PATERSON GROUP

Geotechnical Investigation

Proposed High-Rise Building

110 & 116 York Street
Ottawa, Ontario

Prepared for Claridge Homes

Report PG2733-4 Revision 2 dated July 16, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed high-rise building to be located at 110 & 116 York Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a high-rise building with 4 levels of underground parking.

It is also expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was carried out between May 8th and 9th, 2024 and consisted of a total of 5 boreholes (BH 1-24 through BH 5-24), advanced to a maximum depth of 10.4 m below the existing surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. Previous investigations by Paterson also included 4 boreholes (BH 4-23, BH 6-23, BH 7, and BH 8) at this site, extending to a maximum depth of 15.2 m.

A previous investigation was conducted by others in 2018 at 116 York Street and consisted of 4 boreholes (BH 1 through BH 4) advanced to a maximum of 11 m.

The locations of the boreholes are illustrated on Drawing PG2733-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two-person crew. The field work for the boreholes by Paterson was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes either by sampling directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. The depths at which the auger, split-spoon, and rock core samples were recovered from the test holes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

All samples were visually inspected and initially classified on site. The auger, split-spoon samples were placed in sealed plastic bags, and rock core samples were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification.

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.

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

The subsurface conditions observed in all test holes 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, in Appendix 1.

Groundwater

Monitoring wells and standpipe piezometers were installed in select boreholes during the current and previous investigations to permit monitoring of the groundwater levels subsequent to the completion of the drilling and sampling. The measured groundwater levels are presented in the Soil Profile and Test Data Sheets in Appendix 1, and are discussed further in Section 4.3.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS unit with respect to a geodetic datum. The borehole locations and elevations are presented on Drawing PG2733-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing by others 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 consists of two contiguous properties, 110 & 116 York Street. The 110 York Street property was occupied by a commercial building, which was recently demolished, along with an asphalt-paved access lane. The 116 York Street property was occupied by an asphalt-paved parking lot.

The site is bordered to the north by York Street, to the west by commercial buildings, to the south by an asphalt-paved parking lot, and to the east by a multi-storey building. The site is generally level at-grade with adjacent roadways at approximate geodetic elevation 60 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of an asphaltic pavement structure which is underlain by an approximate 1.0 to 3.8 m thickness of fill, and subsequently by a glacial till deposit. The fill was generally observed to consist of dark brown silty sand with varying amounts of crushed stone, gravel, brick, wood, and concrete.

Underlying the fill, the glacial till deposit was generally observed to consist of compact to very dense, brown to grey silty sand to silty clay with gravel, cobbles and boulders. The glacial till deposit was observed to extend to the bedrock surface.

Bedrock

Practical auger refusal was encountered on the bedrock surface at approximate depths ranging from 3.5 to 5.6 m. Bedrock was cored at several borehole locations, and based on the recovered rock core samples, the bedrock was observed to consist of grey limestone of good to excellent in quality in the upper 1 to 1.5 m, generally becoming excellent in quality with depth. The bedrock was cored to a maximum depth of 15.2 m at borehole BH 4-23.

4.3 Groundwater

The groundwater levels were measured on May 22, 2024 in the monitoring wells from the current investigation.

The measured groundwater levels are presented in Table 1 below.

Table 1 – Summary of Groundwater Level Readings				
Test Hole ID	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date Recorded**
		Depth (m)	**Elevation (m)**	
BH 1-24	60.78	3.46	57.32	May 22, 2024
BH 3-24	60.93	6.15	54.78	May 22, 2024
BH 4-24	61.33	6.29	55.04	May 22, 2024
Notes: Ground surface elevations at monitoring well and piezometer locations were surveyed by Paterson and referenced to a geodetic datum.				

Based on the measured groundwater levels, the long-term groundwater level is estimated to range between approximately 2.5 to 4.5 m below the existing ground surface. However, it should be noted that groundwater levels are subject to seasonal fluctuations; therefore, it could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the excavation for 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 fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the anticipated founding level for the proposed high-rise building, all existing overburden material will be excavated from within the proposed building footprint.

