PATERSON GROUP

Geotechnical Investigation

Proposed High-Rise Development

1994 Scott Street - Ottawa, Ontario

Prepared for Park River Properties

Report PG6991-1 dated February 28, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Park River Properties to conduct a geotechnical investigation for the proposed development to be located at 1994 Scott Street (and surrounding properties) in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 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 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 a part of the scope of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available site plan, it is understood that the proposed development at the subject site will consist of 4 high-rise buildings. It is anticipated that each building will have 2 to 3 levels of underground parking.

Further, associated at-grade access lanes, as well as amenity and landscaped areas, are anticipated immediately around the proposed buildings. It is also expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted between February 7th and 13th 2024, and consisted of advancing a total of 9 boreholes to a maximum depth of 7.7 m below existing ground surface.

The borehole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The borehole locations are presented on Drawing PG6991-1 – Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. Rock cores were obtained using 47.6 mm inside diameter coring equipment. All soil samples were visually inspected and initially classified on site. The auger, grab and split-spoon samples were placed in sealed plastic bags. Rock cores were placed in cardboard boxes.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger, grab, split spoon, and rock core samples were recovered from the boreholes are shown as AU, G, SS, and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

Groundwater

Groundwater monitoring wells were installed in boreholes BH 2-24, BH 5-24, BH 6-24, BH 7-24 and BH 9-24, to permit long-term groundwater measurement subsequent to the field investigation. Flexible polyethylene standpipes were installed in the remaining boreholes to permit further groundwater measurement.

The groundwater observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 1.5 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No. 3 silica sand backfill within annular space around screen.
- 300 mm thick bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The borehole locations, and ground surface elevation at each borehole location, were surveyed by Paterson using a high precision GPS unit and referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG6991- 1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless we are 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 sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is comprised of 3 areas, identified as Parcels 1, 2, and 3 herein, and which are bisected by Athlone Avenue and Tweedsmuir Avenue. The location of each parcel is shown on the attached Drawing PG6991-1 – Test Hole Location Plan.

Parcel 1 (1994 Scott St.; 306, 314, 316, 318, 320, 324 & 328 Tweedsmuir Ave; and 327 Athlone Avenue.

This parcel is currently occupied by residential dwellings and associated at-grade parking and landscaped areas, with the exception of the 1994 Scott Street property which is occupied by a single-storey commercial structure. The ground surface across the site gently slopes downward from south to north, from approximate geodetic elevation between 63 to 64 m, and is relatively at-grade with Tweedsmuir Avenue and the neighboring properties.

This parcel is generally bordered by residential dwellings to the west and south, by Scott Street to the north, and by Tweedsmuir Avenue to the east.

Parcel 2 (322, 326, and 330 Athlone Avenue)

This parcel is currently occupied by residential dwellings and associated at-grade parking and landscaped areas. The ground surface across the site is relatively flat ay approximate geodetic elevation of 63 m, and is relatively at-grade with Athlone Avenue and the neighboring properties.

This parcel is bordered by residential dwellings to the north and south, Athlone Avenue to the east, and a commercial building/parking lot to the west.

Parcel 3 (317, 321, 323, 327, 333, and 335 Tweedsmuir Avenue)

This parcel is currently occupied by residential dwellings and associated at-grade parking and landscaped areas. The ground surface across the site is relatively flat at approximate geodetic elevation of 64.5 m and is relatively at-grade with Tweedsmuir Avenue and the neighboring properties.

This parcel is bordered by residential dwellings to the north and south, by a mid-rise residential building to the east, and by Tweedsmuir Avenue to the west.

4.2 Subsurface Profile

Overburden

Parcel 1

Generally, the soil profile at the boreholes consists of a relatively thin layer of fill underlain by glacial till and bedrock. The fill was generally observed to consist of brown silty sand with traces of gravel and clay, extending to approximate depths ranging between 0.5 to 1.2 m below the existing ground surface.

The glacial till was encountered immediately underlying the fill, and was observed to consist of very dense, brown silty sand with gravel, cobbles, and boulders. Refusal to augering was encountered in all the boreholes at approximate depths ranging between 0.5 m and 1.7 m below existing ground surface.

