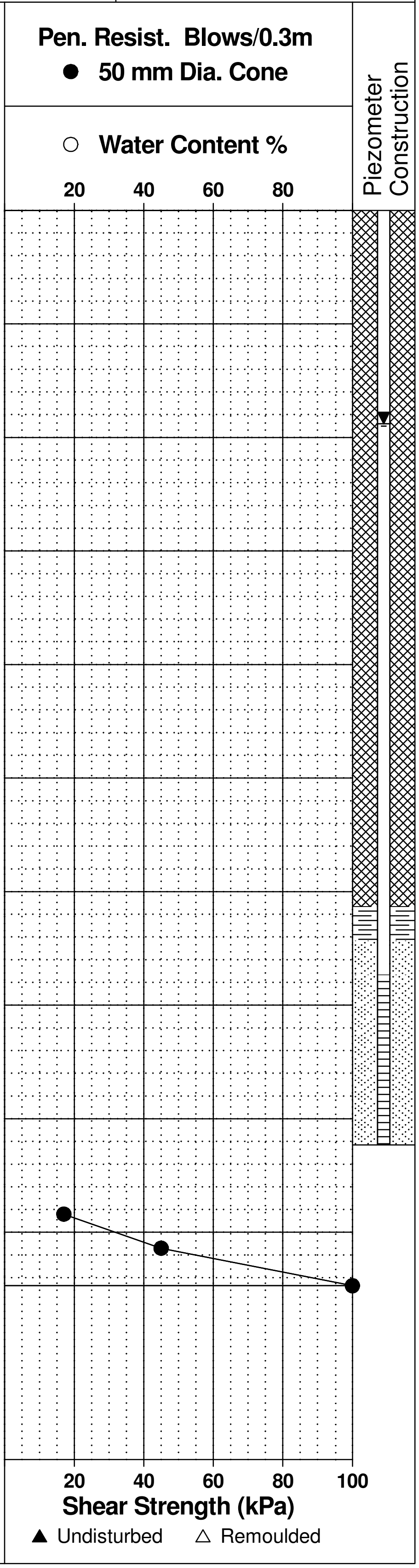
HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	82.02
Asphaltic concrete 0.08						
FILL: Brown silty sand with crushed stone 0.51	AU	1				
FILL: Brown silty sand, some gravel, trace cobbles					1	81.02
	SS	2	25	54		
	SS	3	17	33		
2.13					2	80.02
GLACIAL TILL: Brow silty clay with sand and gravel	SS	4	92	10		
					3	79.02
	SS	5	92	11		
	SS	6	75	18		
					4	78.02
	SS	7	75	23		
					5	77.02
	SS	8	42	13		
6.10					6	76.02
GLACIAL TILL: Grey silty sand, some gravel, trace clay	SS	9	67	3		
					7	75.02
	SS	10	17	1		
	SS	11		25		
8.23					8	74.02
Dynamic Cone Penetration Test commenced at 8.23m depth.						
					9	73.02
Inferred **GLACIAL TILL** 9.47						

End of Borehole

Practical DCPT refusal at 9.47m depth

(GWL @ 1.88m - July 7, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 729 to 753 Ridgewood Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2020

FILE NO. PG5172

HOLE NO. BH 7

SOIL DESCRIPTION	Bottom depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.76
FILL: Brown silty clay, trace sand and gravel	1.37
FILL: Brown silty sand, trace gravel	1.68
FILL: Brown silty clay, trace sand	2.13
GLACIAL TILL: Brown silty clay with sand and gravel, trace cobbles	4.57
GLACIAL TILL: Grey silty sand, trace clay, gravel and cobbles	5.87
BEDROCK: Fair to poor quality, grey limestone	9.04

End of Borehole

(GWL @ 3.15m - July 7, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	10
SS	3	100	14
SS	4	83	22
SS	5	92	39
SS	6	4	29
SS	7	33	15
SS	8	38	19
RC	1	92	65
RC	2	100	48

DEPTH (m)	ELEV. (m)
0	81.61
1	80.61
2	79.61
3	78.61
4	77.61
5	76.61
6	75.61
7	74.61
8	73.61
9	72.61

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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

