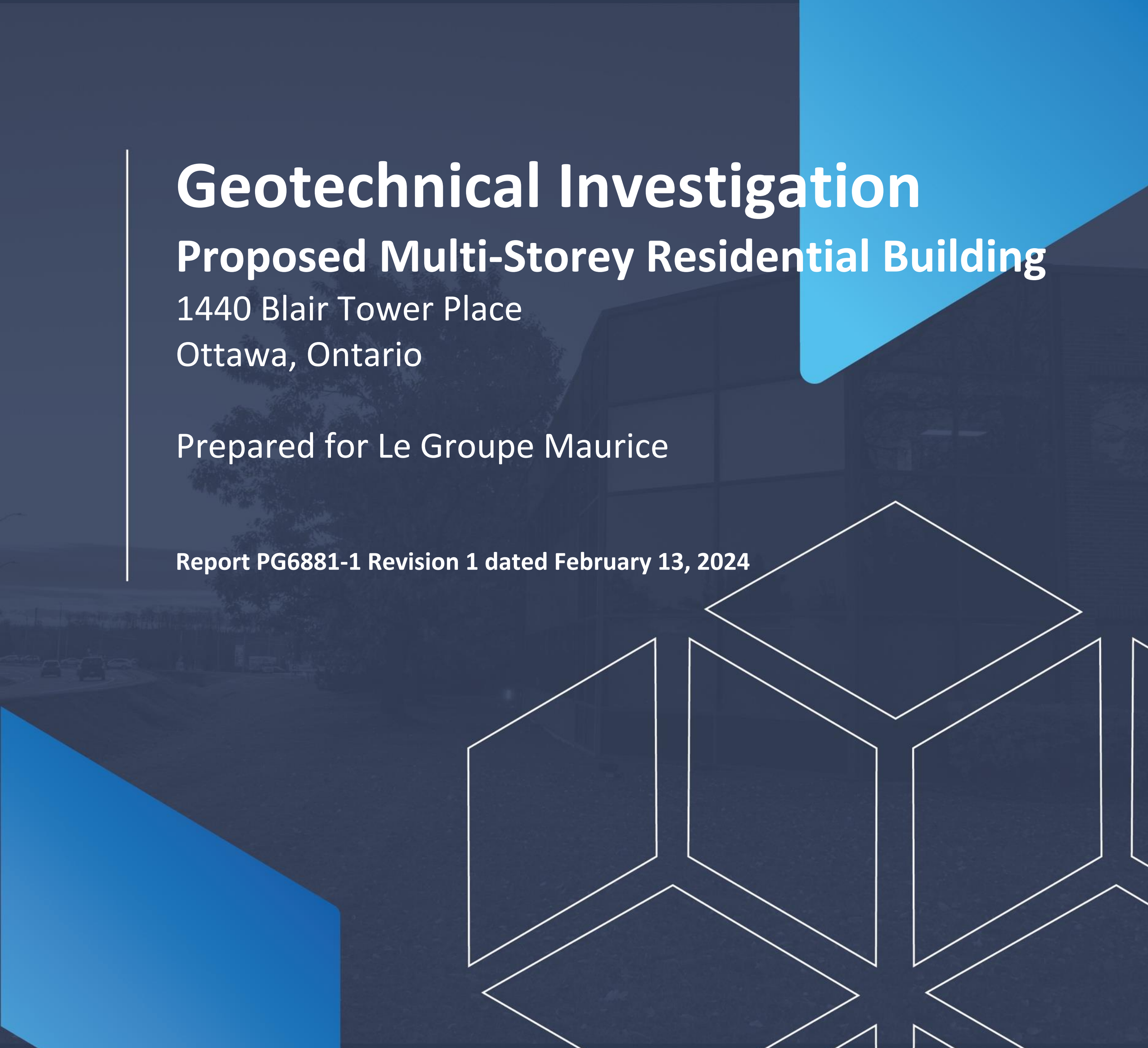

Geotechnical Investigation

Proposed Multi-Storey Residential Building

1440 Blair Tower Place
Ottawa, Ontario

Prepared for Le Groupe Maurice

Report PG6881-1 Revision 1 dated February 13, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Le Groupe Maurice to conduct a geotechnical investigation for the proposed multi-storey residential building consisting of a retirement residence to be located at 1440 Blair Tower Place 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 Project

Based on the available information at the time of completing the report, the proposed project will consist of a multi-storey residential building serving as a retirement residence. It is understood that the proposed building will consist of two high-rise towers connected by a low-rise central portion. It is understood that 1 or 2 below grade levels are being considered. A park area is proposed to the north of the building and paved access roads and landscaped areas are proposed on the east and west side of the proposed building.

It is anticipated that the site will be municipally serviced by water, storm, and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on November 7, 8 and 9, 2023, and consisted of a total of six (6) boreholes sampled to a maximum depth of 9.2 m below the existing grade throughout the subject site. The borehole locations were determined in the field by Paterson personnel in a manner to provide general coverage of the subject site, taking into consideration existing site features and underground services. A previous investigation was conducted by others on December 8 and 9, 2021 on site and consisted of five (5) boreholes advanced to a maximum depth of 11.4 m below the existing grade. The locations of the boreholes are illustrated on Drawing PG6804-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson’s geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, 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.

Rock samples were recovered from all boreholes 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 of this report.

Groundwater

Groundwater monitoring wells were installed in boreholes BH 2-23 and BH 6-23, and flexible standpipe piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevations at the borehole locations, were selected surveyed using a handheld GPS unit and are referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG6881-1 - 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 (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 samples were 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 site is presently occupied by a paved access road to the adjacent parking garage and furthermore grassed and soft landscaped areas surrounding the paved road. The east and west property line are partly covered with some trees and more trees are located within the north portion of the site.

The site is bordered to the north by Ogilvie Road, to the east by a commercial development and parking lot, to the south by a commercial development and parking lot structure and to the west by Blair Road. The site is relatively level and at grade with the adjacent roadway at an approximate geodetic elevation of 80.2 m to 80.6 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the boreholes consists of fill overlying glacial till. The fill layer was observed to be composed of varying amounts of sand, silt, gravel, crushed stone, clay and cobbles. The fill was noted to extend to depths ranging from 1.45 m to 2.44 m below the existing ground surface in BH 1-23 to BH 4-23 and BH 6-23. The fill was noted to extend to a greater depth of 4.9 m below the existing ground surface in BH 5-23

The fill was underlain by a glacial till consisting of brown silty sand with varying amounts of gravel, clay, shale fragments and cobbles, extending to depths ranging from 3.2 m to 5.4 m below the existing ground surface in BH 1-23 to BH 4-23 and BH 6-23. The fill layer was noted to extend to the bedrock surface in BH 5-23.

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 all six borehole locations to confirm refusal. Black shale bedrock was encountered at depths ranging from 3.2 m to 5.4 m below the existing ground surface.

Upon review of the core hole sample, the bedrock was found to be of very poor to fair quality for the first 1.5 m to 3.0 m. The bedrock was noted to be of good to excellent quality at geodetic elevations ranging from 73.9 m to 74.4 m.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Billings Formation. The overburden drift thickness is anticipated to be between 2 to 4 m.

Groundwater

Flexible piezometers and monitoring wells 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.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	80.14	4.30	75.84	November 15, 2023
BH 2-23*	80.64	3.55	77.09	
BH 3-23	80.42	Piezometer Blocked		
BH 4-23	80.31	2.81	77.5	
BH 5-23	80.14	Piezometer Blocked		
BH 6-23*	80.26	2.60	77.66	

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

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, coloring and consistency. Based on these observations, the long-term groundwater level is expected to be at a 3.5 to 4.5 m depth.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed residential building is anticipated to be founded on spread footings or short piles placed on the bedrock surface and/or approved engineered fill.