Parcel 2

Generally, the soil profile at the borehole location consists of a relatively thin layer of fill underlain by glacial till and bedrock. The fill was generally observed to consist of brown silty clay with traces of gravel with sand, extending to an approximate depth of 0.8 m below the existing ground surface.

The glacial till was encountered immediately underlying the fill, and was observed to consist of very dense, brown silty sand with gravel, cobbles, and boulders. Refusal to augering was encountered at an approximate depth of 1.2 m below existing ground surface.

Parcel 3

Generally, the soil profile at the boreholes consists of a relatively thin layer of fill underlain by glacial till and bedrock. The fill was generally observed to consist of brown silty sand with traces of gravel and clay, extending to approximate depths ranging between 0.7 to 0.8 m below the existing ground surface.

The glacial till was observed to consist of very dense, brown silty sand with gravel, cobbles, and boulders. Refusal to augering was encountered at approximate depths of between 0.7 to 1.2 m below existing ground surface.

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 borehole location.

Bedrock

The bedrock was cored in all boreholes, with the exception of boreholes BH 4-24, commencing at approximate depths of 0.5 to 1.7 m. The bedrock was observed to consist of limestone, and based on the recovered bedrock core, was generally weathered and of poor to fair in the upper 1 to 2 m, becoming good to excellent in quality below these depths.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation, with an overburden drift thickness of 1 to 3 m.

4.3 Groundwater

Groundwater levels were recorded during the current investigation on February 21, 2024, at each monitoring well location. The measured groundwater level readings are presented in Table 1 below, and are also shown on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Test hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-24	63.55	4.53	59.02	February 21, 2024
BH 2-24	64.25	6.99	57.26	February 21, 2024
BH 3-24	64.03	4.00	60.03	February 21, 2024
BH 5-24	64.98	5.57	59.41	February 21, 2024
BH 6-24	63.40	4.56	58.84	February 21, 2024
BH 7-24	63.41	4.43	58.98	February 21, 2024
BH 8-24	62.98	3.45	59.53	February 21, 2024
BH 9-24	63.19	4.55	58.64	February 21, 2024

Note:
-The ground surface elevation at each test hole location was surveyed using a high precision GPS and are referenced to a geodetic datum.

Based on these observations, the long-term groundwater level is expected to range between approximately 4 to 5 m below ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed buildings are recommended to be founded on conventional spread footings bearing on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground parking levels. 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, paved areas, pipe bedding, and other settlement sensitive structures. However, due to the depth of bedrock and the anticipated founding level for the proposed buildings, it is anticipated that all existing overburden material will be excavated from within the proposed building footprints.

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 proximity of the blasting operations should be completed prior to commencing site activities.

The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is 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

Fill used for grading beneath the building footprints, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill should be tested and approved prior to delivery to the site.

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If non-specified backfill, reviewed and approved by Paterson, is to be placed to build up the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 98% 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 bedrock overbreak occurs at 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 side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the 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 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

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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 on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein, will be subjected 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.

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 buildings, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Floor Slab

For the proposed development, it is anticipated that all overburden soil will be removed from the proposed building footprints, leaving the bedrock as the founding medium for the basement floor slabs. It is anticipated that the basement areas for the proposed buildings will be mostly parking and the recommended pavement structures noted in Section 5.8 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, is 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).

However, the lower portion of the basement walls are to be poured against a composite drainage blanket which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 23.5 kN/m^3 (effective 15.2 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for the foundation wall which is poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil or bedrock should be sed. 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/m3)
> H = height of the wall (m)

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

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

Seismic Earth Pressures

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

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, 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 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, the any permanent 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 sound limestone bedrock 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 0.3, 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 **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 below:

Table 2 – Parameters used in Rock Anchor Review	
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=0.821 and s=0.00293
Unconfined Compressive Strength – Shale Bedrock	40 MPa
Unit weight – Submerged Bedrock	15.2 kN/m^3
Apex Angle of Failure Cone	60°
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 3 on the next page. The factored tensile resistance values given in Table 3 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 3 – 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	0.9	1.1	2.0	210
	1.8	1.2	3.0	420
	4.0	1.2	5.0	900
	5.0	1.0	6.0	1150
125	0.7	1.4	2.1	250
	1.2	1.7	2.9	470
	2.0	1.9	3.9	800
	3.0	2.0	5.0	1150

Other Considerations

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

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.

5.8 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 4 below.

