PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

1815 Montreal Road
Ottawa, Ontario

Creative Development Ventures

Report PG6594-1 dated May 16, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Creative Development Ventures to conduct a geotechnical investigation for the proposed development to be located at 1815 Montreal Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with 2 levels of underground parking.

Landscaped margins are expected at finished grades surrounding the proposed building. The subject site is expected to be municipally serviced.

It is anticipated that the existing building on-site is to be demolished to allow for construction of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on April 20 and 21, 2023, and consisted of advancing a total of 4 boreholes and 1 probehole to a maximum depth of 11.2 m below the existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The approximate locations of the test holes are shown on Drawing PG6594-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split- spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm diameter coring equipment. All samples were visually inspected and initially classified on site, and subsequently placed in sealed plastic bags or core boxes, then transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC respectively, on the Soil Profile and Test Data Sheets in Appendix 1.

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

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

Bedrock samples were recovered from boreholes BH 1-23 and BH 2-23 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock, which are shown on the Soil Profile and Test Data sheets. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site, taking into consideration the existing site features and underground utilities. The test hole locations, and the ground surface elevation at each test hole location, were surveyed by Paterson using a handheld GPS unit with respect to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG6594-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for 1 month after this report is completed. They 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 sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The southern half of the subject site is currently occupied by an existing 1-storey residential building. A retaining wall is located to the east of the existing residential building. The remainder of the site is generally undeveloped, with mature trees present throughout most of the site.

The site is bordered by residential dwellings to the east and west, Rothwell Drive to the north, and by Montreal Road to the south. The ground surface across the subject site gently slopes downward from south to north from an approximate geodetic elevation of 95 m to 90 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil underlain by an approximate 0.7 to 1.2 m thickness of fill material which extends to a maximum depth of 1.5 m below the existing ground surface. The fill was generally observed to consist of brown silty sand to sandy silt with trace clay and gravel. At boreholes BH 3-23 and BH 4-23, the fill was observed to transition to a brown silty clay to clayey silt.

A hard to very stiff, brown silty clay was encountered underlying the fill layer at boreholes BH 3-23 and BH 4-23 within the northern portion of the site, and was observed to transition to a stiff to firm, grey silty clay at approximate depths of 5.5 and 5.2 m, respectively. The silty clay deposit extended to approximate depths of 7.3 and 7.4 m at boreholes BH 3-23 and BH 4-23, respectively.

A glacial till deposit was observed underlying the fill and/or silty clay at all boreholes with the exception of borehole BH 1-23, which was located at the southern end of the site. The glacial till deposit was encountered underlying the fill layer in boreholes BH 2-23, and extended to an approximate depth of 1.7 m. At boreholes BH 3-23 and BH 4-23, the glacial till deposit was observed underlying the silty clay deposit, extending to approximate depths of 10.4 m and 9.1 m, respectively. The glacial till deposit was generally observed to consist of a compact to dense, grey silty clay and/or grey silty sand with gravel and rock fragments.

Bedrock

Practical refusal to augering was encountered on the bedrock surface at approximate depths of 1.4 and 1.7 m at boreholes BH 1-23 and BH 2-23 within the southern portion of the site. Further, practical refusal to the DCPT was encountered at an approximate depth of 1.2 m at probehole PH 1-23, located within the southern portion of the site. The bedrock was cored at boreholes BH 1-23 and BH 2-23 and, based on the recovered rock core, was observed to consist of good to excellent quality, grey limestone. The bedrock was cored to a maximum depth of about 11.2 m below the existing ground surface.

Practical refusal to augering was encountered at an approximate depth of 9.1 m below the existing ground surface at borehole BH 4-23, and practical refusal to the DCPT was encountered at an approximate depth of 12.1 m at borehole borehole BH 3-23.

Based on available geological mapping, the bedrock in the subject area consists of interbedded limestone and dolomite of the Gull River Formation.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

4.3 Groundwater

The groundwater level was measured at the piezometer on May 4, 2023. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	95.15	3.56	91.59	May 4, 2023
BH 2-22	93.82	2.00	91.82	May 4, 2023
BH 3-22	91.68	0.49	91.19	May 4, 2023
BH 4-22	90.13	0.72	89.41	May 4, 2023

Note: Ground surface elevations at borehole locations are referenced to a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater levels are expected to range between approximate geodetic elevations 89.5 to 91.5. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is expected that the proposed building will be founded on conventional spread footings placed on a combination of clean, surface sounded limestone bedrock and undisturbed, compact to dense glacial till.

It is expected that bedrock removal will be required within the southern portion of the development to complete the underground parking levels.