Bedrock removal could be required to complete the underground level. Hoe ramming is a feasible option where only small quantities of bedrock need to be removed or only the upper poor to fair quality shale bedrock is to be removed. It is expected that the bedrock removal will all be completed by hoe ramming and excavating, and that blasting will not be used for this project.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Bedrock Removal

Based on the bedrock encountered in the area, it is expected the bedrock removal will be completed by hoe-ramming. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be completed by an excavator.

Vibration may be induced by rock removal activities. As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s for low frequencies during a blasting program to reduce the risks of damage to the existing structures. If blasting is considered the 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.

It is not recommended that the shale bedrock be reused under settlement sensitive structure or as backfill in the active frost layer (2.1 m).

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 equipment could be the source of vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. 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 there are several sensitive buildings near the subject site, consideration to lowering these guidelines is recommended. 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.

Bedrock Excavation Face Reinforcement

Where 1:1 sloping of the excavation cannot be accommodated in weathered or fractured horizontal rock, anchors, shotcrete and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface during construction. The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations.

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 imported fill material 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 98% 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 geo-composite drainage membrane such as Miradrain G100N or Delta Drain 6000 connected to a perimeter drainage system as further discussed in Section 6.1.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 150 mm. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. Shale bedrock material should not be used under settlement sensitive structure and pavement areas. The material can be used in landscape areas where settlement will be of minimal impact.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

5.3 Foundation Design

Bearing Resistance Values

Based on current available plans for the project, it is expected that the proposed building will have 1 or 2 underground basement levels.

Should one level of underground basement be selected, it is expected that the footings will be founded on the poor to fair quality weathered shale bedrock at an approximate geodetic elevation ranging from 75.3 m to 77.0 m. Footings placed at or below that elevation can be designed using a bearing resistance value at serviceability limit states (SLS) of **900 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,100 kPa**.

Should two level of underground basement be selected, it is expected that the footings will be founded on the good to excellent quality shale bedrock at an approximate geodetic elevation ranging from 73.9 m to 74.4 m. Footings placed at or below that elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa.**

Footings placed on a compact to 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 **375 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

Footings bearing on an acceptable good to excellent bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 15 and 10 mm respectively.

Footings placed on a soil or weathered bedrock bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 15 mm, respectively.

According to boreholes BH 5 by others, the good to excellent quality bedrock was encountered at an approximate geodetic elevation of 72.5 m which is approximately 1.5 to 1.7 m below the elevation observed during the present investigation. It should be noted that the drilling methods and rock quality designation for the investigation by others could not be confirmed by Paterson and a higher quality rock could be encountered at a higher elevation than noted on BH 5 by others.

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 to 20 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 as provided above.

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:4V (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).

Protection of Potentially Expansive Bedrock

Upon being exposed to air and moisture, shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions. It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20°C, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

Based on the borehole logs, expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 75 mm thick concrete mud slab, consisting of minimum 15 MPa lean concrete, should be placed on the exposed bedrock surface within a 48 hour period of being exposed.

Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

Foundation Option - End Bearing Piled Foundation

Deep foundations such as end bearing piles can provide a higher bearing capacity where the underside of footing is located at the elevation of the weathered shale bedrock surface. A deep caisson or micropile foundation system extending to the good to excellent quality shale bedrock surface would provide minimal spoil generation and a higher bearing capacity.

Driven piles are not recommended for the site. Based on local experience the existing weathered shale bedrock will retain the pile during the driving activities and capacities will not be achieved.

Drilled Shafts and Caissons

Cast-in-place caissons/Drilled shafts can be used where excavation to a proper bearing surface is not achievable. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to fill the caissons. The caissons are to be structurally reinforced over their entire length.

Due to the poor weathered shale surface, it is recommended that caissons be socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 2 below presents the estimated capacity for different typical caisson sizes for a rock socketed caisson extending 7 to 8 m below existing ground surface or to an approximate geodetic elevation of 72.5 m to 73.5 m.

Table 2 – Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)	Factored Capacity Tension at ULS (kN)
inch	mm	Rock Socket	Rock Socket
36	900	5,500	1,250
42	1000	6,000	1,450
48	1200	10,000	1,600
54	1375	13,500	1,700
60	1500	17,000	1,850

notes:
- 5-6 m rock socket in bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

Based on the recent field investigation and bedrock coring it is expected that a highly weathered layer of bedrock will need to be removed to reach a sound surface. The thickness of the weathered layer was evaluated to vary from 0.6 to 1.0 m across the site. It is expected that the deep foundation rig will be able to auger through the weathered bedrock layer.

Caisson lateral capacities have been provided assuming a minimum reinforcement ratio of 0.2% and inclusion of shear reinforcement consisting of reinforcement rings placed 250 to 300 mm apart along the pile length. An increase lateral capacity can be achieved by increasing the reinforcement ratio and the position of the shear reinforcement. This will increase the general stiffness of the element.

The structural caisson designer should review design loads to provide sufficient resistance to the proposed caissons.

Micropiles

Micropiles are small diameter, deep foundation element composed a high strength steel casing and a high yield threaded steel bar core. The piles are advanced by rotary drills and normally develop their strength within the underlaying rock layer. The steel elements are grouted in place using non shrink grout in a similar fashion to rock anchors. Pressure grouting can be used to achieve greater bond strengths. The geotechnical resistance presented in table 3 below can be used for preliminary purposes, based on the available geotechnical information.
It should be noted that the bond length is expected to be located entirely within the good to excellent quality bedrock.

A structural engineer should review the buckling potential, lateral capacity and other structural elements of the micropile design.

Table 3 – Micropile Geotechnical Resistance

Casing Size (mm)	Threaded Bar Size (mm)	Cased Length (m)	Bond Length (m)	Compression Capacity (KN) ULS	Tension Capacity (KN) ULS
150	43	4	4	700	400
175	57	4	4	750	475
250	57	4	5	1,500	775
250	63	4	9	1,500	1,400

Since micropiles are considered permanent for a life span of 100 years, double corrosion and/or sacrificial steel corrosion allowance are to be considered for the threaded bar and steel casing. It is recommended that the micropile pile casing have a minimum wall thickness of 12 mm with a minimum of 3 mm of sacrificial steel.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the foundations considered as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2020. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 for a full discussion of the earthquake design requirements.

It is expected that a higher site class (Class A or B) can be achieved on site. Site specific seismic testing should be completed to confirm.

5.5 Basement Slab and Slab on Grade

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.

The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of conventional spread footings.

For structures with slab-on-grade construction, the upper 300 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD.
Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

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.

It is recommended that a minimum of 500 mm be kept from vertical rock face to foundation wall to avoid supplemental lateral pressure from potential expansive shale.

Where blind side pours are proposed on a vertical rock face, a supplemental layer of a minimum 25 mm of compressible insulation material should be used.

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

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

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 material(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 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 Ottawa is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4. The flexible pavement structure presented in Table 5 and Table 6 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 4 – Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Compact to dense glacial till, or OPSS Granular A or OPSS Granulart B Type II granular fill material placed over in situ soil.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m).

The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 – 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 or fill	

Table 6 – Recommended Pavement Structure – Heavy-Truck Traffic, Loading Areas and Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, bedrock or OPSS Granular B Type I or II material placed over in situ soil, bedrock or fill.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

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 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.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Based on current available information of the proposed project, it is understood that one to two underground basement levels are being considered. The long-term groundwater level on site was observed to vary from 76.0 to 77.0 m. It is expected that localized dewatering will be achievable for the project.

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

Waterproofing 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 and lap under the perimeter footings.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the underground basement level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s) or gravity outlet. The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each floor span and connected to an interior perimeter drainage pipe. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elevator shaft foundation wall and a waterproofing membrane such as BSW H (or approved other) be applied below the elevator pit footing (horizontal application).