Table 4 - 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 hours after the concrete has been poured during warm temperatures, and up to 12 hours during cooler temperatures.

the following pavement structures may be considered for at-grade car only parking and heavy traffic areas, should they be required. The proposed pavement structures are shown in Tables 5 and 6.

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 in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Table 6 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

It is recommended that the proposed building foundation walls located below finished grades be blind-poured and 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 vertical bedrock face, the following is recommended:

- ❑ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).
- ❑ Mechanically remove bedrock along the foundation walls, up to approximately 150 mm from the finished vertical excavation face.
- ❑ Grind the bedrock surface up to the outer face of the line drilled holes to create a satisfactory surface for the waterproofing membrane and/or composite drainage board.
- ❑ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features of the bedrock surface, as required based on site inspection by Paterson.
- ❑ Place a suitable waterproofing membrane (such as Tremco Paraseal or approved equivalent) against the prepared vertical bedrock surface. The membrane liner should extend from 5 m below finished grade, down to footing level.
- ❑ Place a composite drainage board, 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 board.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast at the foundation wall/footing interface to allow for the infiltration of 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.

Elevators and any other pits located below the underslab drainage system should be waterproofed. A full waterproofing detail for the foundation walls and the mechanical pits can be provided by Paterson, if required.

Perimeter and Underslab Drainage System

The perimeter and underslab drainage system is recommended to control water infiltration below the underground parking level slab and to re-direct water from the buildings foundation drainage system to the building's sump pit(s). For preliminary design purposes, it is recommended that 150 mm perforated pipes provided with a geosock, surrounded on all sides by a minimum 150 mm thick layer of 19 mm clear crushed stone, be placed at approximate 6 m centres underlying the underground parking level slab.

The perimeter drainage system should be mechanically connected to the 150 mm drainage sleeves and gravity connected to the underslab drainage system, which in turn is connected to the building's sump pit(s).

The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials, such as clean sand or OPSS Granular B Type I granular material.

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.

Exterior unheated footings, such as isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation. It is recommended that Paterson review the proposed frost protection detail for the proposed development.

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 be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations).

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The 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.

Excavation side slopes carried out for the building footprint are recommended to be provided with surface protection from erosion by rain and surface water runoff, where 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 overburden 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 the side slopes.

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

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

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

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations, where insufficient room is available for open cut methods. The shoring requirements, designed by a structural engineer specializing in those works, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. The geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should consider the impact of a significant precipitation event and designate design measures to ensure that 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 designer prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system. Any additional loading due to street traffic, neighbouring buildings, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below. These systems could be cantilevered, anchored, or braced.

The earth pressures acting on the temporary shoring system may be calculated with the parameters presented in Table 7, presented below.

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
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated to full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Bedrock Stabilization

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 chainlink fencing, shotcrete, and/or rock bolts should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project. It is anticipated that such measures will be required, at a minimum, for the upper, weathered limestone bedrock.

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.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material for areas over a soil subgrade. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

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

Generally, it should be possible to re-use the moist (not wet) silty sand to sandy silt and glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions.

Wet sub-excavated soil should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. All stones greater than 300 mm in their greatest dimension should be removed prior to reuse of site-generated glacial till.

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 consist of 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 SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be relatively low to moderate, and controllable using open sumps.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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.

Adverse Effects of Dewatering on Adjacent Properties

Given the shallow bedrock present at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on the bedrock surface. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

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.

6.7 Corrosion Potential and Sulphate

The analytical test results of the soil sample indicate that the sulphate content is less than 0.1%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (Type GU) would be appropriate for this site. The chloride content and the pH of the sample indicate they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to slightly aggressive environment.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing and landscaping plans, from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the foundation drainage systems prior to construction, if applicable.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.

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

- ❑ Review and inspection of the installation of the foundation drainage and waterproofing systems.
- ❑ 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 and follow-up field density tests to determine the level of compaction achieved.
- ❑ 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 soil must 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 Park River Properties, 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.

Puneet Bandi, M.Eng

Scott Dennis, P.Eng.