Due to the presence of a silty clay deposit throughout the northern portion of the subject site, a permissible grade raise restriction has been provided. This is discussed further in Section 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with 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 the proximity of the blasting operations should be carried out 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 or claims related to the blasting operations.

As a general guideline, peak particle velocity (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 must be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels 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 a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations 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).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

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

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

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 300 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.

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. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

Lean Concrete Filled Trenches

Where footings are designed to be supported on undisturbed, compact to dense glacial till or clean, surface sounded bedrock which is encountered below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength).

Typically, the excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of excavation. 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 undisturbed, compact to dense glacial till or clean, surface sounded bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Footings placed on an undisturbed, compact to dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings supported directly on clean, surface sounded limestone bedrock can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **2,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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 supported directly on clean, surface sounded bedrock, and designed for the bearing resistance values provided above, will be subject to negligible post-construction total and differential settlements.

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A soil bearing medium, or a heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or shallower).

Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended at the soil/bedrock and bedrock/soil transitions that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material, see Figure 1 below. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. 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 footings and foundation walls.

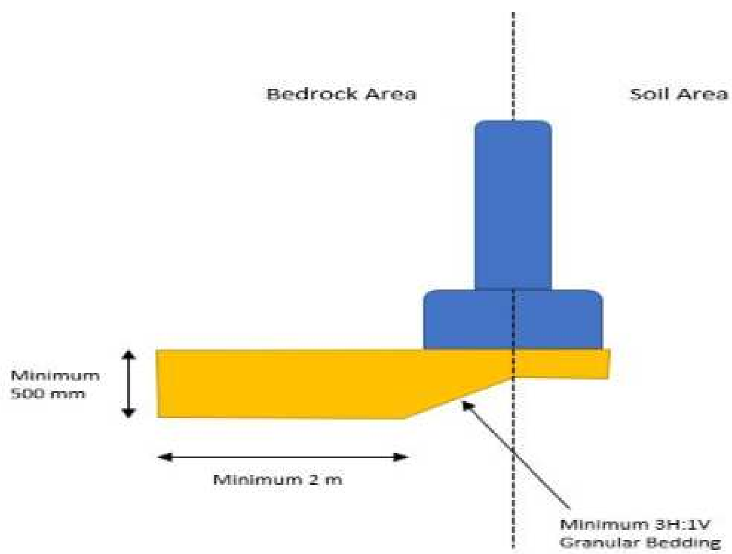

Figure 1 – Bedrock/Soil Transition Treatment

Permissible Grade Restriction

Due to the presence of the silty clay deposit at the site, a permissible grade raise restriction of **2 m** is recommended for grading at the subjected site.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class (Class A or B) is required for the proposed building, and the proposed footings are to be located within 3 m of the bedrock surface, a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

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

It is anticipated that the underground levels for the proposed building will be mostly parking and the recommended pavement structures noted in Section 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing 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. 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.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. 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 drained unit weight of 20 kN/m^3 (effective unit weight 13 kN/m^3).

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).
The seismic earth force (ΔP) can be calculated using $0.375 \cdot a \cdot H^2/g$ 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 $P_o = .5\ K_o\ \gamma\ H^2$, where K = 0.5 for the soil conditions noted above.

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

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

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

5.7 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed building 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 2 below. The flexible pavement structure presented in Tables 3 and 4 should be used for car only parking areas, at grade access lanes and heavy loading parking areas.

Table 2 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
125	**Exposure Class C2 – 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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 lower 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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 3 – Recommended Asphalt Pavement Structure – Car only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** – Superpave 12.5 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 4 – Recommended Asphalt Pavement Structure – Access Lanes and Heavy Loading Parking Areas	
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
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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

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

The pavement granular (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable compaction equipment.

5.8 Rock Anchor Design

Overview of Anchor Features

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 a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed building, if required, any rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 5 on the next page.

Table 5 - Parameters used in Rock Anchor Design	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^{o}
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 4 below.

The factored tensile resistance values given in Table 6 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 6 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel, and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes.

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 that grout is prepared.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

It is recommended that the portion of the proposed building foundation walls located below the long-term groundwater table (approximately geodetic elevation 90 m) be placed against a groundwater infiltration control system which is fastened to the temporary shoring system or vertical bedrock face. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the portion of the groundwater infiltration control system installed against the vertical bedrock face, the following is recommended:

- ❑ Line drill the excavation perimeter.
- ❑ Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
- ❑ Place a suitable membrane against the prepared bedrock surface, such as a Tremco Paraseal or an approved equivalent. The membrane liner should extend from geodetic elevation 90 m, down to the footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- ❑ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- ❑ Pour foundation wall against the composite drainage system and waterproofing membrane.