The Colphene Torch’n Stick waterproofing membrane should extend over the vertical portion of the slab and down to the top of the footing in accordance with the manufacturer’s specifications. The BSW H waterproofing membrane should be placed horizontally below the footing and extend up the sides of the footing. The Colphene Torch’n Stick should overlap the BSW H waterproofing membrane.

A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.
The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

The foundation wall of the elevator shaft and buildings sump pit should host a PVC sleeve to allow any water trapped within the interior side of the structures to be discharged to the associated sump pump. A minimum 100 mm diameter perforated, corrugated drainage pipe should extend from the sleeve towards the associated drainage system by gravity drainage and mechanical connection to the associated system. Also, the contractor should ensure that the opening is properly sealed to prevent water from entering the subject structure.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the pit structure and bedrock/soil excavation face can be in-filled with lean concrete, OPSS Granular A or Granular B Type II crushed stone.

It should be noted that a waterproofed concrete (with Xypex Additive, or equivalent) is optional for this waterproofing option.

Adverse Effects from Dewatering on Adjacent Structures

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 suppression system in place. The existing buildings along the east 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 3.5 to 4.5 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.

Since the neighboring 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

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.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

The underground basement level is expected to be heated. Thus, perimeter footings of the underground basement 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.

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

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible (i.e., weathered bedrock or bedrock with significant fissures filled with soil), foundation insulation will need to be provided. Alternatively, frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 15 to 20 MPa 28-day strength). It is recommended Paterson field personnel review the frost susceptibility of bedrock surface located within 1.8 m of finished grade.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for the majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending through fill material and weathered shale bedrock should be excavated at 1H:1V or shallower. The flatter slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

Excavation in the fractured and sound bedrock is expected to be carried with near vertical side walls. Shotcrete protection is expected to be required in the fractured layer of bedrock for worker's protection where the vertical excavation is greater than 1.2m. Paterson should review the excavation side walls during construction.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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

Temporary Shoring

Temporary shoring may be required to support the overburden soils 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 system could consist of soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.
The earth pressures acting on the shoring system may be calculated using the following parameters provided in Table 7.

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 is 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.

Table 7 – Geotechnical Parameters Lateral Properties

Material Description	Unit Weight (kN/m³)		Friction Angle (°) φ'	Friction Factor, tanδ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
Existing Fill	18	10	30	0.4	0.33	0.5	3
Sandy Silt	18	10	30	0.4	0.33	0.5	3
Engineered Fill (Granular A)	22	13.5	40	0.5	0.22	0.36	4.6
Engineered Fill (Granular B Type II)	22.5	14	42	0.5	0.2	0.33	5.04
Bedrock	23.5	15.2	55	0.6	0.18	0.1	10

Notes:

I. The earth pressure coefficients provided are for horizontal profile.

II. For soil above the groundwater level the “drained” unit weight should be used and below groundwater level the “effective” unit weight should be used.

III. Existing fill should be free of significant amounts of deleterious material such as those containing organic materials, wood chips and peat. The fill should be approved by Paterson prior to placement

6.4 Pipe Bedding and Backfill

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

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

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

Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

For typical ground or surface water volumes, being pumped during the construction phase, 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 bedrock and overburden material present on site are considered frost susceptible.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions, in particular, where a shoring system is constructed, 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.

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

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summertime conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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 samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects should be performed by Paterson:

☐ Review preliminary and detailed grading, servicing, and structural plan(s) from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction, if applicable.

☐ Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

☐ Review the bedrock stabilization and excavation requirements at the time of construction.

☐ Review and inspection of the installation of the foundation and underfloor waterproofing and drainage systems and elevator waterproofing.

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

☐ 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 the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson. All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

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

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractor’s construction methods, equipment capabilities and schedules.

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 Le Groupe Maurice 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.

LICENSED PROFESSIONAL ENGINEER
Feb. 13, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Nicolas Seguin

Nicolas Seguin, EIT, CPI

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

Report Distribution:

☐ Le Groupe Maurice (e-mail copy)
☐ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374659.712 **NORTHING:** 5033074.891 **ELEVATION:** 80.14

PROJECT: Proposed Residential Development

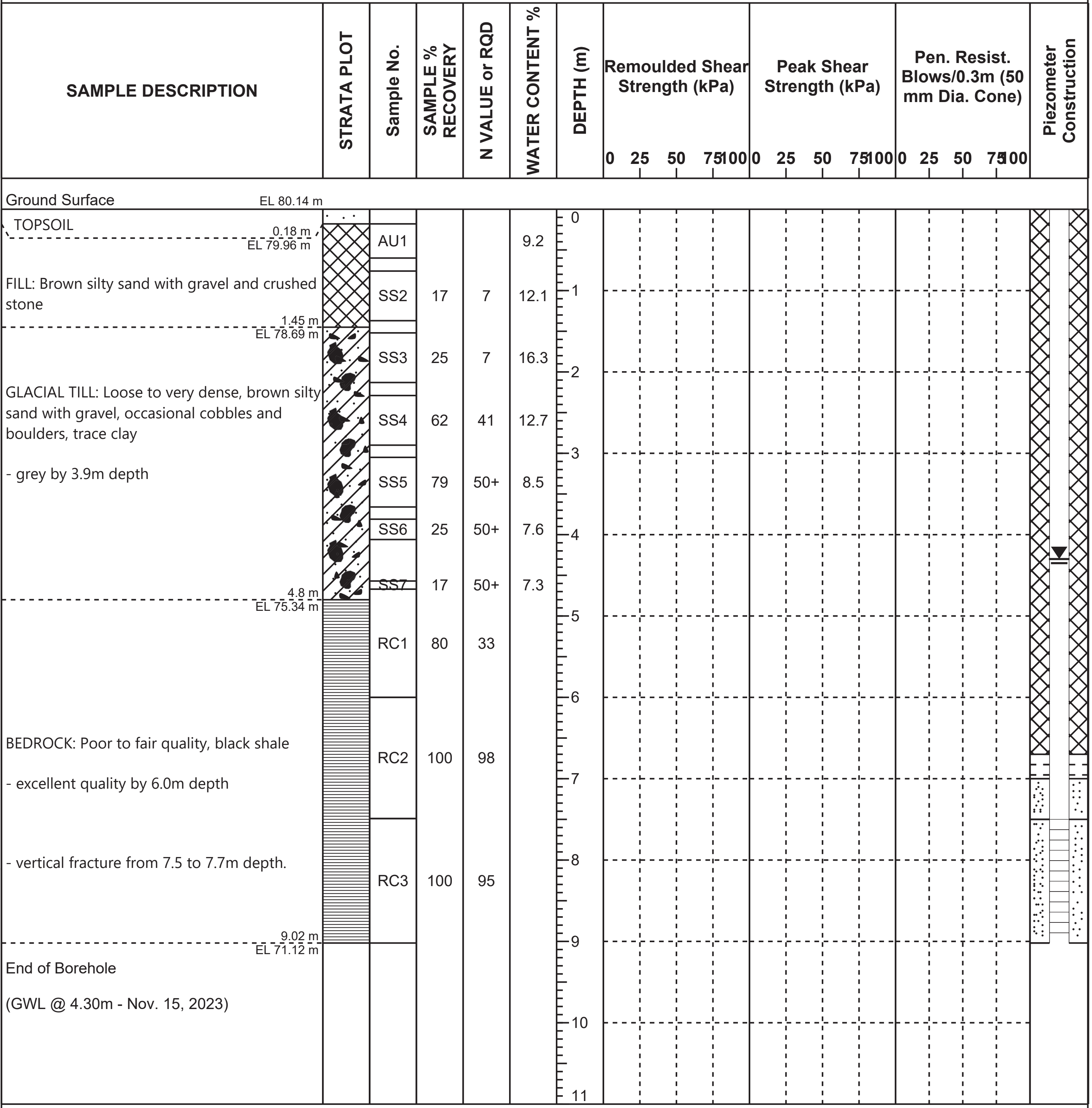
BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: November 7, 2023