Report Distribution:

- ❑ Park River Properties
- ❑ Paterson Group

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 Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363412.742 **NORTHING:** 5028724.498 **ELEVATION:** 63.55

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-Mounted Drill **DATE:** February 7, 2024

FILE NO. PG6991

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	63.55
ASPHALT (0.10)							
FILL: Brown silty sand with gravel, trace clay (0.84)		AU	1				
GLACIAL TILL: Very dense, brown silty sand with gravel		SS	2	42	50	1	62.55
(1.65)		SS	3	0	+50		
BEDROCK: Good quality limestone bedrock						2	61.55
		RC	1	100	84		
						3	60.55
		RC	2	100	76		
						4	59.55
		RC	3	100	86	5	58.55
		RC	4	1000	100	6	57.55
(6.20) End of Borehole							
(GWL @ 4.53m - Feb. 21, 2024)							

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

○ Water Content %

20 40 60 80

PIEZOMETER CONSTRUCTION

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363444.685 **NORTHING:** 5028720.724 **ELEVATION:** 64.25

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-Mounted Drill **DATE:** February 7, 2024

FILE NO. PG6991

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone / ○ Water Content %	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	64.25		
ASPHALT 0.05									
FILL: Brown silty sand with gravel 0.69		AU	1						
BEDROCK: Good to excellent quality limestone bedrock		RC	1	100	88	1	63.25		
- poor quality between 1.1m to 2.6m depth		RC	2	86	47	2	62.25		
		RC	3	100	80	3	61.25		
		RC	4	95	83	4	60.25		
						5	59.25		
		RC	5	89	73	6	58.25		
		RC	6	100	92	7	57.25		
7.54									
End of Borehole									
(GWL @ 6.99m - Feb. 21, 2024)									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363430.074 **NORTHING:** 5028670.642 **ELEVATION:** 64.03

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-Mounted Drill **DATE:** February 8, 2024

FILE NO. PG6991

HOLE NO. BH 3-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
ASPHALT	0.05				
FILL: Brown silty sand with gravel, trace clay	0.51	AU	1		
BEDROCK: Fair to good quality limestone bedrock		RC	1	94	53
		RC	2	76	54
		RC	3	98	86
- excellent quality by 4.1m depth		RC	4	98	93
	7.11	RC	5	98	95
End of Borehole					
(GWL @ 4.00m - Feb. 21, 2024)					

DEPTH (m)	ELEV. (m)
0	64.03
1	63.03
2	62.03
3	61.03
4	60.03
5	59.03
6	58.03
7	57.03

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363414.027 **NORTHING:** 5028683.825 **ELEVATION:** 63.47
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-Mounted Drill **DATE:** February 8, 2024

FILE NO. PG6991
HOLE NO. BH 4-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	63.47		
ASPHALT 0.05									
FILL: Brown silty sand with gravel, trace clay 0.69		AU	1						
GLACIAL TILL: Very dense, brown silty sand with gravel 1.17		SS	2	85	+50	1	62.47		
End of Borehole Practical refusal to augering on inferred boulder or bedrock at 1.17m depth									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363474.105 **NORTHING:** 5028664.211 **ELEVATION:** 64.98

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-Mounted Drill **DATE:** February 12, 2024

FILE NO. PG6991

HOLE NO. BH 5-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.98
ASPHALT	0.03						
FILL: Brown silty sand with gravel, trace clay	0.69	AU	1				
GLACIAL TILL: Very dense, brown silty sand with gravel	1.17	SS	2	50	+50	1	63.98
BEDROCK: Fair to good quality limestone bedrock		RC	1	83	63	2	62.98
						3	61.98
		RC	2	97	85	4	60.98
		RC	3	97	77	5	59.98
- excellent quality by 5.8m depth						6	58.98
		RC	4	100	97	7	57.98
	7.70	RC	5	100	100		

End of Borehole

(GWL @ 5.57m - Feb. 21, 2024)

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

MONITORING WELL CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363376.246 **NORTHING:** 5028739.503 **ELEVATION:** 63.40
DATUM: Geodetic
REMARKS:
BORINGS BY: Truck-Mounted Drill **DATE:** February 12, 2024

FILE NO. PG6991
HOLE NO. BH 6-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
ASPHALT	0.05				
FILL: Brown silty sand with gravel	0.69	AU	1		
GLACIAL TILL: Brown silty sand with gravel	1.22	SS	2	36	+50
BEDROCK: Fair to good quality limestone bedrock		RC	1	81	64
- excellent quality by 2.7m depth		RC	2	100	95
		RC	3	100	98
		RC	4	100	100
	7.70	RC	5	100	100
End of Borehole					
(GWL @ 4.56m - Feb. 21, 2024)					

