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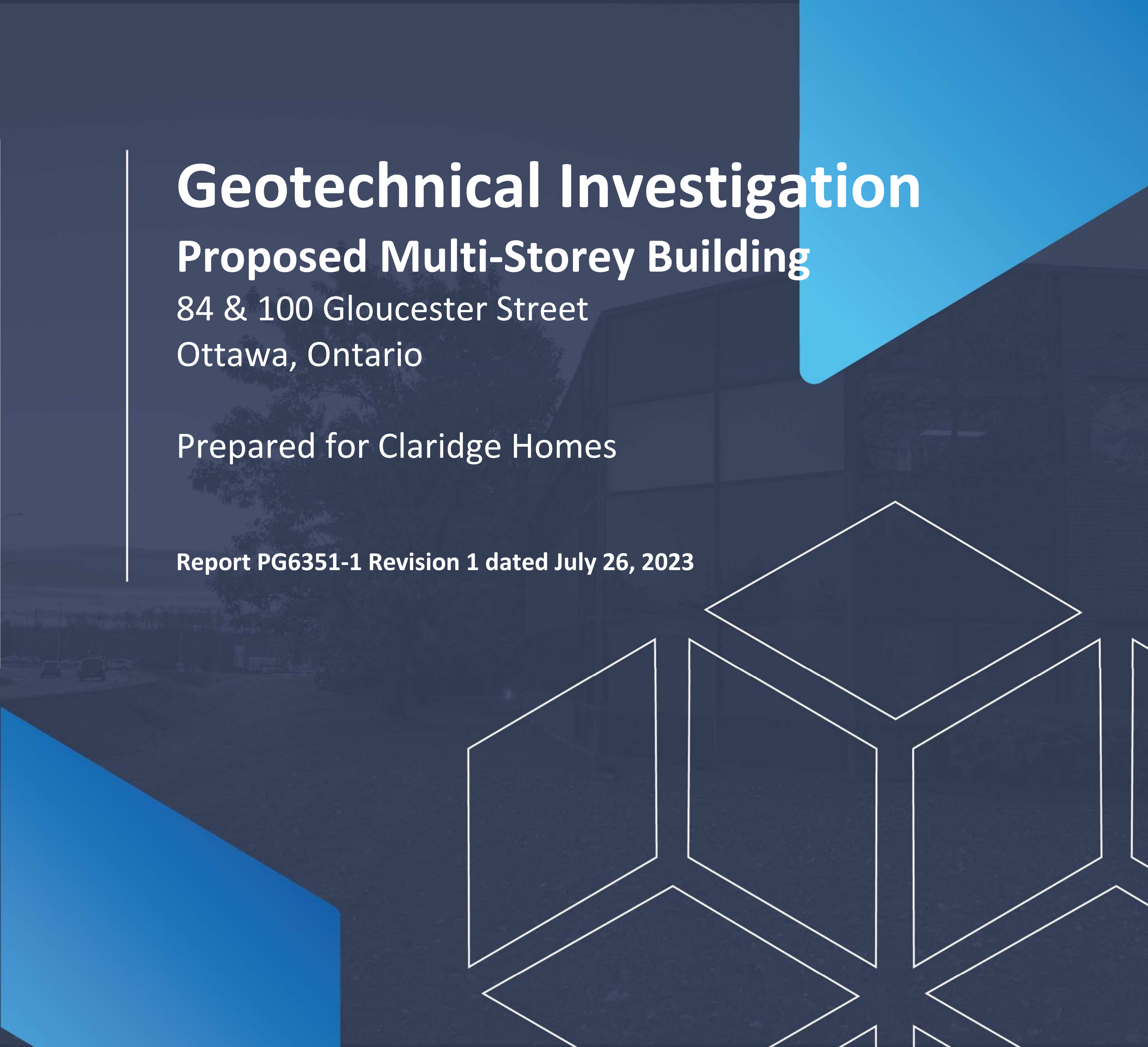

Geotechnical Investigation

Proposed Multi-Storey Building

84 & 100 Gloucester Street
Ottawa, Ontario

Prepared for Claridge Homes

Report PG6351-1 Revision 1 dated July 26, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to prepare a Geotechnical Investigation Report for the proposed multi-storey building to be located at 84 & 100 Gloucester Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the Geotechnical Investigation Report are 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.

2.0 Proposed Development

Based on the available drawings, the proposed development will consist of a high-rise building with 5 levels of underground parking, which will occupy the majority of the subject site. It is also understood that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out from March 31 to April 5, 2022, and consisted of advancing a total of 7 boreholes to a maximum depth of 8.2 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6351-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a truck-mounted power auger 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 testing procedure consisted of augering and excavating to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

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

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

Borehole BH 4-22 was fitted with a 51 mm diameter PVC groundwater monitoring well to monitor the long-term groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The ground surface elevations at the borehole locations were surveyed using a handheld GPS unit and are understood to be referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG6351-1 - 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.