It is recommended that 100 mm diameter sleeves be cast at 3 m centres at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

A waterproofing system should also be provided for any elevator pits (pit bottom and walls). Detailed designs can be provided for the overall recommended waterproofing system, if required.

Underslab Drainage System

Underslab drainage will be required to control water infiltration for the underground parking levels. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls, where required, should consist of free-draining, non-frost susceptible granular materials such as clean sand or OPSS Granular B Type I material. The greater part of the site excavated materials will be relatively 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 1, Granular A or Granular B Type II 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

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation should be provided in this regard.

Other exterior unheated footings, such as isolated 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, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

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 soil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Bedrock Stabilization

Where required, 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.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for temporary rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

Temporary Shoring

Due to the expected depth of excavation to accommodate the underground parking and the proximity of the proposed multi-storey building to the site boundaries, temporary shoring may be required to support the overburden soils. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event 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 designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 7 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m^3	21
Submerged Unit Weight , kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater table.

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 should be calculated full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. 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 98% of the SPMDD.

It is generally possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water 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 Persons 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.

Impacts on Neighbouring Properties

It is understood that 2 levels of underground parking are planned for the proposed building with the lower portion of the foundation walls having a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place, long-term groundwater lowering is anticipated to be negligible for the area. Therefore, no adverse effects to neighbouring properties are expected.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing from the neighbouring area show that the sulphate content is less than 0.1%. These results along with the result chloride content and the pH are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the resistivity indicate the presence of a moderate to aggressive corrosive environment.

6.8 Slope Stability Assessment

Paterson completed a review of the slope at the subject site at the time of the geotechnical field investigation. The existing slope face was observed to be vegetated with grass and mature trees with an approximate 15H:1V slope profile. No signs of active or historic erosion were observed along the face of the slope.

Based on the available drawings it is understood that the proposed development will consist of a multi-storey building with 2 levels of underground parking. It is anticipated that grading across the majority of the site will be adjusted in conjunction with construction of the proposed building.

As a result of the observations made during the geotechnical investigation, which include the gentle slope gradient, the presence of shallow bedrock within the southern portion of the site, as well as the lack of a watercourse at the toe of slope, it is anticipated that the stability of the slope is well in excess of a factor of safety 1.5 under static conditions, and well in excess of a factor of safety of 1.1 under seismic conditions. As such, the subject slope is considered stable from a geotechnical perspective, and construction of the proposed multi-storey building will not impact the slope stability.

Due to the slope stability, a stable slope allowance is not considered to be required. Additionally, as a watercourse is not located along the toe of slope, a toe erosion allowance is not considered to be required. As neither a stable slope allowance nor a toe erosion allowance is applicable, an erosion access allowance along the top of slope is not considered to be required.

Accordingly, a Limit of Hazard Lands setback from the top of slope is not required, from a geotechnical perspective.

6.9 Tree Planting Restrictions

As the proposed building will include 2 underground levels, the foundation will be located well below the depth of influence of tree roots. Accordingly, there are no required tree planting setbacks for the proposed development, from a geotechnical perspective.

7.0 Recommendations

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

- ❑ Review of the foundation plan, from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's temporary shoring design, if required, prior to construction.
- ❑ Review of the proposed groundwater infiltration control system and requirements.
- ❑ Review of the bedrock stabilization and excavation requirements.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

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

8.0 Statement of Limitations

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Creative Development Ventures 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.

Kevin A. Pickard, P.Eng.

Scott S. Dennis, P.Eng

Report Distribution:

- ❑ Creative Development Ventures (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1815 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 20, 2023

FILE NO. PG6594

HOLE NO. BH 1-23

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface				
TOPSOIL (to 0.69)	AU	1		
FILL: Brown silty sand to sandy silt, trace gravel (to 1.40)	SS	2	50	12
BEDROCK: Poor to good quality, grey limestone	RC	1	100	42
	RC	2	100	85
- excellent quality by 3.6m depth	RC	3	100	100
	RC	4	100	100
	RC	5	100	95
	RC	6	100	97
(to 11.20)	RC	7	100	100

End of Borehole

(GWL @ 3.56m - May 4, 2023)

DEPTH (m)	ELEV. (m)
0	95.15
1	94.15
2	93.15
3	92.15
4	91.15
5	90.15
6	89.15
7	88.15
8	87.15
9	86.15
10	85.15
11	84.15

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1815 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 20, 2023