DEPTH (m)	ELEV. (m)
0	63.40
1	62.40
2	61.40
3	60.40
4	59.40
5	58.40
6	57.40
7	56.40

Pen. Resist. Blows / 0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

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

MONITORING WELL CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363383.893 **NORTHING:** 5028749.401 **ELEVATION:** 63.41

DATUM: Geodetic

REMARKS:

BORINGS BY: Truck-Mounted Drill **DATE:** February 12, 2024

FILE NO. PG6991

HOLE NO. BH 7-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone / ○ Water Content %	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	63.41		
ASPHALT 0.05									
FILL: Brown silty sand with gravel and crushed stone, some clay		AU	1						
1.17		SS	2	14	+50	1	62.41		
BEDROCK: Good to excellent quality limestone bedrock		RC	1	98	83	2	61.41		
- fair quality between 2.7m to 4.2m depth						3	60.41		
		RC	2	87	73	4	59.41		
		RC	3	92	85	5	58.41		
						6	57.41		
		RC	4	100	100	7	56.41		
7.67		RC	5	100	100				
End of Borehole									
(GWL @ 4.43m - Feb. 21, 2024)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363358.408 **NORTHING:** 5028636.587 **ELEVATION:** 62.98

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 13, 2024

FILE NO. PG6991

HOLE NO. BH 8-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	62.98
ASPHALT	0.05						
FILL: Brown silty clay with gravel and sand	0.84	AU	1				
GLACIAL TILL: Very dense, brown silty sand with gravel	1.22	SS	2	29	+50	1	61.98
BEDROCK: Poor to fair quality limestone bedrock		RC	1	79	36		
		RC	2	90	80	2	60.98
- excellent quality by 3.1m depth						3	59.98
		RC	3	100	98	4	58.98
		RC	4	97	90	5	57.98
	6.10					6	56.98
End of Borehole							

(GWL @ 3.45m - Feb. 21, 2024)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

PIEZOMETER CONSTRUCTION

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1994 Scott Street
Ottawa, Ontario

EASTING: 363398.439 **NORTHING:** 5028757.149 **ELEVATION:** 63.19

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 13, 2024

FILE NO. PG6991

HOLE NO. BH 9-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
ASPHALT	0.05				
FILL: Brown silty sand with gravel	0.91	AU	1		
BEDROCK: Poor to fair quality limestone bedrock		RC	1	84	33
		RC	2	92	78
- excellent quality by 3.1m depth		RC	3	100	97
		RC	4	100	97
	7.59	RC	5	100	93
End of Borehole					
(GWL @ 4.55m - Feb. 21, 2024)					

DEPTH (m)	ELEV. (m)
0	63.19
1	62.19
2	61.19
3	60.19
4	59.19
5	58.19
6	57.19
7	56.19

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

○ Water Content %

20 40 60 80

MONITORING WELL CONSTRUCTION

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

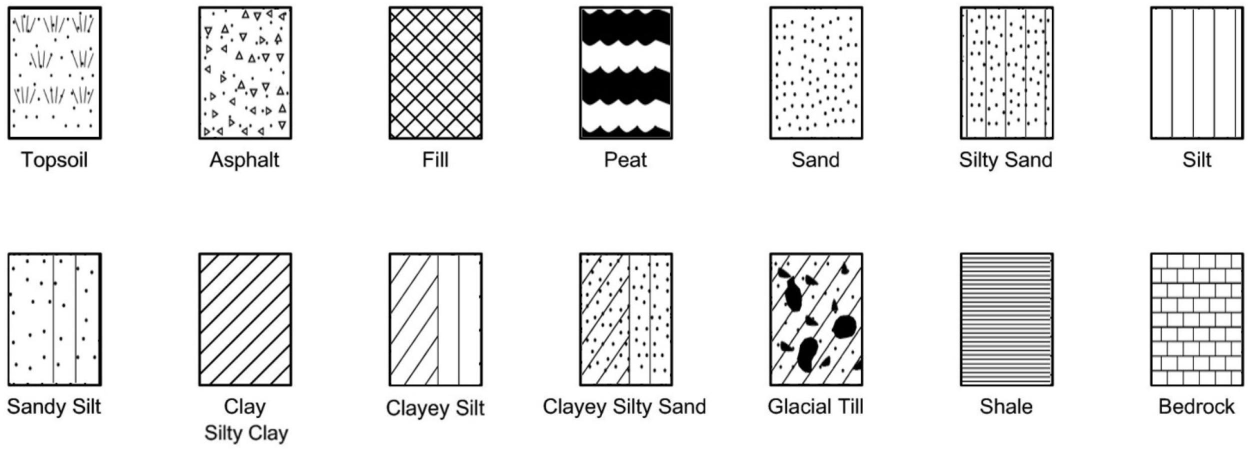
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