4.0 Observations

4.1 Surface Conditions

The subject site, which consists of 2 contiguous properties, is currently occupied by a multi-storey building and outdoor pool in the western portion of the site, and asphalt-paved access lanes and parking areas in the eastern portion of the site.

The site is bordered to the north by Gloucester Street, to the east, south, and west by multi-storey buildings. The subject site is at grade with Gloucester Street at approximate geodetic elevation 71 m.

A 900 mm diameter backbone watermain is located approximately 10 m to the north of the site, underlying Gloucester Street.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the test hole locations consists of asphaltic concrete or a concrete slab underlain by fill, silty clay and glacial till. The fill material generally extended to approximate depths of 0.3 to 2.7 m below the existing ground surface, and consists of brown silty sand to silty clay with varying amounts of gravel, clay, and brick.

The silty clay deposit was encountered underlying the fill, and consists of a stiff to soft, brown silty clay which extends to approximate depths ranging from 0.9 to 5 m below the existing ground surface, generally increasing from west to east across the site.

A glacial till deposit was encountered underlying the silty clay, and consists of a loose to very dense, dark brown silty sand with varying amounts of clay, gravel cobbles and boulders. The glacial till was observed to extend to depths ranging from 4.5 to 8.1 m below the existing ground surface, where penetrated.

Reference should be made to the Log of Borehole sheets by others in Appendix 1 for the details of the soil profile encountered at each borehole location.

Bedrock

Bedrock was encountered at approximate depths ranging from 4.2 to 8.1 m below the existing ground surface, and, based on the recovered split spoon samples, was observed to consist of shale.

Based on available geological mapping, the bedrock in the subject area consists of shale of the Billings Formation.

4.3 Groundwater

Groundwater levels were measured on April 22, 2022 in the monitoring wells installed at the borehole locations. The measured groundwater levels noted at that time are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 2-22	69.72	4.61	65.11	April 22, 2022
BH 3-22	71.22	8.13	63.09	
BH 4-22	71.51	3.72	67.79	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to a geodetic datum.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate geodetic elevation 65 to 67 m

The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater 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 multi-storey building is recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Should the clean, surface sounded bedrock be encountered below the underside of footing elevation, lean concrete trenches can be used to raise grades from the clean, surface sounded bedrock to the founding elevation.

Given the presence of a silty clay deposit, a permissible grade raise restriction has been provided for the subject site. Bedrock removal will likely be required to complete the underground parking levels.

In addition, due to the proximity of the 900 mm diameter backbone watermain, additional precautions should be taken during excavation activities to ensure that the existing service is not affected.

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.

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

Bedrock Removal

Should it be required, 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.

Vibration Monitoring Plan for 900 mm Diameter Backbone Watermain

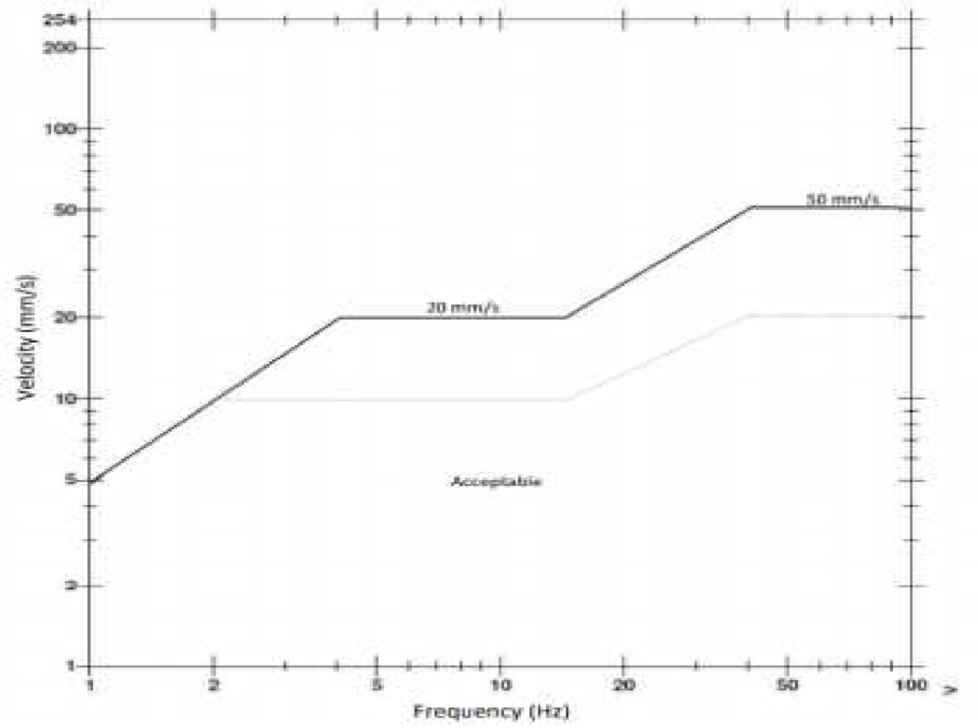
The following vibration monitoring program is recommended to ensure that excessive vibrations do not occur at the nearby watermain location:

- ☐ Vibration levels will be continuously monitored using 2 vibration monitors which are installed at the nearest site boundary.