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

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming, where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

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

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations could be 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 the 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 or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, 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 current construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these criteria. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. As noted above, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

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

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

If excavated bedrock 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 this fill material is open-graded, 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.

Lean Concrete Filled Trenches

Where rock overbreak occurs at the underside of footing (USF) elevation, lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate the subgrade from the bedrock surface to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement 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 will suffice in providing a direct transfer of the footing load to the underlying bedrock.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded limestone 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 clean, surface sounded bedrock, 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 flatter).

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site designation for the proposed building in accordance with the Ontario Building Code (OBC) 2024. The seismic shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic shear wave velocity testing location is presented in Drawing PG2733-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound head blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between five (5) to ten (10) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3, 4.5 and 16.5 m away from the first geophone, 3, 4.5 and 12 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the seismic shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

It is anticipated that the proposed building will be founded directly on the bedrock surface. Based on the testing results, the bedrock shear wave velocity is **2,860 m/s**.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation provided in OBC 2024, and as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2,860m/s}\right)}$$

$$V_{s30} = 2,860m/s$$

Based on the results of the seismic testing, the average shear wave velocity, $Vs_{30,}$ for foundations placed on bedrock is **2,860 m/s**. Therefore, a **Site Designation $X_{2,860}$** is applicable for design of the proposed building founded on bedrock, as per OBC 2024. Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction.

5.5 Basement Floor Slab

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

It is anticipated that the underground levels for the proposed building will be mostly parking, and the recommended pavement structures noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level slab of the proposed building. 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 bulk (drained) unit weight of 20 kN/m^3. Where undrained conditions are anticipated (i.e. below groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3.

However, it is understood that the majority of the basement walls are to be poured against a composite drainage board, which will be placed against the exposed bedrock face. A nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). Further, a seismic earth pressure component will not be applicable for the foundation walls which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

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

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

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

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

5.7 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.

It should be further noted that the centre to centre spacing between bond lengths be at least four (4) times the diameter of the anchor holes and greater than one fifth (1/5) of the total anchor length or a minimum of 1.2 m to decrease the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post-tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, and a free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, then therefore, less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

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

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 40 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 on the next page:

Table 2 - Parameters used in Rock Anchor Design	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3.

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	1.0	0.6	1.6	230
	1.5	1.0	2.5	350
	2.5	1.7	4.2	575
125	0.8	0.6	1.4	300
	1.3	0.9	2.2	500
	2.2	1.4	3.6	850

Other Considerations

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

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The following rigid pavement structure is recommended.

Table 4 - Recommended Rigid Pavement Structure – Underground Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 – 32MPa 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 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 underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Pavement Structure Over Podium Deck

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 over the top of the podium structure, should they be required.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas Over Podium	
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 Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

Table 6 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Loading Areas Over Podium	
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
300*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

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 existing pavement structures. 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 40 (HI-40). The base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed 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.

Other Considerations

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 vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that the proposed building foundation walls be blind-poured and placed against a composite drainage board which is fastened to the temporary shoring system or vertical bedrock face.

For the installation of the composite drainage board against the 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 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 composite drainage board, such as Delta Drain 6000 or equivalent, against the prepared vertical bedrock surface. 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 from the composite drainage board to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area. This is illustrated on the attached Figure 5 – Groundwater Infiltration Control System.

Elevators and any other pits located below the underslab drainage system should be waterproofed. This is illustrated on the attached Figure 6 – Waterproofing System for Elevator and Sump Pit.

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 building's foundation drainage system to the building's sump pit(s). For preliminary design purposes, it is recommended that 150 mm diameter perforated pipes be placed at approximate 6 m centres underlying the lowest level slab. The underslab drainage pipes should also be provided with a geosock and surrounded on all sides by a minimum 150 mm thick layer of 19 mm clear crushed stone.

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.

Transition from Foundation Wall to Podium Deck

It is anticipated that a waterproofing system will be provided for the podium deck surface. It is recommended that the podium deck waterproofing system consist of a layer of hot rubber membrane applied to the concrete surface. The concrete should be cleaned of any dust, dirt, or debris prior to the application of the hot rubber. The hot rubber should be overlain by a foundation drainage board (6000 series by DeltaDrain, G100N MiraDrain, or approved equivalent) installed with the geotextile side facing up, and further overlain by a rigid insulation, or equivalent as specified by the Architect.