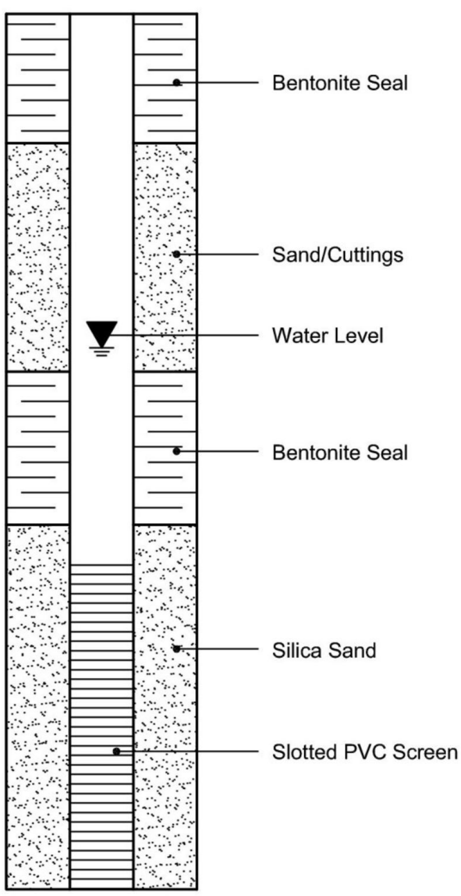
SYMBOLS AND TERMS (continued)

STRATA PLOT

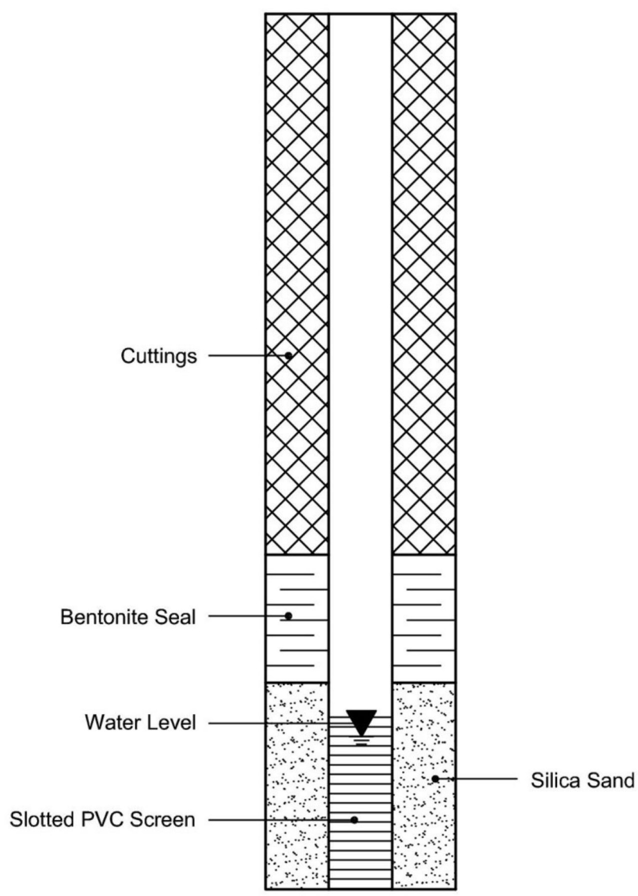


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL
LABORATORIES LTD.