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel.

The vibration limits, provided below, are recommended for the construction operation to be completed in the vicinity of the 900 mm diameter watermain.

Vibration Criteria Figure - Proposed Vibration Limits at the Watermain



The monitoring protocol should include the following information:

Warning Level Event

❑ Paterson will review all vibrations over the established warning level, illustrated by the blue line in the above figure, and;

❑ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event

❑ Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in the above figure.

❑ Ensure monitors are functioning

❑ Issue the vibration exceedance result

The data collected should include the following:

❑ Measured vibration levels
❑ Distance from the construction activity to monitoring location
❑ Vibration type

Monitoring should be compliant with all related regulations.

Incidence & Exceedance Reporting

In case a vibration incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

❑ The location of vibration exceedance
❑ The date, time and nature of the exceedance/incident
❑ Purpose of the exceeded monitor and current vibration criteria
❑ The likely cause of the exceedance/incident
❑ The response action that has been completed to date
❑ The proposed measures to address the exceedance/incident.

The contractor should implement mitigation measures for future excavation or any construction activities as necessary and provide updates on the effectiveness of the improvement. Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any vibrations to identify issues and react in a quick manner in the event of an exceedance.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill, where required, should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Lean Concrete Filled Trenches

Where the clean, surface sounded bedrock is encountered below the underside of footing elevation, vertical trenches can 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 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

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock, or on lean concrete which is placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 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. A soil bearing medium, or a heavily fractured, weathered bedrock bearing medium, will require a lateral support zone of 1H:1V (or shallower).

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1 m** is recommended for grading at the subject 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 footings are 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. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

For the proposed development, the fill, native soil, and/or bedrock are considered suitable subgrades for the basement floor slab. It is anticipated that the basement area for the proposed building 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).

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/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·a ·H^2/g where:

$a_c = (1.45 - a_{max}/g)a_{max}$
ɣ = 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 $P_o = 0.5\ K_o \cdot ɣ \cdot H^2$, 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 shale bedrock ranges between 40 to 50 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 on the next page:

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 Shale 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 below. The factored tensile resistance values give 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	410
	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.

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 pavement structures presented in Tables 5 and 6 should be used for car only parking areas, at grade access lanes and heavy loading parking areas overtop of a podium deck structure, should they be required:

Table 5 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck)

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in proceeding paragraph

Table 6 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck)

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in proceeding paragraph

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab, a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load (HI-40). The pavement structures base layer thickness will be dependent on the grade of insulation considered for this project and should be re-assessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is not considered suitable for this application.

Pavement Structure Over Overburden

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The proposed pavement structures are shown in Tables 7 and 8.

Table 7 - 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 8 - 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 Waterproofing and Foundation Drainage

It is recommended that any portion of the proposed building foundation walls located below the long-term groundwater table (approximate geodetic elevation 65 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 any 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 65 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 the elevator pits (pit bottom and walls).

Underslab drainage will be required to control water infiltration below the lowest level floor slab. 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.

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 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 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 and Temporary Shoring

The side slopes of excavations in the overburden materials and weathered bedrock 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 cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

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 horizontal rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Due to the anticipated proximity of the underground parking levels to the site boundaries, temporary shoring is anticipated to be required for the support of the overburden soils and weathered or poor quality bedrock during the excavation. The design and approval of the temporary 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 will not negatively impact the shoring system or soils supported by the system.

The temporary shoring system may generally consist of a soldier pile and lagging system or steel sheet piles. However, on the west side of the excavation, which is adjacent to a multi-storey building, it is recommended that the temporary shoring system consist of a secant pile wall. Generally, the temporary shoring systems should be provided with tie-back rock anchors to ensure their stability.

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

Table 9 – 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	21
Effective Unit Weight (γ’), kN/m^3	13

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described above.

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

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

6.4 Pipe Bedding and Backfill

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

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 should generally be 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 95% 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

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

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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts to Neighbouring Properties

As noted above, any portion of the foundation walls located below the groundwater level will have 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 by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost 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.

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 geotechnical aspects of the excavating contractor’s shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed waterproofing and foundation drainage design and requirements

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

☐ Sampling and testing of the concrete and fill materials.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

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

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

The soils investigation by others is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations by others, 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 Claridge Homes, 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.

Otillia McLaughlin B.Eng.