FILE NO. PG6594

HOLE NO. BH 2-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Water Content %
Ground Surface						0	93.82	
TOPSOIL	0.18							
FILL: Brown silty sand to sandy silt	0.60	SS	1	50	2			
FILL: Brown silty sand, trace gravel	1.22	SS	2	42	8	1	92.82	
GLACIAL TILL: Dense, brown silty sand with gravel	1.73	SS	3	84	34			
BEDROCK: Good to excellent quality, grey limestone	9.73	RC	1	100	88	2	91.82	
		RC	2	100	92	3	90.82	
		RC	3	100	85	4	89.82	
		RC	4	100	100	5	88.82	
		RC	5	100	100	6	87.82	
		RC	6	100	98	7	86.82	
						8	85.82	
						9	84.82	

End of Borehole

(GWL @ 2.00m - May 4, 2023)

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1815 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger **DATE** April 21, 2023

FILE NO. PG6594

HOLE NO. BH 3-23

SOIL DESCRIPTION	Layer bottom depth (m)
Ground Surface	
TOPSOIL	0.30
FILL: Brown silty sand to sandy silt	0.91
FILL: Brown silty clay to clayey silt	1.22
Hard to very stiff, brown **SILTY CLAY** - trace shells by 4.9m depth - firm and grey by 5.5m depth	7.32
GLACIAL TILL: Grey silty clay with shells, trace gravel	8.46
GLACIAL TILL: Grey silty sand with gravel, trace clay	10.36
Dynamic Cone Penetration Test commenced at 10.36m depth.	12.06

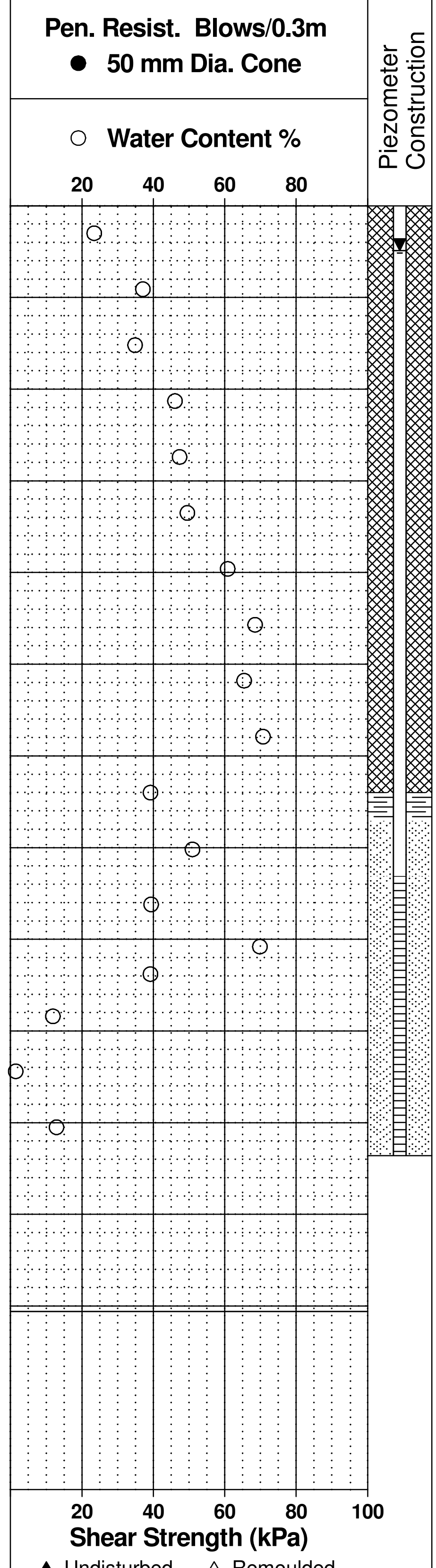
End of Borehole

Practical DCPT refusal at 12.06m depth

(GWL @ 0.49m - May 4, 2023)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	50	4
SS	2	75	19
SS	3	100	28
SS	4	100	33
SS	5	100	27
SS	6	100	21
SS	7	100	14
SS	8	100	9
SS	8	100	5
SS	9	42	1
SS	10	100	1
SS	11	100	1
SS	12	100	P
SS	13	83	2
SS	14	50	33
SS	15	17	9
SS	16	50	3

DEPTH (m)	ELEV. (m)
0	91.68
1	90.68
2	89.68
3	88.68
4	87.68
5	86.68
6	85.68
7	84.68
8	83.68
9	82.68
10	81.68
11	80.68
12	79.68



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1815 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 21, 2023