Order #: 2407393

Certificate of Analysis

Report Date: 22-Feb-2024

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

Order Date: 15-Feb-2024

Client PO: 59435

Project Description: PG6991

	Client ID:	BH1-24-SS2	-	-	-	-	-
	Sample Date:	15-Feb-24 09:00	-	-	-		
	Sample ID:	2407393-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	97.7	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.33	-	-	-	-	-
Resistivity	0.1 Ohm.m	42.4	-	-	-	-	-
Anions							
Chloride	10 ug/g	39	-	-	-	-	-
Sulphate	10 ug/g	57	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6991-1 – TEST HOLE LOCATION PLAN

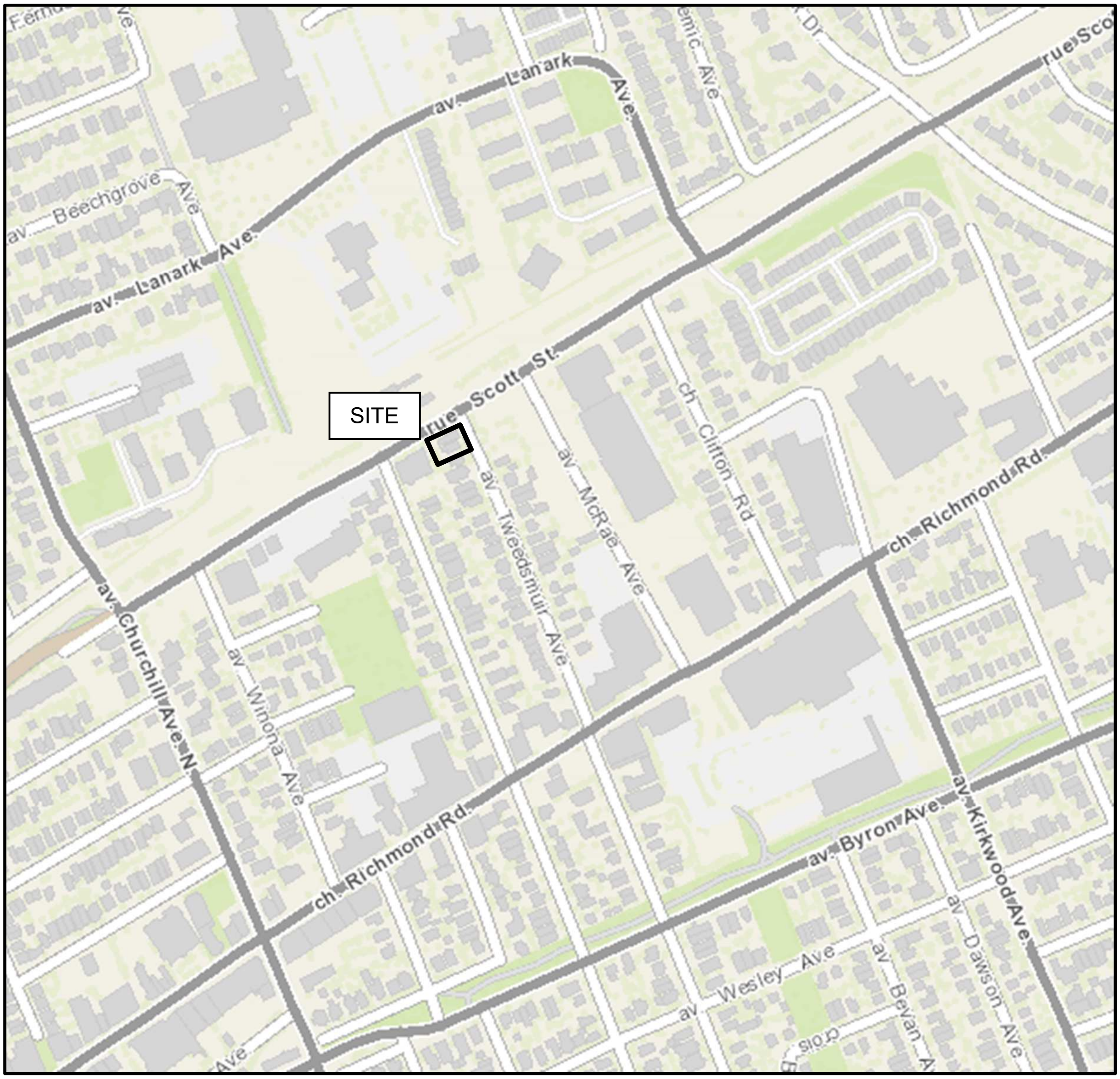
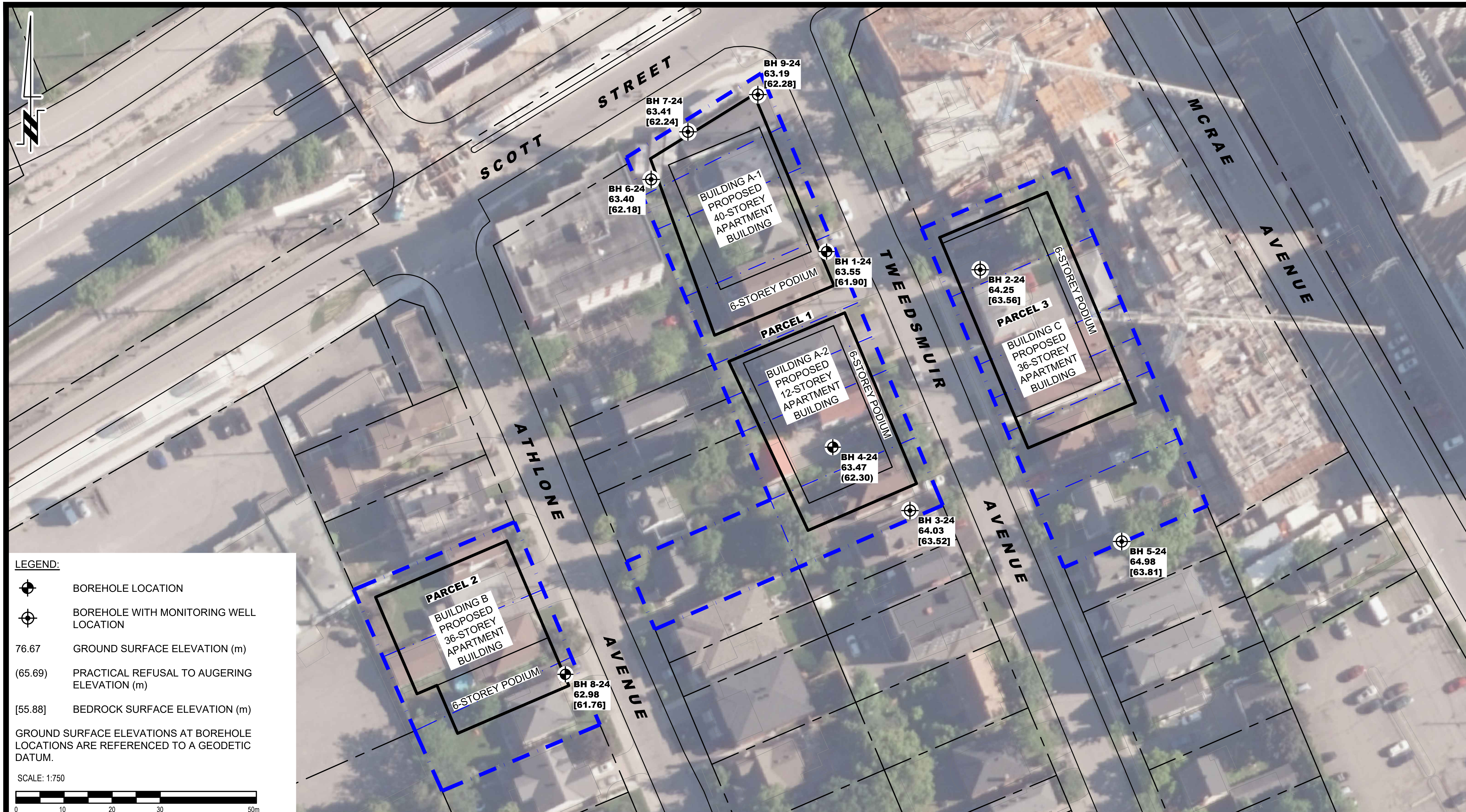


FIGURE 1

KEY PLAN

PATERSON GROUP



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

NO.	REVISIONS	DATE	INITIAL

PARK RIVER PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
1994 SCOTT STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 02/2024
Drawn by: ZS	Report No.: PG6991-1
Checked by: PB	Dwg. No.: PG6991-1
Approved by: SD	Revision No.:

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