LICENSED PROFESSIONAL ENGINEER
July 26, 2023
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution:

☐ Claridge Homes (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

DATE March 31, 2022

FILE NO. PG6351

HOLE NO. BH 1-22

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 % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	69.72		
Concrete slab 0.20									
FILL: Crushed stone with gravel, some clay 0.30		AU	1						
Brown **SILTY CLAY** 0.91		SS	2	50					
End of Borehole Practical spoon refusal at 0.91m depth.									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

DATE April 1, 2022

FILE NO. **PG6351**

HOLE NO. **BH 2-22**

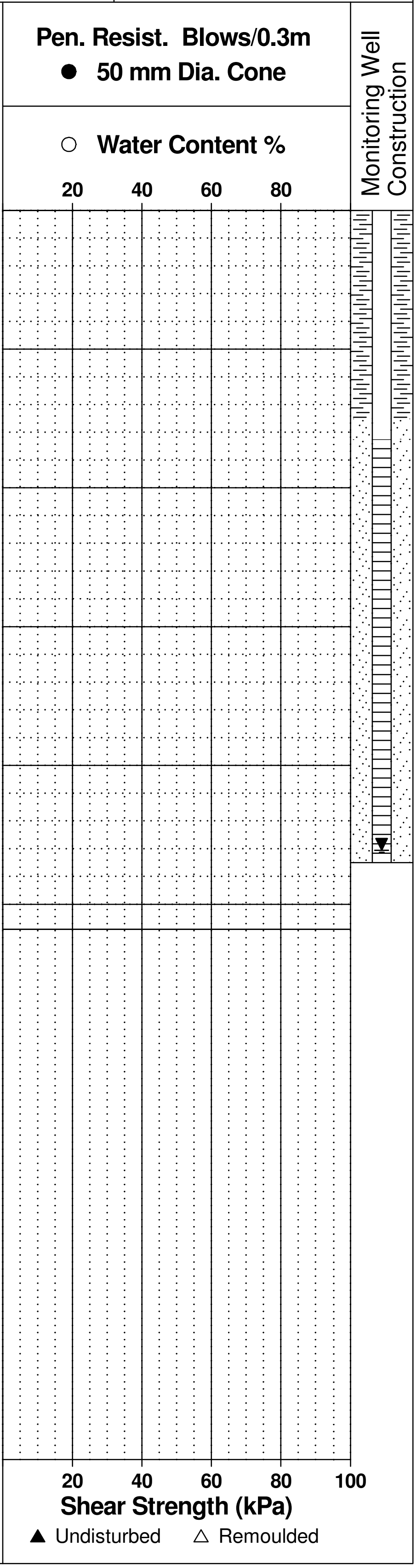
SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Concrete slab	0.20
FILL: Brown silty clay, trace gravel	0.30
Brown **SILTY CLAY**	1.52
GLACIAL TILL: Brown silty clay with sand, gravel and weathered shale, occasional cobbles and boulders	4.50
BEDROCK: Weathered shale	5.18

End of Borehole

(GWL @ 4.61m - May 16, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	
SS	3	75	
SS	4	100	
SS	5	100	
SS	6	100	
SS	83	7	
SS	8	100	
SS	9	33	

DEPTH (m)	ELEV. (m)
0	69.72
1	68.72
2	67.72
3	66.72
4	65.72
5	64.72



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Truck-Mount Power Auger

DATE April 4, 2022

FILE NO. **PG6351**

HOLE NO. **BH 3-22**

SOIL DESCRIPTION

GROUND SURFACE

SOIL DESCRIPTION	Bottom depth (m)
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone and gravel, trace clay	1.32
Soft to firm, brown **SILTY CLAY,** trace sand	3.48
GLACIAL TILL: Loose to very dene, dark brown silty clay with sand and gravel, occasional cobbles and boulders	8.05
Weathered **BEDROCK**	8.23