FILE NO. PG6594

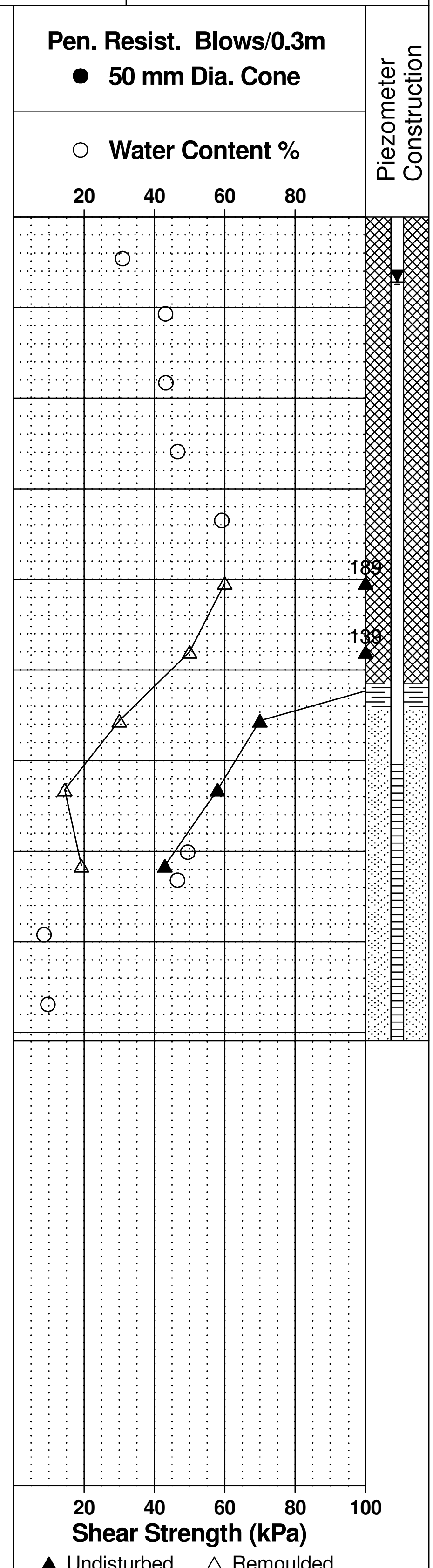
HOLE NO. BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	90.13
TOPSOIL 0.25							
FILL: Brown silty sand to sandy silt, trace clay 0.69		AU	1				
FILL: Brown silty clay 1.45		SS	2	75	5	1	89.13
Hard to very stiff, brown **SILTY CLAY**		SS	3	100	15	2	88.13
		SS	4	100	11	3	87.13
		SS	5	100	7	4	86.13
- stiff to firm and grey by 5.2m depth						5	85.13
						6	84.13
- with shells by 6.9m depth 7.39		SS	6	83	P	7	83.13
GLACIAL TILL: Dense to compact, grey silty sand with gravel, trace clay and rock fragments		SS	7	21	34	8	82.13
9.09		SS	8	25	18	9	81.13

End of Borehole

Practical refusal to augering at 9.09m depth.

(GWL @ 0.72m - May 4, 2023)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1815 Montreal Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

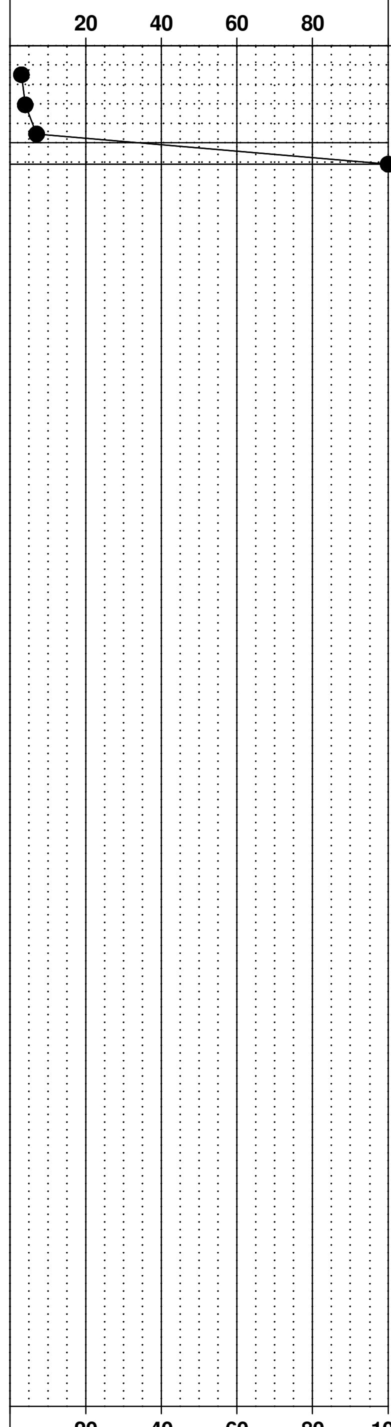
BORINGS BY Track-Mount Power Auger

DATE April 21, 2023

FILE NO. PG6594

HOLE NO. PH 1-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
Ground Surface						0	94.39		
Dynamic Cone Penetration Test commenced at ground surface.						1	93.39		
1.22									
End of Probehole Practical DCPT refusal at 1.22m depth.									