FILE NO. PG6881

HOLE NO. BH 1-23



SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
Ground Surface EL 80.14 m				
TOPSOIL 0.18 m EL 79.96 m	AU1			9.2
FILL: Brown silty sand with gravel and crushed stone	SS2	17	7	12.1
1.45 m EL 78.69 m				
GLACIAL TILL: Loose to very dense, brown silty sand with gravel, occasional cobbles and boulders, trace clay - grey by 3.9m depth	SS3	25	7	16.3
	SS4	62	41	12.7
	SS5	79	50+	8.5
	SS6	25	50+	7.6
	~~SS7~~	17	50+	7.3
4.8 m EL 75.34 m				
BEDROCK: Poor to fair quality, black shale - excellent quality by 6.0m depth - vertical fracture from 7.5 to 7.7m depth.	RC1	80	33	
	RC2	100	98	
	RC3	100	95	
9.02 m EL 71.12 m				
End of Borehole (GWL @ 4.30m - Nov. 15, 2023)				

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374623.796 **NORTHING:** 5033089.578 **ELEVATION:** 80.64

PROJECT: Proposed Residential Development

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: November 7, 2023

FILE NO. **PG6881**

HOLE NO. BH 2-23

SAMPLE DESCRIPTION

Ground Surface EL 80.64 m

TOPSOIL 0.18 m EL 80.46 m

FILL: Brown silty sand with topsoil, some gravel and crushed stone 0.84 m EL 79.8 m

FILL: Brown silty sand, some gravel, trace clay, occasional shale fragments 1.45 m EL 79.19 m

GLACIAL TILL: Compact to very dense, brown silty sand, some gravel, occasional cobbles 4.98 m EL 75.66 m

BEDROCK: Very poor quality, black shale

- excellent quality by 6.2m depth

9.2 m EL 71.44 m

End of Borehole

(GWL @ 3.55m - Nov. 15, 2023)

STRATA PLOT

Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1			19.6
SS2	50	12	16.3
SS3	25	8	24.4
SS4	88	25	14.9
SS5	58	50+	10.7
RC1	64	11	
RC2	67	17	
RC3	100	100	
RC4	100	100	

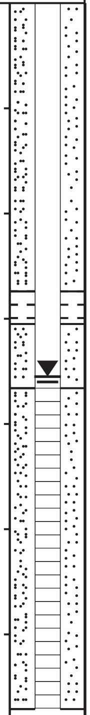
DEPTH (m) 0 1 2 3 4 5 6 7 8 9 10 11

Remoulded Shear Strength (kPa) 0 50 100

Peak Shear Strength (kPa) 0 50 100

Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100

Monitoring Well Construction



PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374614.814 **NORTHING:** 5033110.827 **ELEVATION:** 80.42

PROJECT: Proposed Residential Development

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: November 8, 2023

FILE NO. PG6881

HOLE NO. BH 3-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Piezometer Construction
Ground Surface EL 80.42 m						0				
TOPSOIL 0.23 m EL 80.19 m		AU1			18.8					
FILL: Brown silty sand with crushed stone, trace clay, occasional cobbles 1.45 m EL 78.97 m		SS2	29	10	24.8	1				
GLACIAL TILL: Compact, brown silty sand with gravel, occasional cobbles and boulders, trace clay - shale fragments by 2.3m depth 4.8 m EL 75.62 m		SS3	54	15	16.6	2				
		SS4	79	16	15.6	3				
		SS5	25	50+	11.7					
		RC1	35	0		4				
BEDROCK: Poor to fair quality, black shale - excellent quality by 6.1m depth 9.12 m EL 71.3 m		RC2	77	35		5 6				
		RC3	100	100		7				
		RC4	100	100		8 9				
End of Borehole (Piezometer blocked - Nov. 15, 2023)						10 11				

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374611.967 **NORTHING:** 5033150.584 **ELEVATION:** 80.31

PROJECT: Proposed Residential Development

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: November 8, 2023

FILE NO. **PG6881**

HOLE NO. BH 4-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Piezometer Construction
Ground Surface EL 80.31 m						0				
TOPSOIL 0.15 m EL 80.16 m		AU1			9.49					
FILL: Brown silty sand with crushed stone, trace clay		SS2	79	6	9 16.2	1				
1.45 m EL 78.86 m		SS3	38	11	9.8	2				
GLACIAL TILL: Loose to compact, brown silty sand, some gravel, trace clay and shale fragments		SS4	71	6						
		SS5	58	50	11.4	3				
3.76 m EL 76.55 m		RC1	100	27		4				
BEDROCK: Poor quality, black shale		RC2	92	33		5				
- excellent quality by 6.1m depth		RC3	100	98		6 7				
		RC4	100	98		8				
9.12 m EL 71.19 m						9				
End of Borehole						10				
(GWL @ 2.81m - Nov. 15, 2023)						11				

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374659.03 **NORTHING:** 5033116.657 **ELEVATION:** 80.14

PROJECT: Proposed Residential Development

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: November 8, 2023

FILE NO. PG6881

HOLE NO. BH 5-23

SAMPLE DESCRIPTION	Depth / Elevation
Ground Surface	EL 80.14 m
TOPSOIL	0.1 m EL 80.04 m
FILL: Brown silty sand, trace gravel	0.69 m EL 79.45 m
FILL: Brown silty clay with sand, gravel, some shale fragments, occasional cobbles and boulders	3.73 m EL 76.41 m
FILL: Brown silty sand with gravel, trace clay and shale fragments	4.9 m EL 75.24 m
BEDROCK: Poor quality, black shale - excellent quality by 6.2m depth	8.86 m EL 71.28 m
End of Borehole (Piezometer blocked - Nov. 15, 2023)	

Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1			20
SS2	42	33	15.8
SS3	12	3	18.5
SS4	42	2	19.2
SS5	33	3	37.6
SS6	50	14	14.3
RC1	83	44	
RC2	100	95	
RC3	100	98	

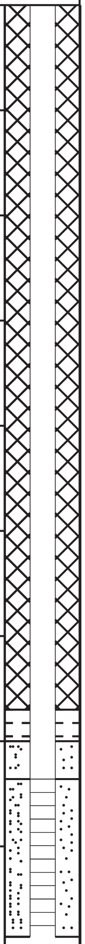
DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11

Remoulded Shear Strength (kPa): 0 25 50 75 100

Peak Shear Strength (kPa): 0 25 50 75 100

Pen. Resist. Blows/0.3m (50 mm Dia. Cone): 0 25 50 75 100

Piezometer Construction



SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1440 Blair Tower Place, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 374581.69 **NORTHING:** 5033155.076 **ELEVATION:** 80.26

PROJECT: Proposed Residential Development

BORINGS BY: CME Low Clearance Drill

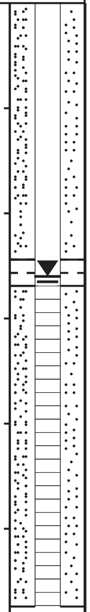
REMARKS:

DATE: November 9, 2023

FILE NO. PG6881

HOLE NO. BH 6-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 80.26 m						0				
TOPSOIL 0.25 m EL 80.01 m		AU1			20.1					
FILL: Brown silty sand with crushed stone, some to trace clay 1.68 m EL 78.58 m		SS2	42	8	13.9	1				
FILL: Brown silty clay with sand 2.44 m EL 77.82 m		SS3	8	3	22.4	2				
GLACIAL TILL: Dense, brown silty sand with gravel and shale fragments, occasional cobbles 3.2 m EL 77.06 m		SS4	79	38	29.5					
		SS5	75	50+	7.4	3				
		RC1	72	40		4				
BEDROCK: Poor to fair quality, black shale - excellent quality by 6.2m depth		RC2	100	57		5				
		RC3	100	95		6 7				
9.17 m EL 71.09 m		RC4	100	100		8 9				
End of Borehole (GWL @ 2.60m - Nov. 15, 2023)						10 11				



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

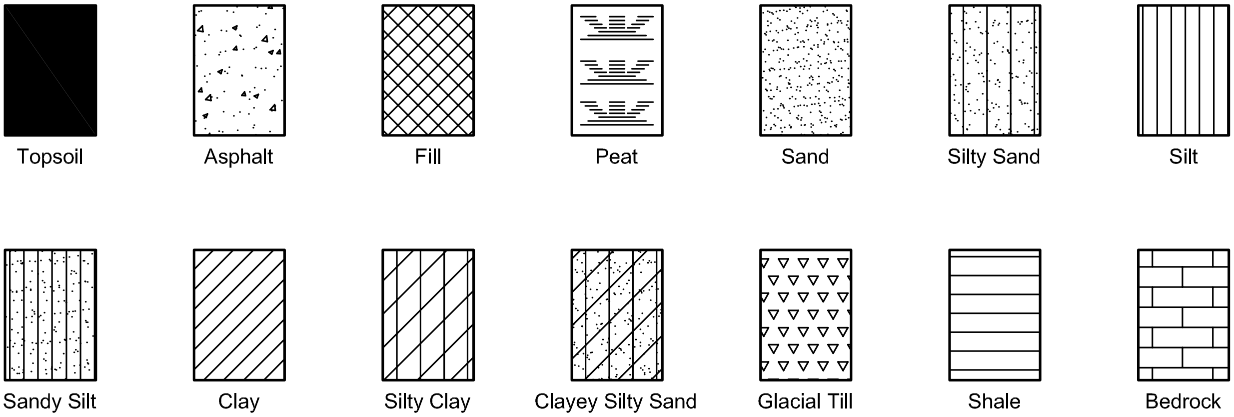
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

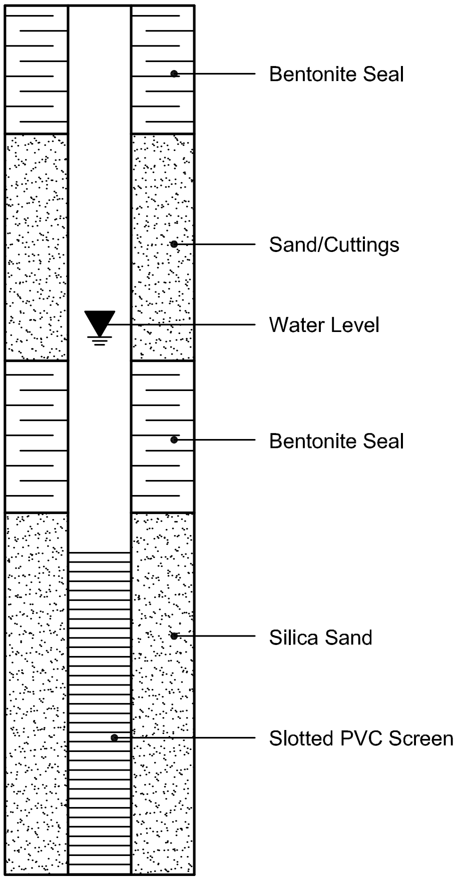
SYMBOLS AND TERMS (continued)

STRATA PLOT

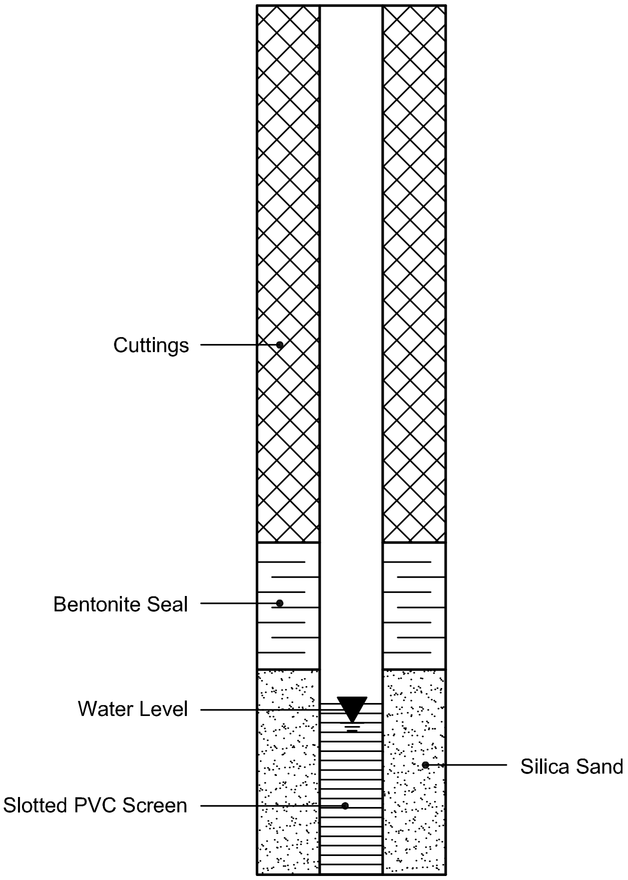


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 14-Nov-2023

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 9-Nov-2023

Client PO: 58797

Project Description: PG6881

Client ID:		BH3-23 SS3	-	-	-		
Sample Date:		08-Nov-23 09:00	-	-	-	-	-
Sample ID:		2345427-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	83.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.43	-	-	-	-	-
Resistivity	0.1 Ohm.m	24.0	-	-	-	-	-
Anions							
Chloride	10 ug/g	152	-	-	-	-	-
Sulphate	10 ug/g	47	-	-	-	-	-

Log of Borehole: BH1

Project #: 299657.002 **Logged By:** MK

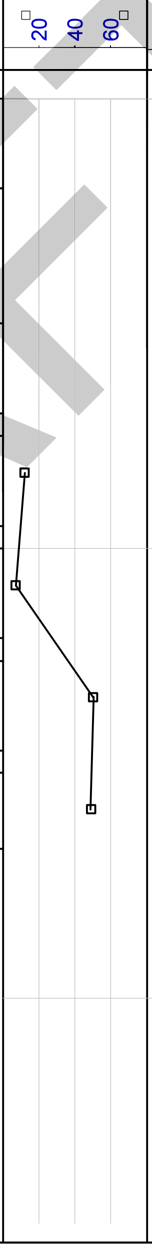
Project: Preliminary Geotechnical Investigation

Client: Crown Realty Properties Inc.

Location: 1400 - 1430 Blair Place, Ottawa, ON

Drill Date: December 8, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value ▫ 20 40 60 ▫	Shear Strength ▵ kPa ▵ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.15											
0		**Organics** ~ 150 mm		No Monitoring Well Installed	AS1	1	100	N/A					0/0	
1		**Glacial Till** Sandy silt, some gravel, some clay, damp, brown, loose to compact												
2			96.86		SS	2	20	N/A					0/0	
		Organics ~150 mm			SS	3	50	12					0/0	
3		**Glacial Till** Sandy silt, some gravel, some clay, grey, moist, loose to compact	95.34		SS	4	40	7			10.3		0/0	Hyd.
4		**Highly Weathered Shale**			SS	5	10	50					0/0	
5					SS	6	10	>50					0/0	PHC VOC PAH
6			93.05											
7		End of Borehole Borehole terminated at approximately 6.10 mbgs due to auger refusal on probable bedrock. At drilling completion, no free groundwater was encountered.												