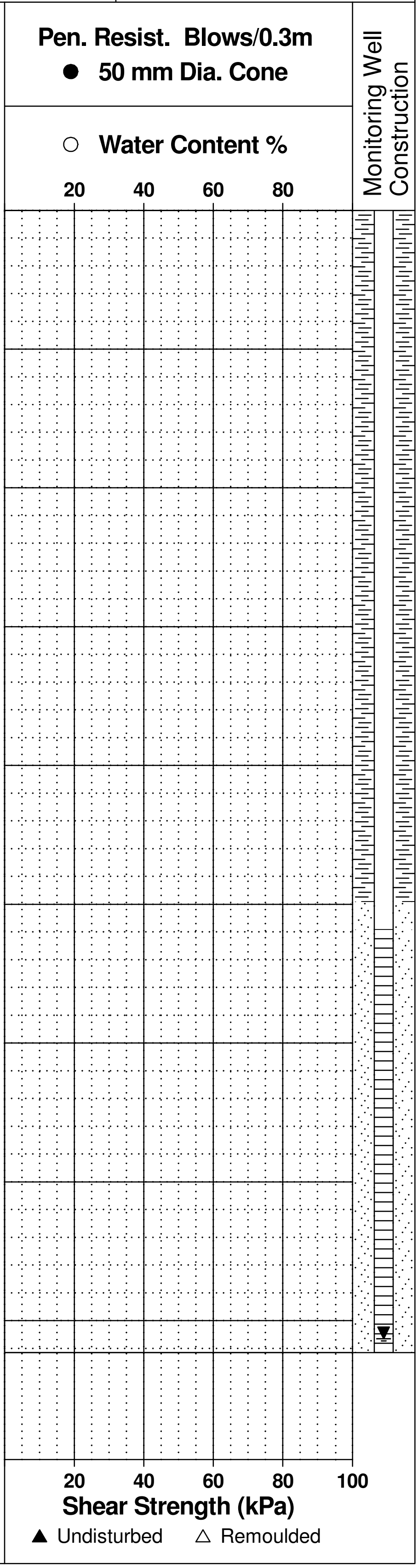
End of Borehole

(GWL @ 8.13m - May 16, 2022)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	13
SS	3	46	4
SS	4	100	3
SS	5	100	2
SS	6	62	6
SS	7	67	9
SS	8	67	6
SS	9	71	9
SS	10	33	50+
SS	11	50	34

DEPTH (m)	ELEV. (m)
0	71.22
1	70.22
2	69.22
3	68.22
4	67.22
5	66.22
6	65.22
7	64.22
8	63.22



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Truck-Mount Power Auger

DATE April 5, 2022

FILE NO. **PG6351**

HOLE NO. **BH 4-22**

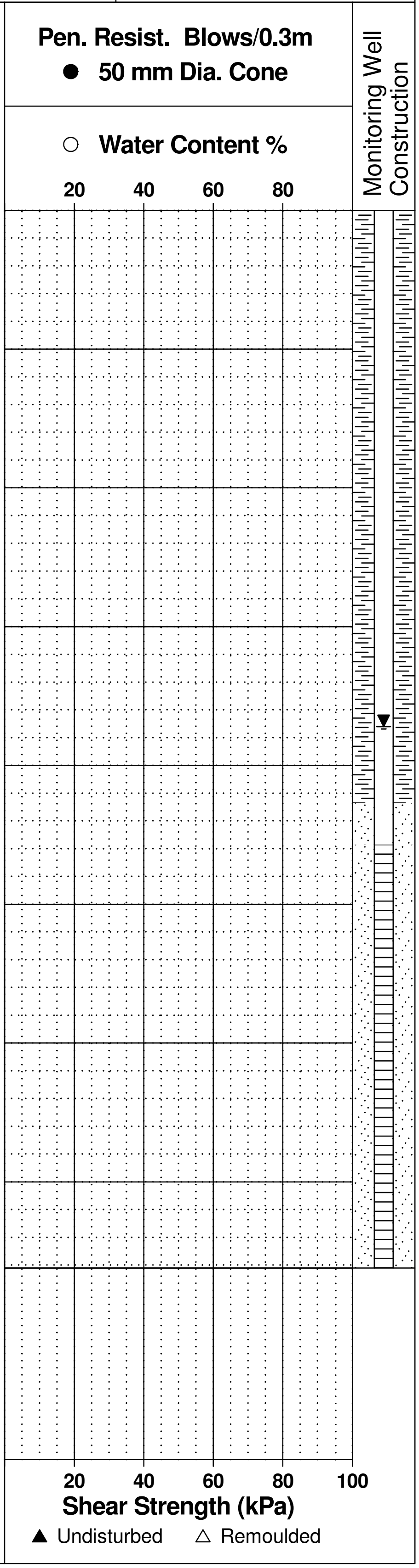
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone and gravel, trace clay and brick	2.74
Soft to firm, brown **SILTY CLAY**	5.03
GLACIAL TILL: Dark brown silty clay with sand and gravel, occasional cobbles and boulders	7.62

End of Borehole

(GWL @ 3.72m - April 22, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	40
SS	3	58	4
SS	4	58	15
SS	5	100	4
SS	6	100	2
SS	7	100	5
SS	8	67	3
SS	9	38	3
SS	10	71	1

DEPTH (m)	ELEV. (m)
0	71.51
1	70.51
2	69.51
3	68.51
4	67.51
5	66.51
6	65.51
7	64.51



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Truck-Mount Power Auger

DATE April 4, 2022

FILE NO. PG6351

HOLE NO. BH 5-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.31
Asphaltic concrete 0.08						
FILL: Brown silty sand with crushed stone and gravel, trace clay 1.01	AU	1				
Firm, brown **SILTY CLAY**	SS	2	100	6	1	70.31
	SS	3	100	5	2	69.31
2.64	SS	4	100	8		
GLACIAL TILL: Loose to dense, dark brown silty clay with sand and gravel, occasional cobbles and boulders	SS	5	58	7	3	68.31
4.22	SS	6	56	50+	4	67.31
End of Borehole Practical refusal to augering at 4.22m depth.						