The hot rubber membrane should extend to the termination bar at the top of the drainage board, and covering the cold joint at the podium deck/foundation wall interface by a minimum of 150 mm. The podium deck drainage board can then be overlapped to cover the cold joint a minimum of 150 mm. Reference should be made to Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-In Detail in Appendix 2 for further details.

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 required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

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

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

6.3 Excavation Side Slopes

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

Based on the proximity of the proposed building to the neighbouring property lines, it is expected that a temporary shoring system will be required for the majority of the proposed building excavation.

Excavation 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 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.

Temporary Shoring

Due to the anticipated depth of the proposed building excavation and the proximity of the proposed building to the abutting property boundaries, temporary shoring is anticipated to be required to support the overburden soil during the required excavation. The shoring requirements will depend on the depth of the excavation, and the proximity and depth of the adjacent buildings and underground structures.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer, who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

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

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the temporary shoring system or soils supported by the system. Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system or which could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring system be adequately supported to resist toe failure by means of rock bolts or extending into the bedrock through pre-augered holes if a shoulder pile and lagging system is used.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 7 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m^3	21
Submerged Unit Weight , kN/m^3	13

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

The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater table.

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

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

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, shotcrete and/or chain link fencing connected to the excavation face may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for temporary rock anchors, shotcrete, and/or chain link fencing should be evaluated during the excavation operations by Paterson and should be discussed with the structural engineer during the design stage of the project.

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.

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

6.5 Groundwater Control

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.

Impacts on Neighbouring Properties

A silty clay deposit was not encountered at the subject site, which is typically the soil type which is, in some cases, susceptible to settlement from groundwater lowering. Therefore, no issues are expected with respect to groundwater lowering that could 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 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 in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above the frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland Cement would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity in indicative of a slightly to moderately aggressive corrosive environment.

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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, if required.
- ❑ Review of the bedrock stabilization and excavation requirements.
- ❑ Review of the proposed groundwater infiltration control system, foundation drainage 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.

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

8.0 Statement of Limitations

The recommendations provided herein 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 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.

LICENSED PROFESSIONAL ENGINEER
July 16, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Puneet Bandi. M.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Claridge Homes (1 digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
116 York Street
Ottawa, Ontario

EASTING: 368143.796 **NORTHING:** 5032427.793 **ELEVATION:** 60.78

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 8, 2024

FILE NO. PG2733

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	60.78
FILL: Compact brown silty sand with crushed concrete and stone, trace brick, metals, wood	AU	1				
	SS	2	21	10	1	59.78
	SS	3	42	4	2	58.78
2.29						
FILL: Stiff to very stiff brown silty clay with some sand, curshed stone and concrete, trace cobbles, brick, wood, metals	SS	4	25	9	3	57.78
	SS	5	54	31		
3.81						
GLACIAL TILL: Very dense grey sandy silt with gravel, cobbles and boulders	SS	6	29	+50	4	56.78
4.42						
BEDROCK: Excellent quality grey limestone bedrock with interbedded shale seams	RC	1	100	97	5	55.78
					6	54.78
	RC	2	100	100	7	53.78
	RC	3	100	100	8	52.78
					9	51.78
	RC	4	100	100		
10.03					10	50.78
End of Borehole						
(GWL @ 3.46 m - May 22, 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

STRATA PLOT

MONITORING WELL CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
116 York Street
Ottawa, Ontario

EASTING: 368156.266 **NORTHING:** 5032440.634 **ELEVATION:** 60.38

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 8, 2024

FILE NO. PG2733

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	60.38
ASPHALT	0.10						
FILL: Compact granualr with crushed stone and gravel	0.20	AU	1				
FILL: Compact brown silty sand with some clay, trace graveland crushed stone	1.07	SS	2	29	5	1	59.38
GLACIAL TILL: Dense to very dense brown silty sand with clay, trace gravel, occasional cobbles and boulders	3.96	SS	3	63	15	2	58.38
		SS	4	42	16	3	57.38
		SS	5		49		
End of Borehole							