Contractor: Marathon Drilling

Drilling Method: Hollow Stem Auger / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.15 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH2

Project #: 299657.002 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: Crown Realty Properties Inc.

Location: 1400 - 1430 Blair Place, Ottawa, ON

Drill Date: December 8, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	99.04
Organics ~ 150 mm	
Glacial Till Sand and silt, some gravel, trace clay, damp, brown, compact	
	97.36
Organics ~100 mm	
Glacial Till Sand and silt, some gravel, trace clay, damp, brown, loose to very dense	
	96.45
Highly Weathered Shale	
	94.47
End of Borehole	

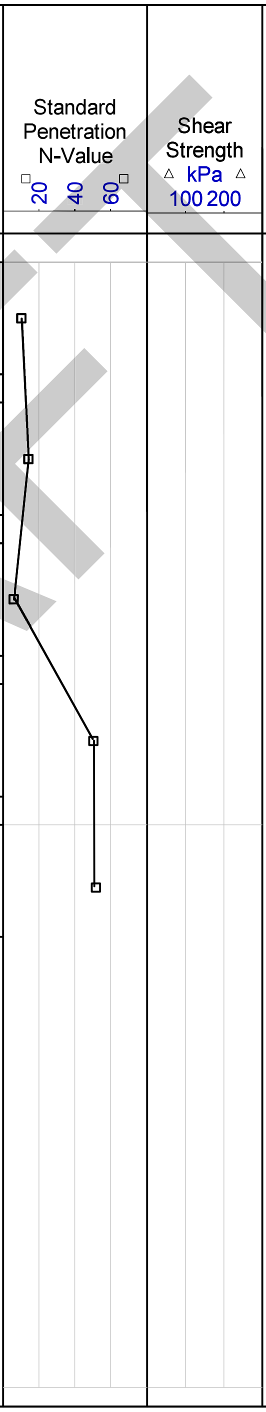
Borehole terminated at approximately 4.57 mbgs due to auger refusal on probable bedrock. At drilling completion, no free groundwater was encountered.

Monitoring Well Details: No Monitoring Well Installed

Depth (m): 0, 1, 2, 3, 4, 5, 6

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	50	10			0/0	
SS	2	60	14			0/0	pH
SS	3	60	6	15.4		0/0	Hyd.
SS	4	20	>50			0/0	pH
SS	5	10	>50			0/0	PHC VOC PAH



Contractor: Marathon Drilling **Grade Elevation:** 99.04 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH3

Project #: 299657.002 **Logged By:** MK

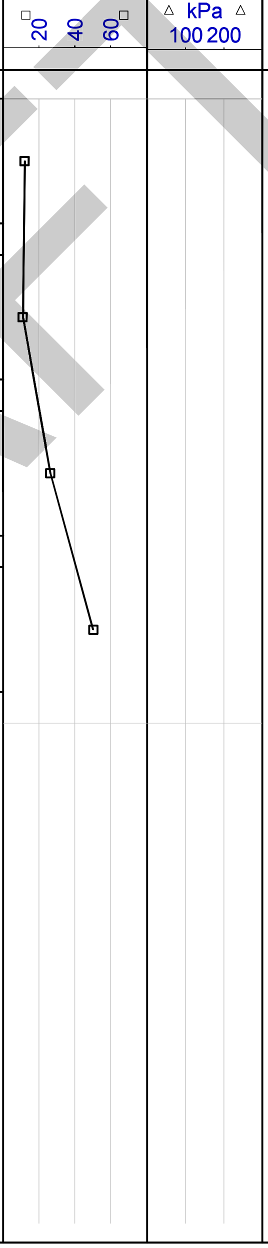
Project: Preliminary Geotechnical Investigation

Client: Crown Realty Properties Inc.

Location: 1400 - 1430 Blair Place, Ottawa, ON

Drill Date: December 8, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	98.57											
0		**Organics** ~ 150 mm		No Monitoring Well Installed										
		Glacial Till Sand and silt, some gravel, trace clay, damp to moist, brown, compact			SS	1	50	12					0/0	
1					SS	2	70	11					0/0	
2					SS	3	80	26					0/0	
		Highly Weathered Shale	96.28		SS	4	50	>50					0/0	PHC VOC PAH
3														
4		End of Borehole	94.76											
		Borehole terminated at approximately 3.81 mbgs due to auger refusal on probable bedrock. At drilling completion, no free groundwater was encountered.												
5														



Contractor: Marathon Drilling **Grade Elevation:** 98.57 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH4

Project #: 299657.002 **Logged By:** MK

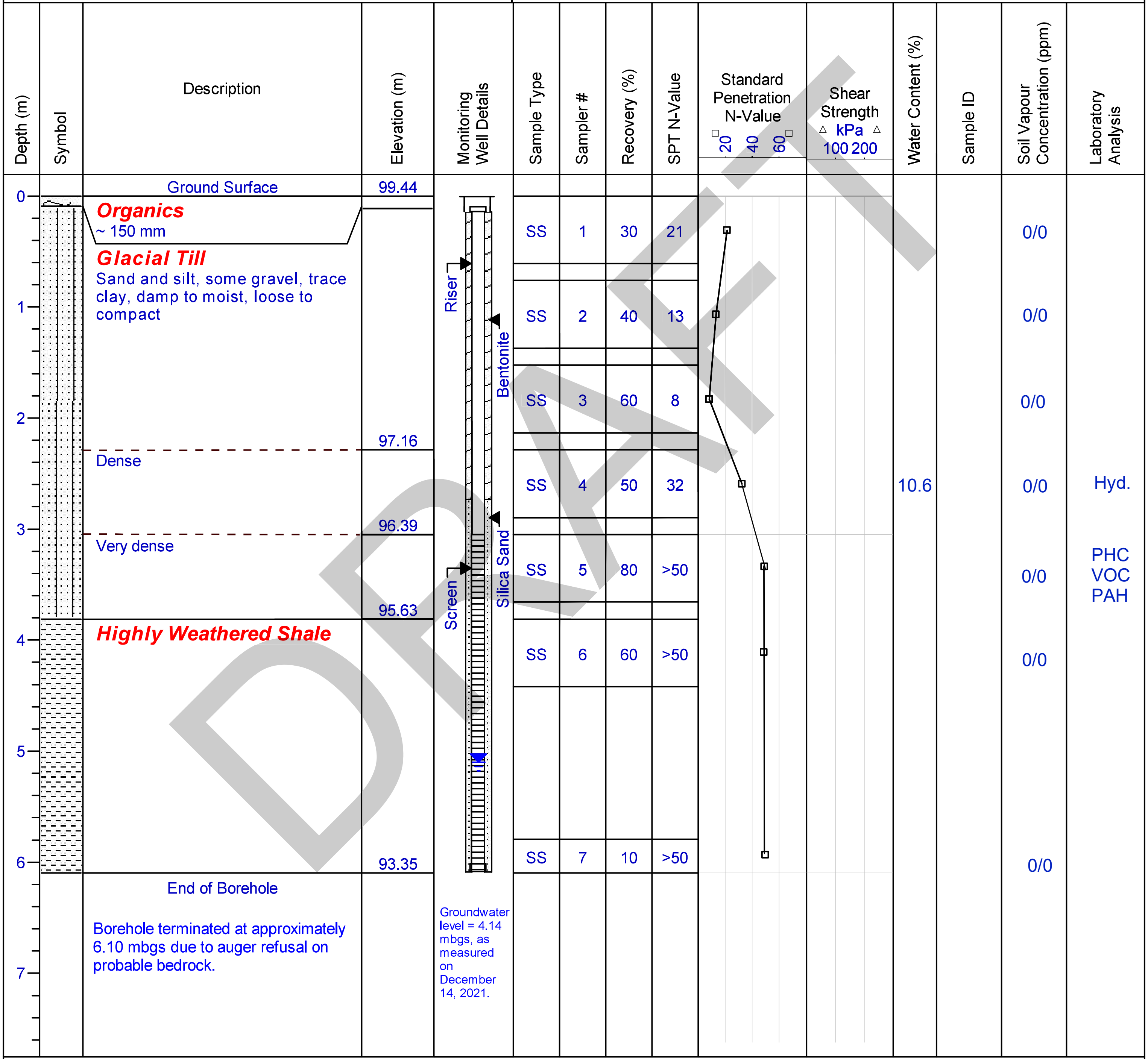
Project: Preliminary Geotechnical Investigation

Client: Crown Realty Properties Inc.