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 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

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

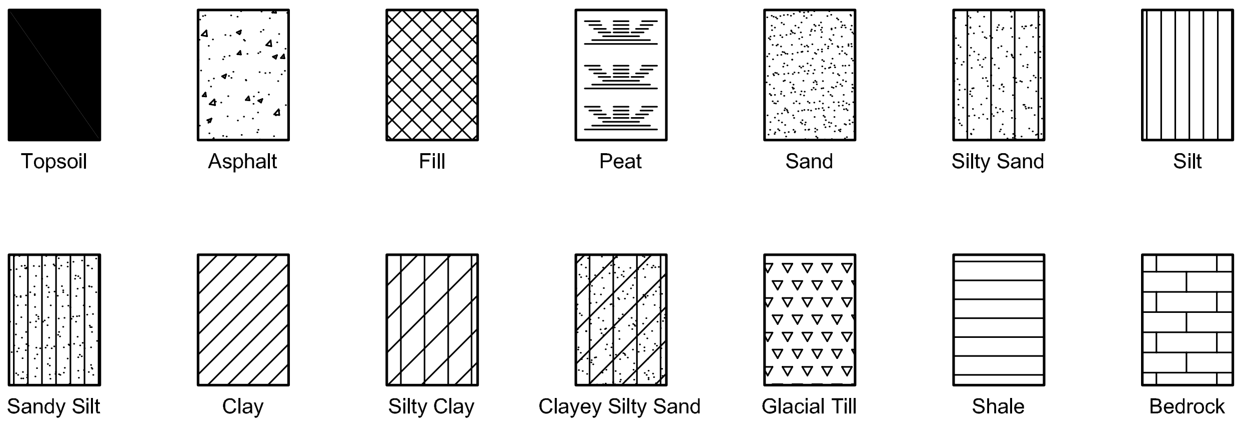
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

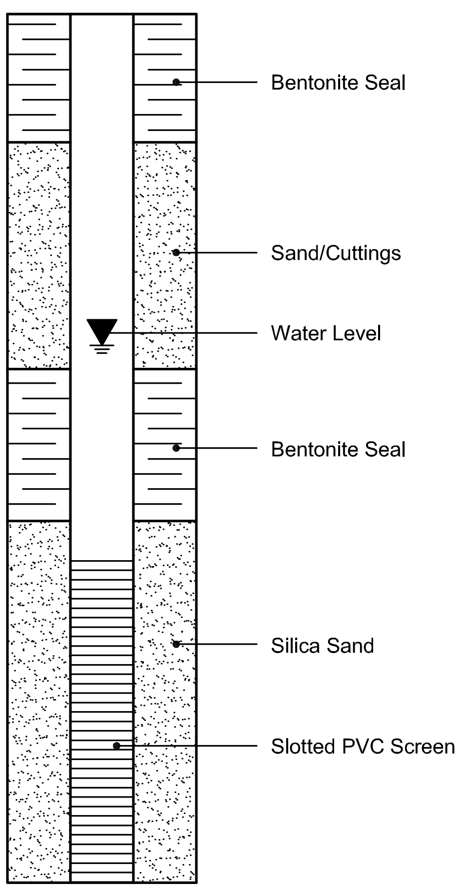
SYMBOLS AND TERMS (continued)