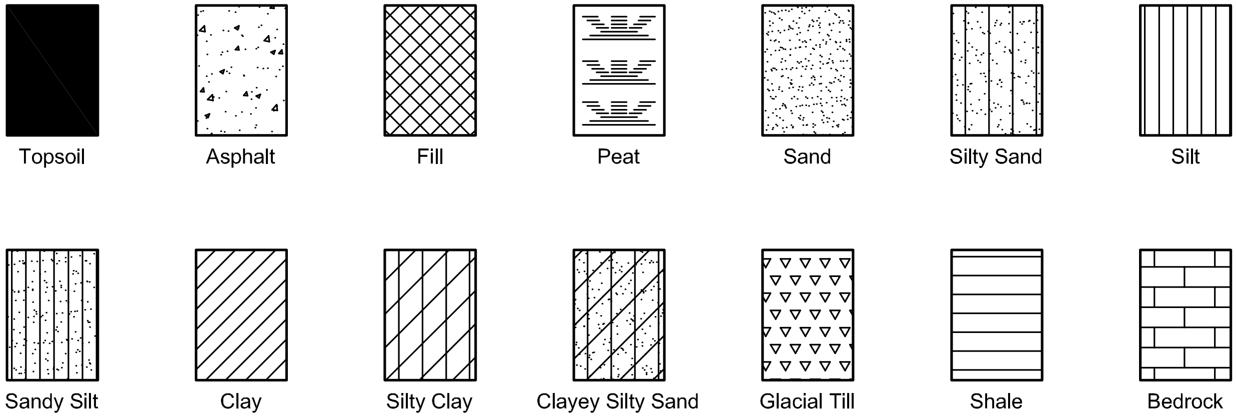
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

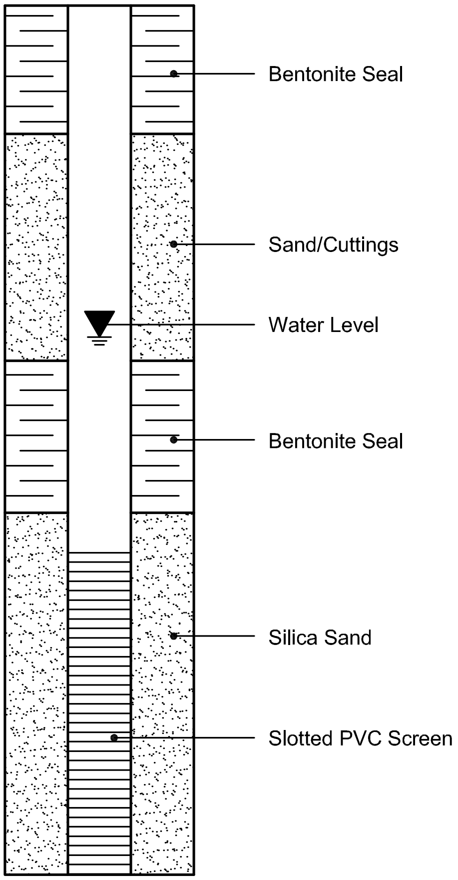
SYMBOLS AND TERMS (continued)

STRATA PLOT

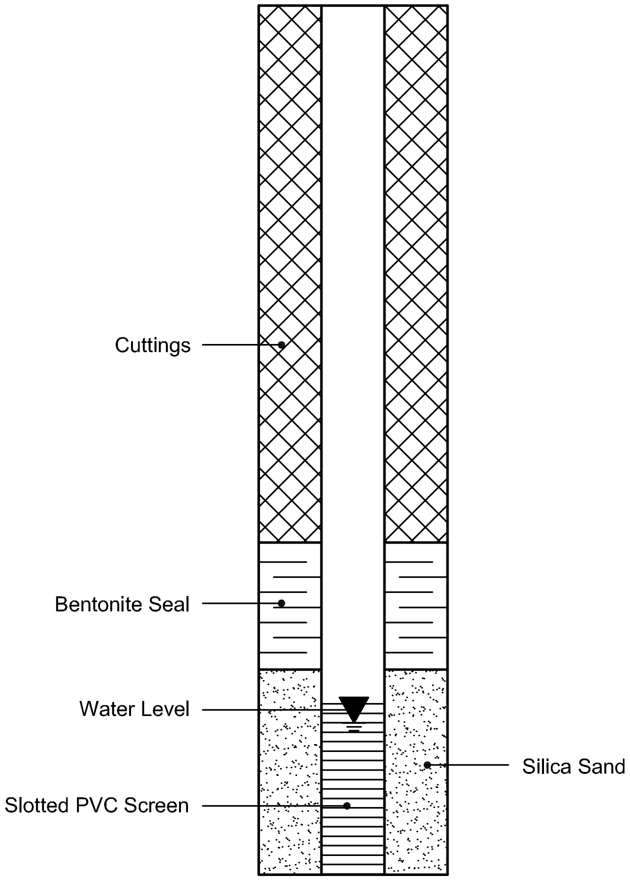


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24713

Report Date: 03-Jul-2020

Order Date: 26-Jun-2020

Project Description: PG5172

Client ID:		BH3-SS3	-	-	-
Sample Date:		25-Jun-20 13:00	-	-	-
Sample ID:		2026529-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.58	-	-	-
Resistivity	0.10 Ohm.m	13.3	-	-	-
Anions					
Chloride	5 ug/g dry	179	-	-	-
Sulphate	5 ug/g dry	389	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5172-1 - TEST HOLE LOCATION PLAN