Location: 1400 - 1430 Blair Place, Ottawa, ON

Drill Date: December 9, 2021 **Project Manager:** WT

SUBSURFACE PROFILE | **SAMPLE**



Description	Elevation (m)
Ground Surface	99.44
Organics ~ 150 mm	
Glacial Till Sand and silt, some gravel, trace clay, damp to moist, loose to compact	
Dense	97.16
Very dense	96.39
Highly Weathered Shale	95.63
End of Borehole	93.35

Borehole terminated at approximately 6.10 mbgs due to auger refusal on probable bedrock.

Groundwater level = 4.14 mbgs, as measured on December 14, 2021.

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	30	21			0/0	
SS	2	40	13			0/0	
SS	3	60	8			0/0	
SS	4	50	32	10.6		0/0	Hyd.
SS	5	80	>50			0/0	PHC VOC PAH
SS	6	60	>50			0/0	
SS	7	10	>50			0/0	

Contractor: Marathon Drilling **Grade Elevation:** 99.44 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** 100.40 m

Well Casing Size: 51 mm **Sheet:** 1 of 1

Log of Borehole: BH5

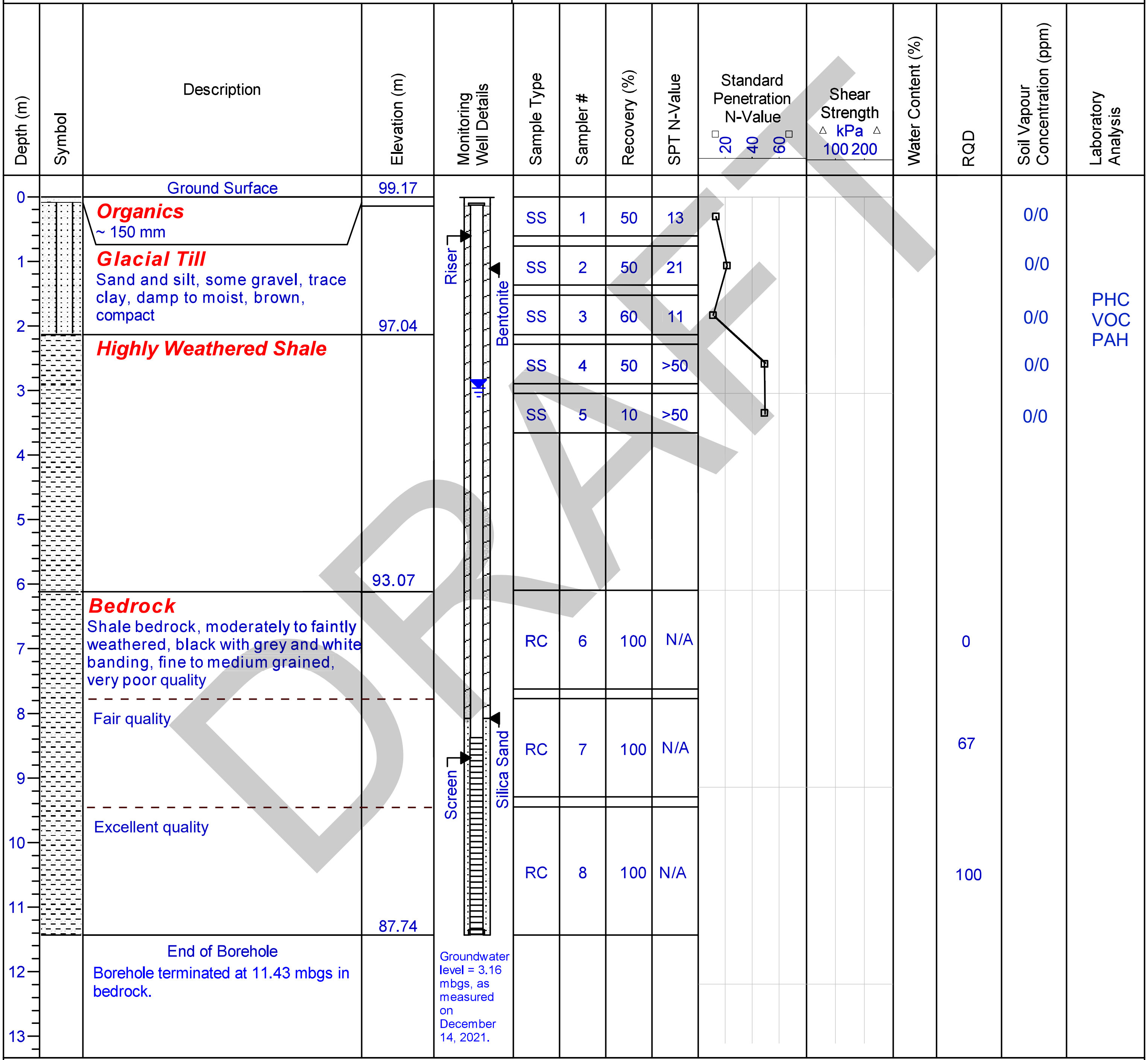
Project #: 299657.002 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: Crown Realty Properties Inc.

Location: 1400 - 1430 Blair Place, Ottawa, ON

Drill Date: December 9, 2021 **Project Manager:** WT



SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	99.17
Organics ~ 150 mm	
Glacial Till Sand and silt, some gravel, trace clay, damp to moist, brown, compact	97.04
Highly Weathered Shale	93.07
Bedrock Shale bedrock, moderately to faintly weathered, black with grey and white banding, fine to medium grained, very poor quality	
Fair quality	
Excellent quality	87.74
End of Borehole Borehole terminated at 11.43 mbgs in bedrock.	

Groundwater level = 3.16 mbgs, as measured on December 14, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	RQD	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	50	13			0/0	
SS	2	50	21			0/0	
SS	3	60	11			0/0	PHC VOC PAH
SS	4	50	>50			0/0	
SS	5	10	>50			0/0	
RC	6	100	N/A		0		
RC	7	100	N/A		67		
RC	8	100	N/A		100		

Contractor: Marathon Drilling **Grade Elevation:** 99.17 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** 100.10 m

Well Casing Size: 51 mm **Sheet:** 1 of 1

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6881-1 - TEST HOLE LOCATION PLAN