STRATA PLOT

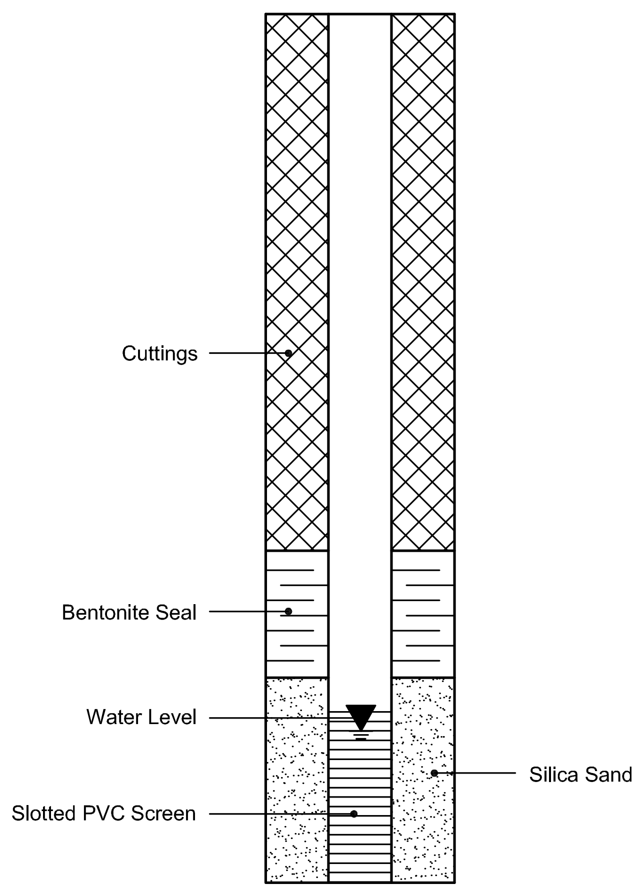


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 57322

Report Date: 27-Apr-2023

Order Date: 24-Apr-2023

Project Description: PG6594

	Client ID:	BH4-23 SS3	-	-	-
	Sample Date:	21-Apr-23 09:00	-	-	-
	Sample ID:	2317108-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	69.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.06	-	-	-
Resistivity	0.1 Ohm.m	30.4	-	-	-
Anions					
Chloride	10 ug/g dry	61	-	-	-
Sulphate	10 ug/g dry	179	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6594-1 – TEST HOLE LOCATION PLAN