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
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Truck-Mount Power Auger

DATE April 4, 2022

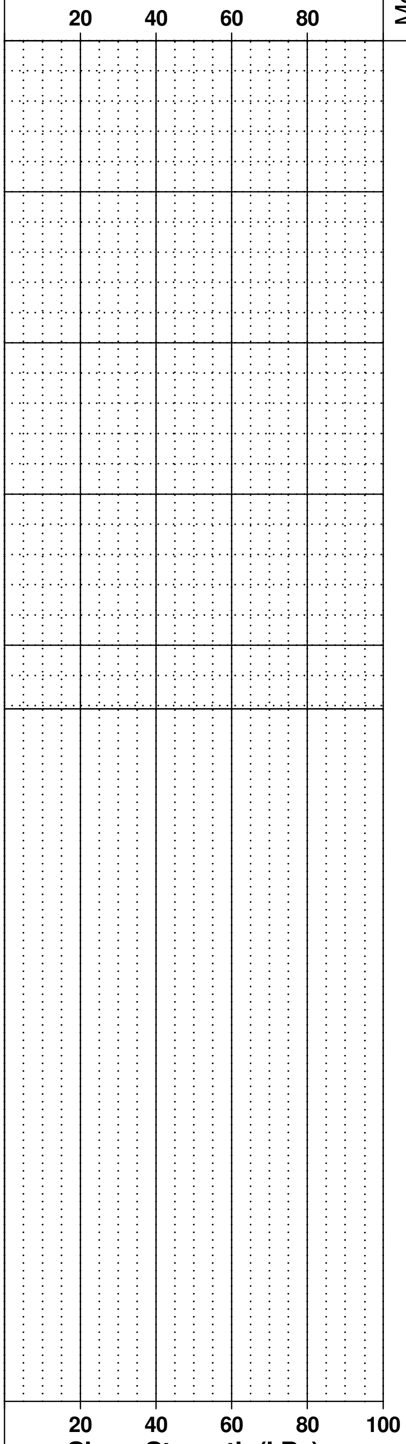
FILE NO. PG6351

HOLE NO. BH 6-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0	71.46	
Asphaltic concrete 0.08								
FILL: Brown silty clay with crushed stone and gravel, trace sand 0.60		AU	1					
FILL: Brown silty sand with crushed stone and gravel 0.91								
Firm to stiff, brown **SILTY CLAY**		SS	2	100	15	1	70.46	
		SS	3	100	7	2	69.46	
		SS	4	100	4	3	68.46	
3.58		SS	5	75	10			
GLACIAL TILL: Compact, dark brown silty clay with sand and gravel, occasional cobbles and boulders 4.42		SS	6	62	13	4	67.46	
End of Borehole								

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 84 & 100 Gloucester St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

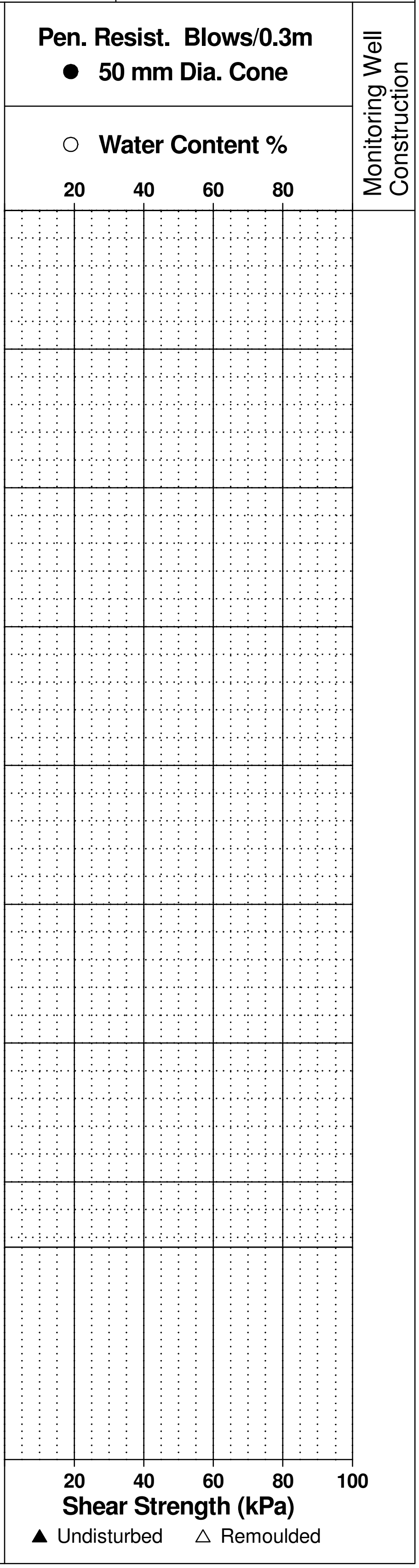
BORINGS BY Truck-Mount Power Auger

DATE April 5, 2022

FILE NO. **PG6351**

HOLE NO. **BH 7-22**

SOIL DESCRIPTION	Bottom of stratum (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.29
Asphaltic concrete	0.08						
FILL: Brown silty sand with crushed stone and gravel, trace clay	1.22	AU	1				
		SS	2	50	6	1	70.29
Firm, brown **SILTY CLAY**	3.73	SS	3	100	7	2	69.29
		SS	4	100	5	3	68.29
		SS	5	46	5		
GLACIAL TILL: Compact, dark brown silty clay with sand and gravel, occasional cobbles and boulders	7.47	SS	6	71	6	4	67.29
		SS	7	50	9	5	66.29
		SS	8	38	7	6	65.29
		SS	9	79	6	7	64.29
		SS	10	54	17		
End of Borehole							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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)