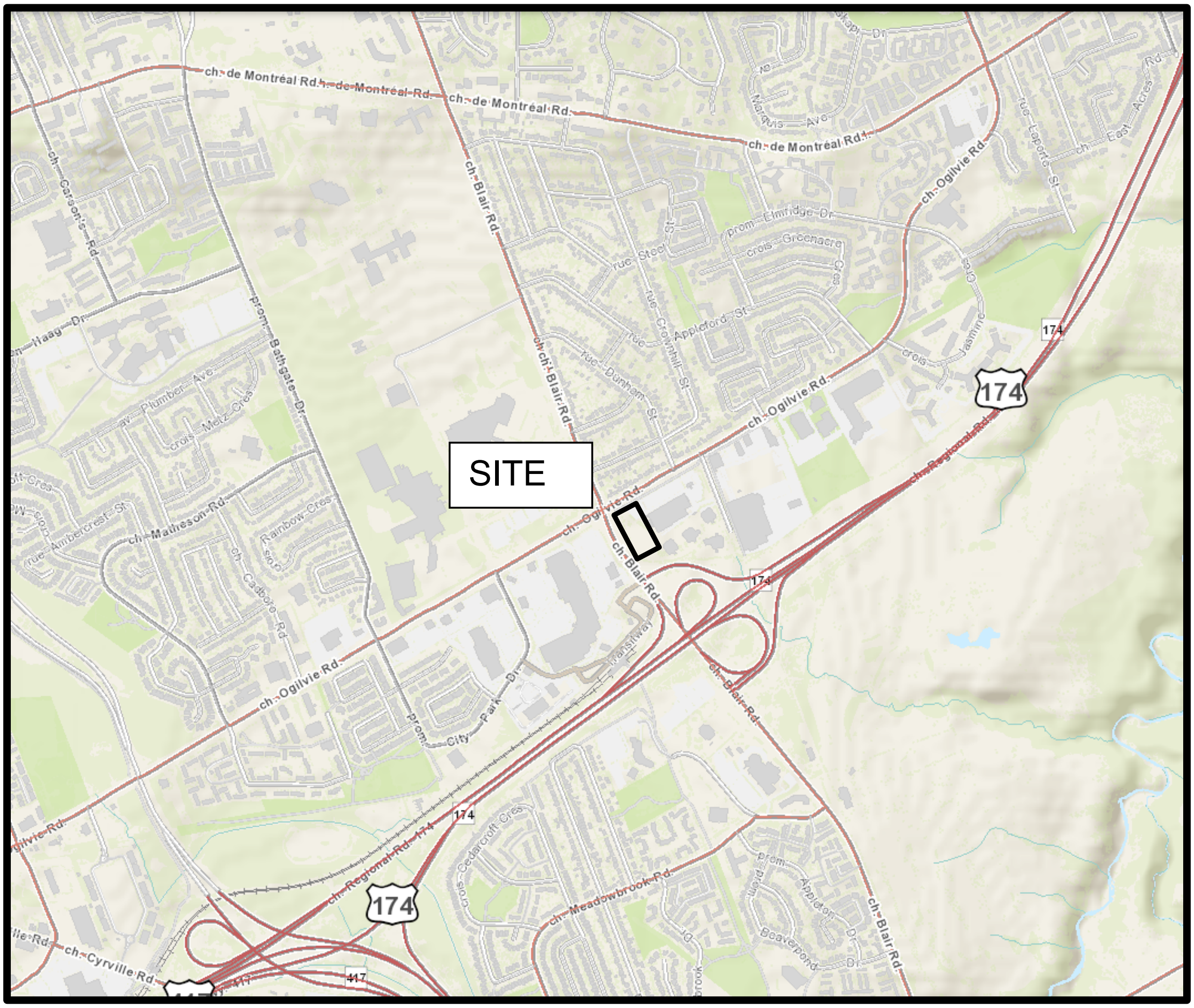
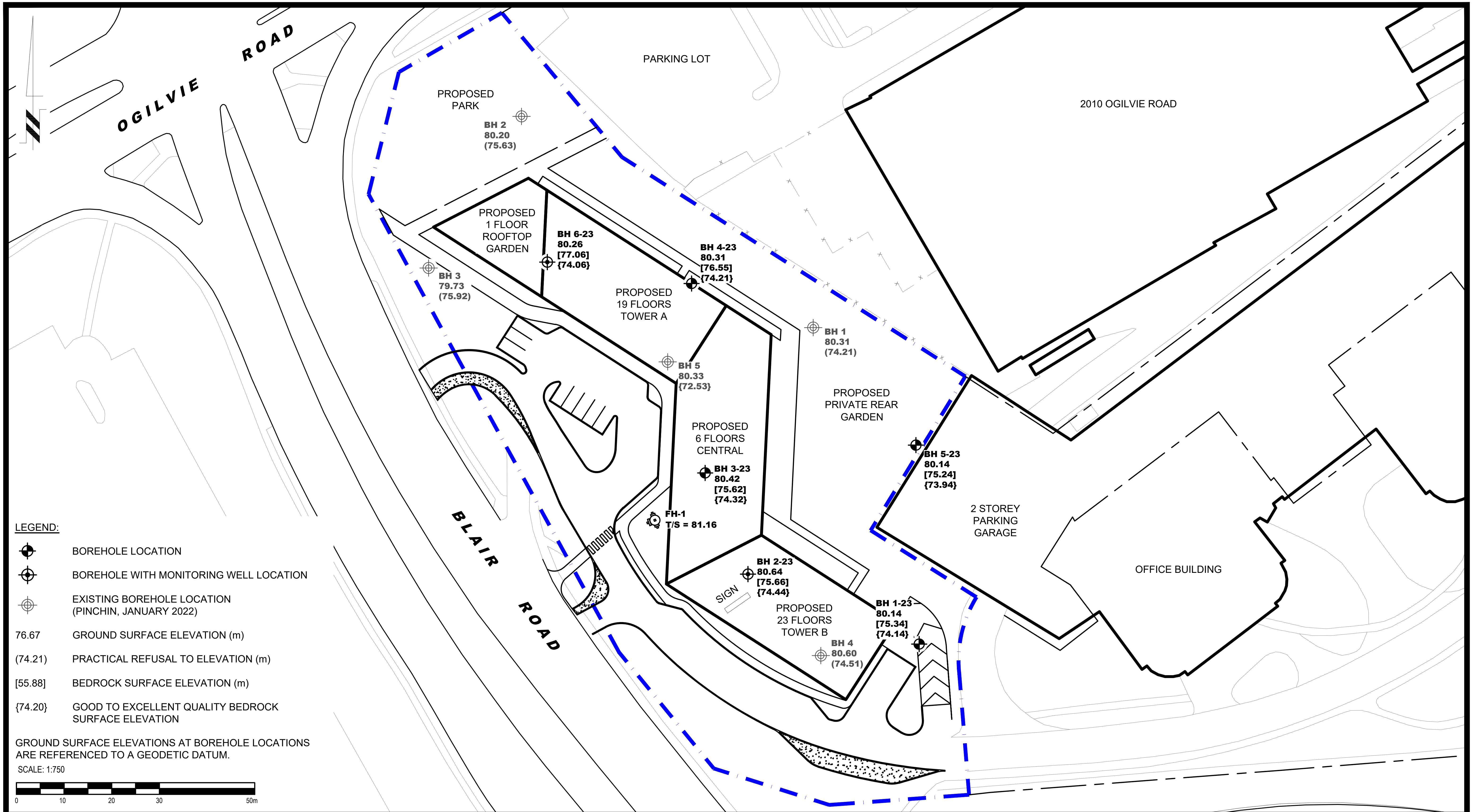


FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⊕ BOREHOLE LOCATION

⊕ BOREHOLE WITH MONITORING WELL LOCATION

⊕ EXISTING BOREHOLE LOCATION
(PINCHIN, JANUARY 2022)

76.67 GROUND SURFACE ELEVATION (m)

(74.21) PRACTICAL REFUSAL TO ELEVATION (m)

[55.88] BEDROCK SURFACE ELEVATION (m)

{74.20} GOOD TO EXCELLENT QUALITY BEDROCK SURFACE ELEVATION

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

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

LE GROUPE MAURICE
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1440 BLAIR TOWER PLACE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750

Date: 11/2023

Drawn by: ZS

Report No.: PG6881-1 R1

Checked by: NS

Dwg. No.: PG6881-1

Approved by: JV

Revision No.:

11x17

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