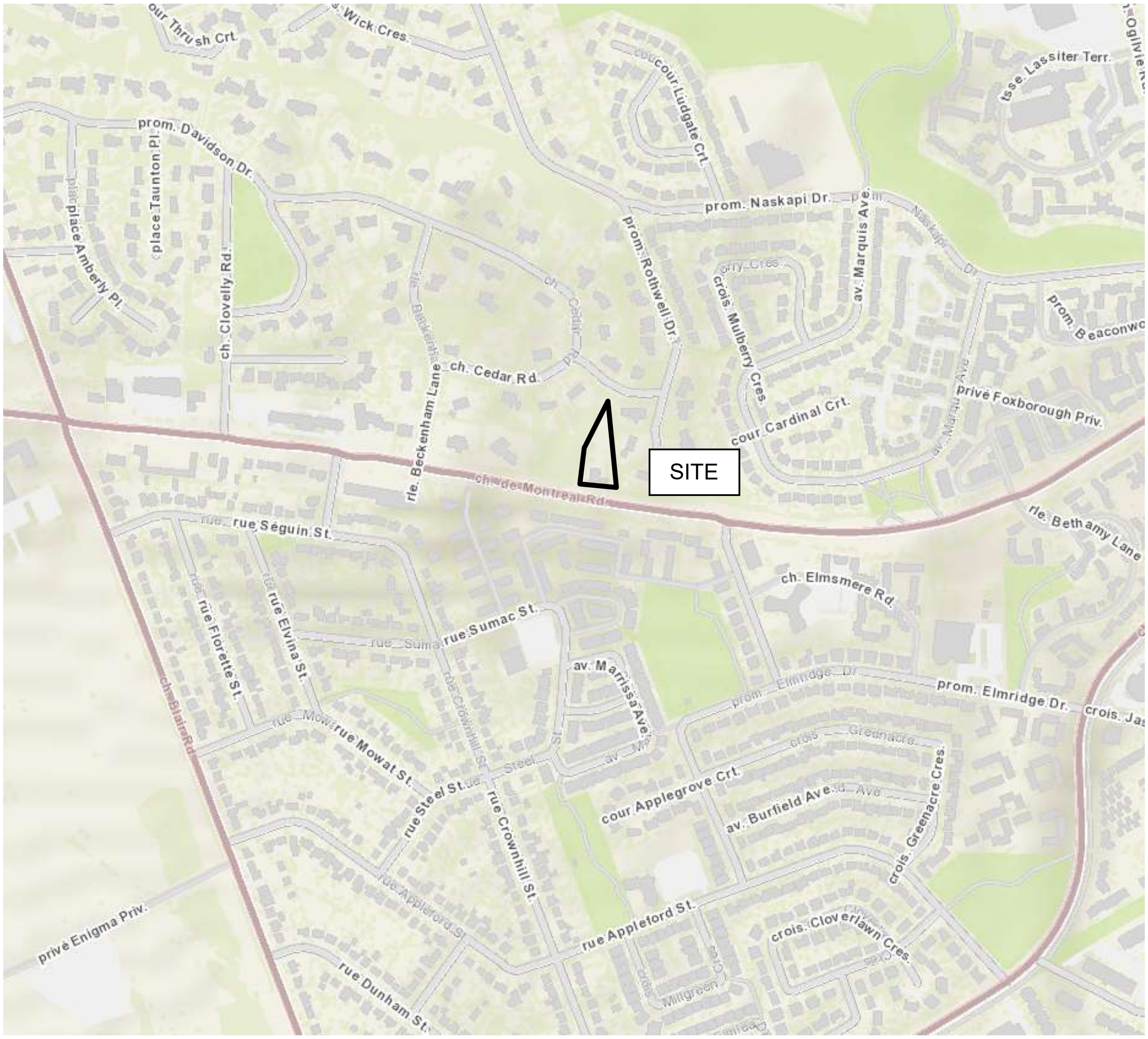
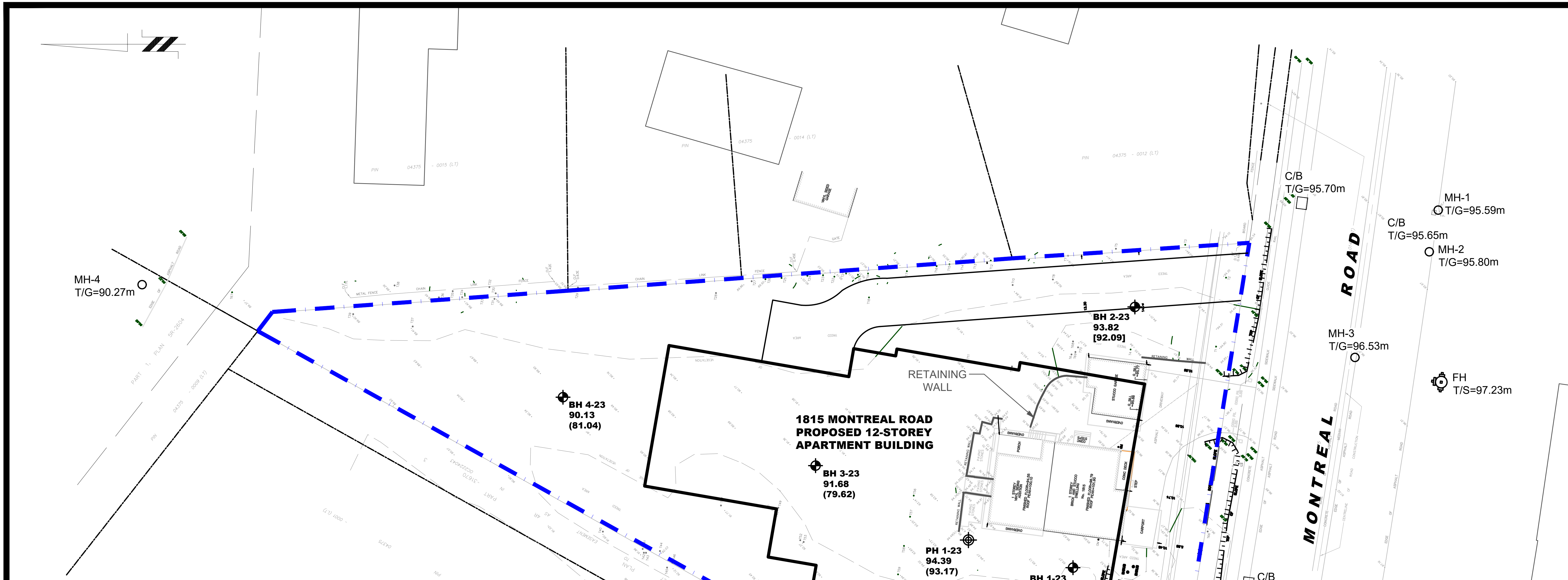


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

BOREHOLE LOCATION

PROBEHOLE LOCATION

95.15 GROUND SURFACE ELEVATION (m)

[93.75] BEDROCK SURFACE ELEVATION (m)

(79.62) PRACTICAL DCPT REFUSAL ELEVATION (m)

BASE PLAN PROVIDED BY J.D. BARNES

CONCEPTUAL PLAN PROVIDED BY CSV ARCHITECTS

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

SCALE: 1:500

0 5 10 15 20 25 30m

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

NO.	REVISIONS	DATE	INITIAL

CREATIVE DEVELOPMENT VENTURES

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
1815 MONTREAL ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 04/2023
Drawn by: YA	Report No.: PG6594-1
Checked by: MF	Dwg. No.: PG6594-1
Approved by: SD	Revision No.:

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11x17