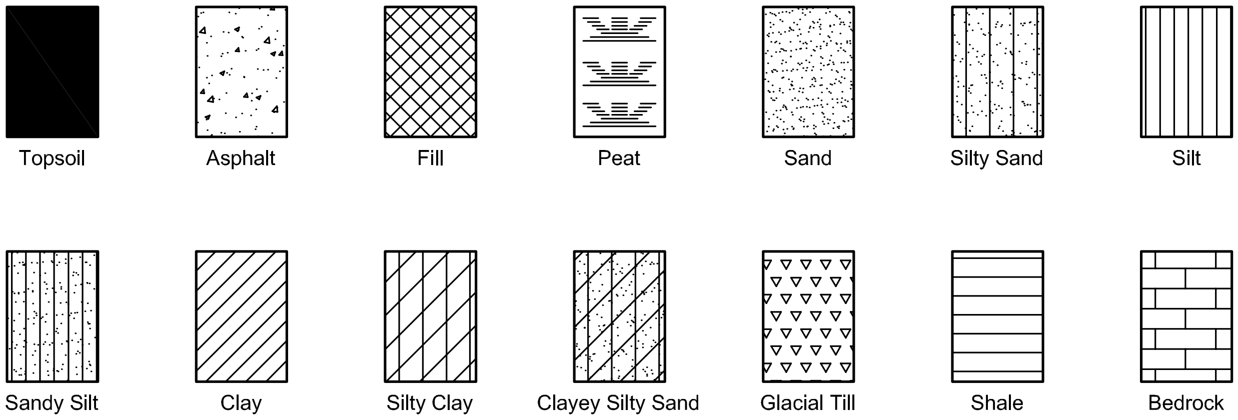
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

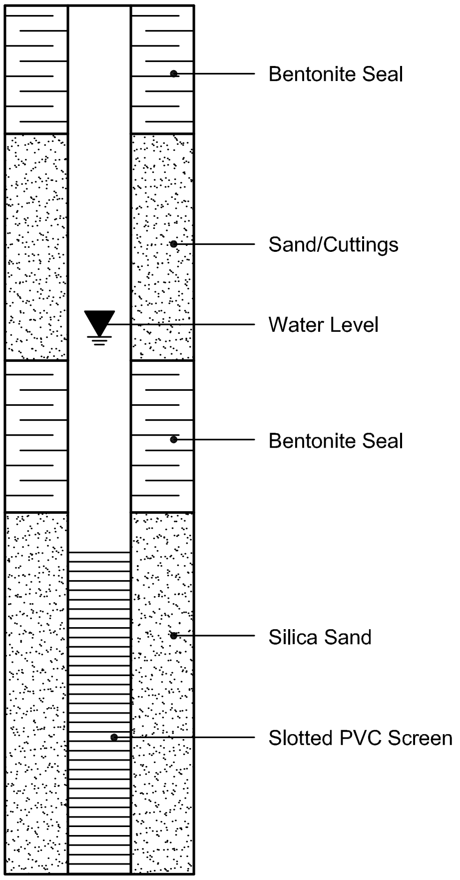
SYMBOLS AND TERMS (continued)