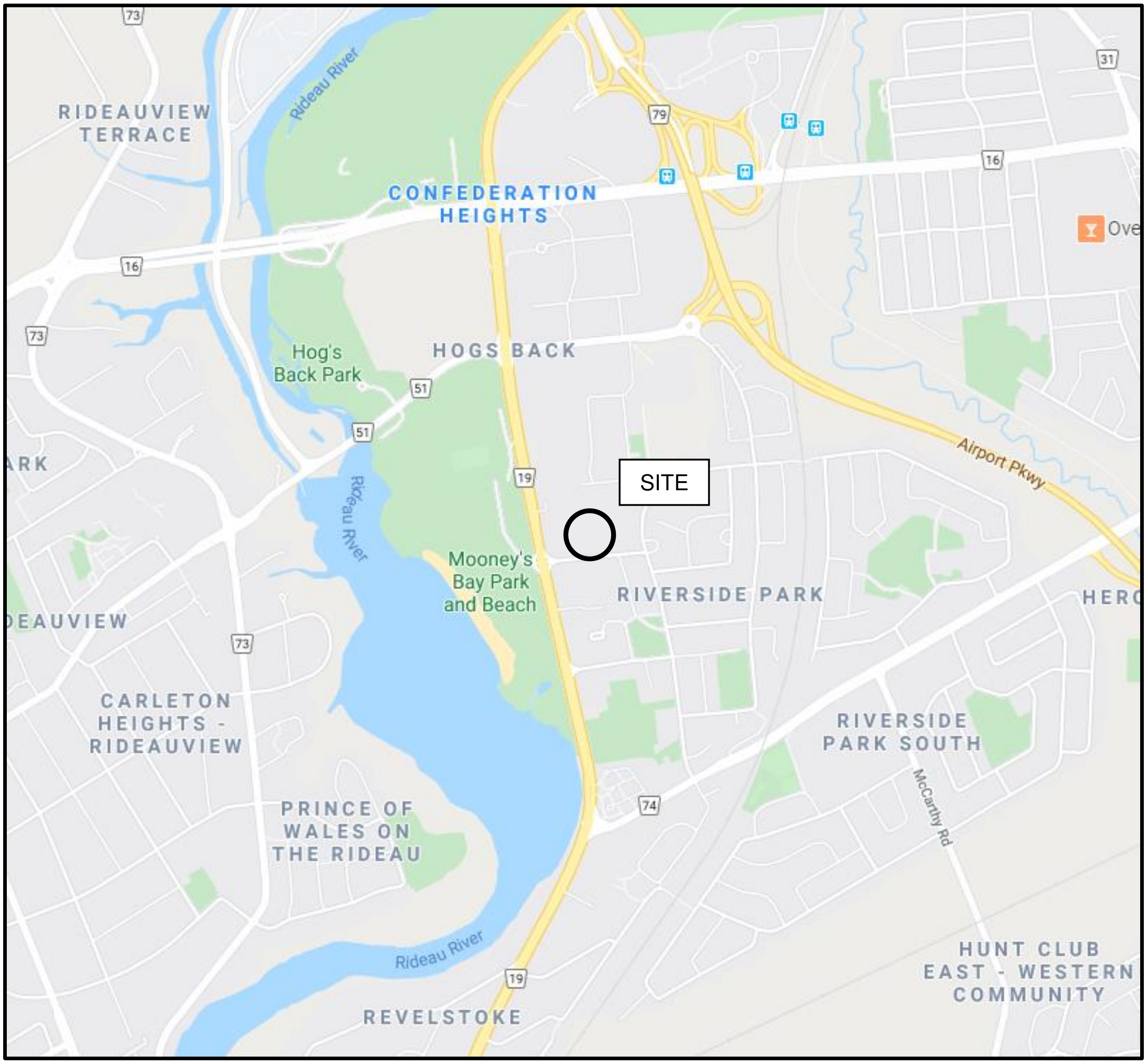


FIGURE 1

KEY PLAN

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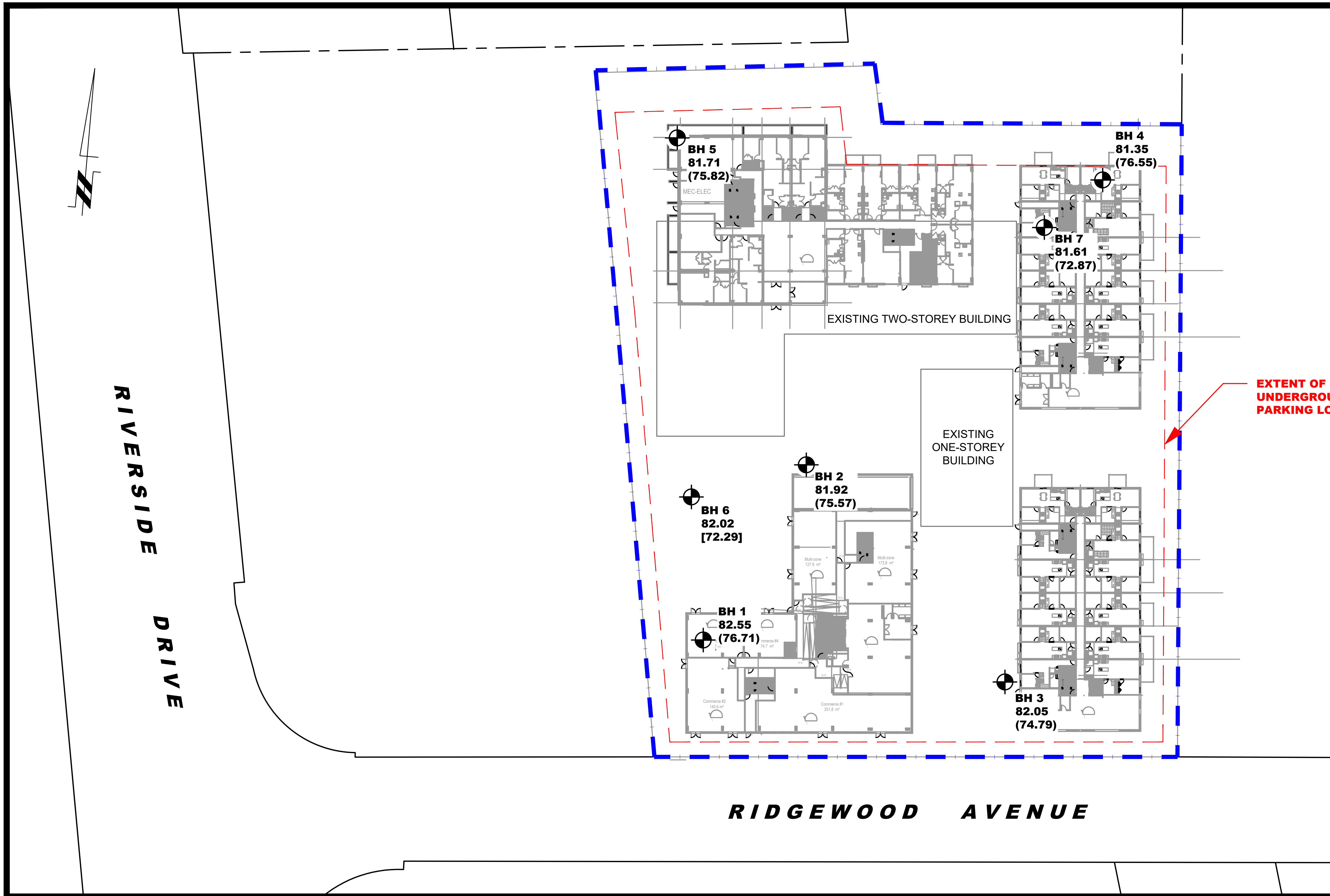

LEGEND:

⌖	BOREHOLE LOCATION
82.05	GROUND SURFACE ELEVATION (m)
(74.79)	REFUSAL TO AUGERING ELEVATION (m)
[72.29]	BEDROCK SURFACE ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS REFERENCE A GEODETIC DATUM (NAD83)

SCALE: 1:750

0 10 20 30 50m

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NO.	REVISIONS	DATE	INITIAL
1	CONCEPTUAL SITE PLAN UPDATED	16/06/2021	JV

BRIGIL
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Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 07/2020
Drawn by: NFRV	Report No.: PG5172-1
Checked by: JV	Dwg. No.: PG5172-1
Approved by: DJG	Revision No.: 1

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