STRATA PLOT

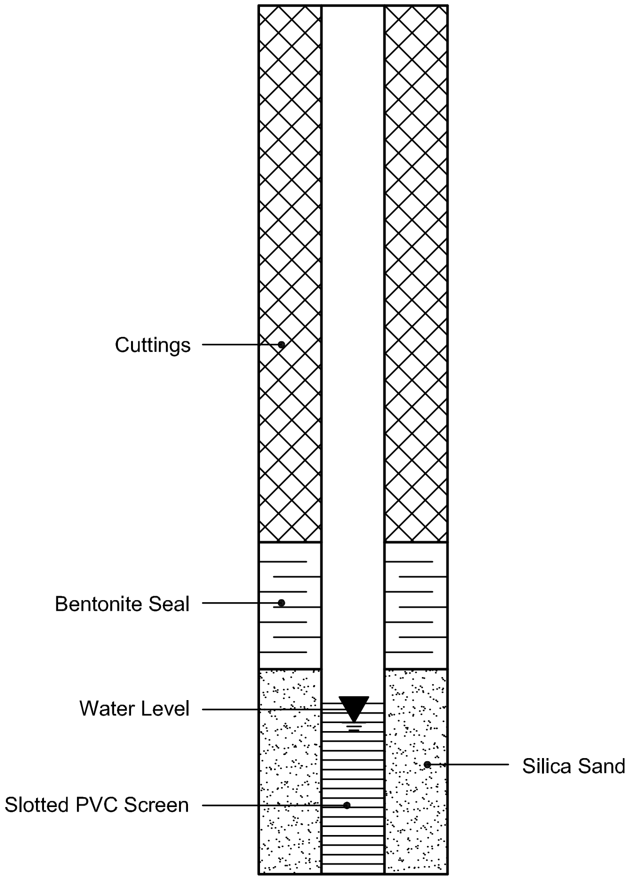


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

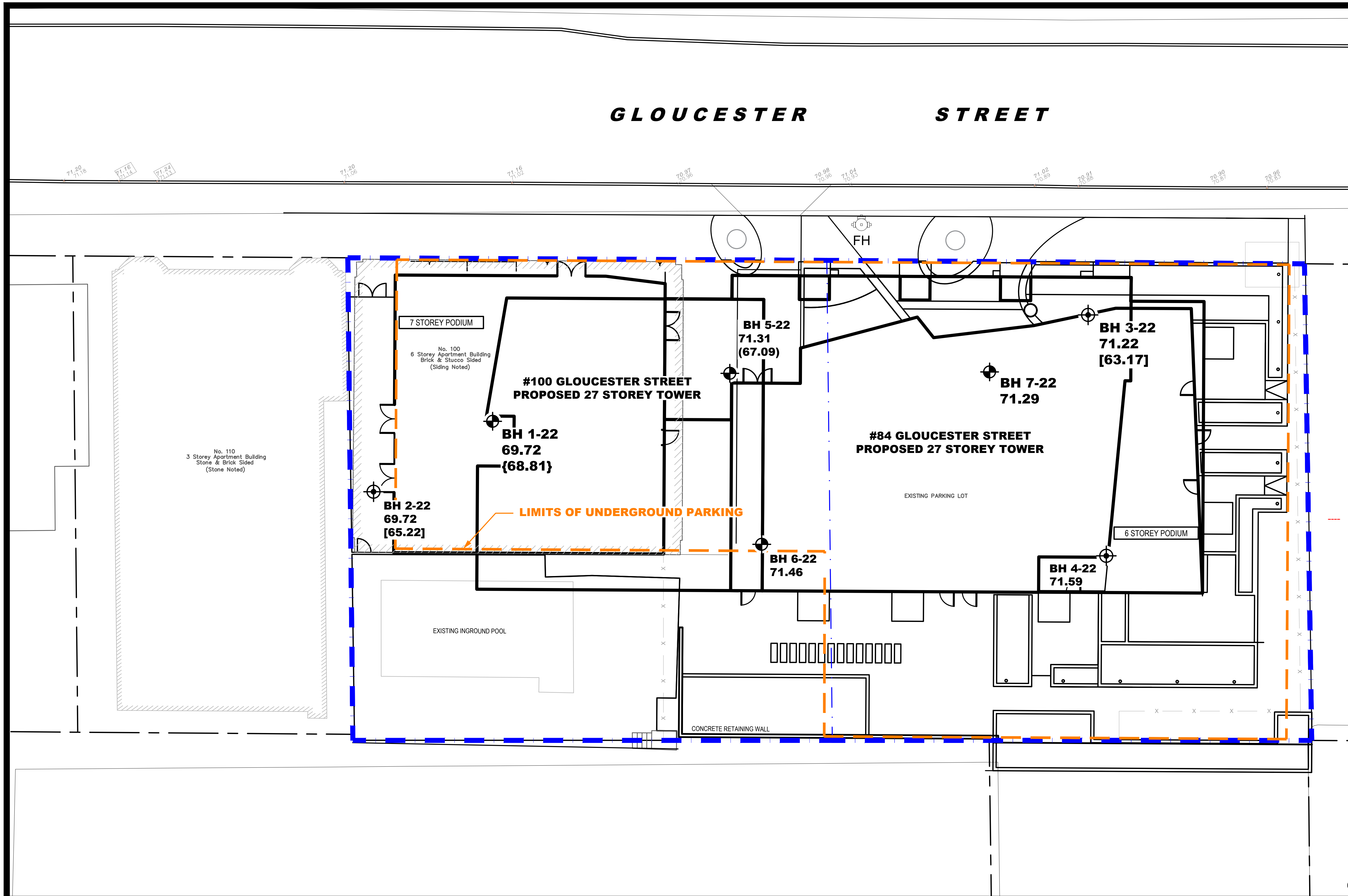
FIGURE 1 – KEY PLAN

DRAWING PG6351-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⌖	BOREHOLE LOCATION
⌖	MONITORING WELL LOCATION
69.72	GROUND SURFACE ELEVATION (m)
{68.81}	PRACTICAL SPOON REFUSAL ELEVATION (m)
(67.09)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[63.17]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH.

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

9 AURIGA DRIVE
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NO.	REVISIONS	DATE	INITIAL
1	ADDED CONCEPTUAL PLAN	26/07/2023	SD

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
84 & 100 GLOUCESTER STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 07/2022
Drawn by: NFRV
Report No.: PG6351-1
Checked by: OM
Dwg. No.: PG6351-1
Approved by: SD
Revision No.: 1

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