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

STRATA PLOT

PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
116 York Street
Ottawa, Ontario

EASTING: 368157.789 **NORTHING:** 5032440.535 **ELEVATION:** 60.35
DATUM: Geodetic
REMARKS:
BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 8, 2024

FILE NO. PG2733
HOLE NO. BH 2A-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 %	PIEZOMETER CONSTRUCTION
Ground Surface								20 40 60 80	
Inferred **OVERBURDEN**						0	60.35		
						1	59.35		
						2	58.35		
						3	57.35		
						4	56.35		
		SS	6	17	+50				
		SS	7		+50				
5.18									
End of Borehole									

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
116 York Street
Ottawa, Ontario

EASTING: 368178.7 **NORTHING:** 5032404.654 **ELEVATION:** 60.93

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 8, 2024

FILE NO. PG2733

HOLE NO. BH 3-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	60.93
FILL: Brown silty sand with gravel and crushed stone, occasional concrete, brick and wood, trace clay		AU	1				
		SS	2	17	2	1	59.93
1.83		SS	3	21	6		
GLACIAL TILL: Dense to very dense brown sandy silt with some gravel, occasional cobbles and boulders - Sandy pocket @ 2.29 m						2	58.93
		SS	4	25	32		
		SS	5	29	+50	3	57.93
		RC	1	21	0	4	56.93
5.11		SS	6		+50	5	55.93
BEDROCK: Excellent quality grey limestone bedrock with interbedded shale seams		RC	2	83	100	6	54.93
		RC	3	100	100	7	53.93
		RC	4	100	100	8	52.93
						9	51.93
10.26		RC	5	100	100	10	50.93
End of Borehole (GWL @ 6.15 m - May 22, 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
116 York Street
Ottawa, Ontario

EASTING: 368171.728 **NORTHING:** 5032397.855 **ELEVATION:** 61.33

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 9, 2024

FILE NO. PG2733

HOLE NO. BH 4-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
ASPHALT	0.03	AU	1		
FILL: Granular, light brown silty sand with gravel and crushed stone	0.20				
FILL: Firm dark brown silty clay with sand and gravel, some crushed stone, trace topsoil	1.22	SS	2	54	6
GLACIAL TILL: Compact to very dense brown sandy silt, gravel, occasional cobbles and boulders	4.95	SS	3	38	18
		SS	4	13	29
		SS	5	50	12
		SS	6	50	+50
		SS	7	29	+50
BEDROCK: Good to excellent quality grey limestone bedrock with interbedded shale seams	10.34	RC	1	100	67
		RC	2	100	86
		RC	3	100	100
		RC	4	100	8
End of Borehole					

(GWL @ 6.29 m - May 22, 2024)

DEPTH (m)	ELEV. (m)
0	61.33
1	60.33
2	59.33
3	58.33
4	57.33
5	56.33
6	55.33
7	54.33
8	53.33
9	52.33
10	51.33

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
116 York Street
Ottawa, Ontario

EASTING: 368165.269 **NORTHING:** 5032418.67 **ELEVATION:** 60.79

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** May 9, 2024

FILE NO. PG2733

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	60.79
ASPHALT	0.05						
FILL: Granular, brown silty sand with gravel and crushed stone	0.23	AU	1				
FILL: Firm brown silty sand with clay, gravel, crushed stone	1.07	SS	2	54	5	1	59.79
GLACIAL TILL: Compact to dense brown silty sand with clay and gravel, occasional cobbles and boulders		SS	3	63	29	2	58.79
		SS	4	38	+50	3	57.79
	3.76	SS	5	54	20		
End of Borehole Practical refusal to augering @ 3.76 m							

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

Supplemental Geotechnical Investigation
137 & 141 George Street and 110 York Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE February 27, 2023

FILE NO. PG2733

HOLE NO. BH 4-23

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	61.03
Asphaltic concrete	0.05	AU	1				
FILL: Crushed stone with sand	0.25	AU	2				
FILL: Brown sand - some crushed stone by 0.6m depth	1.45	SS	3	25	18	1	60.03
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	4.65	SS	4	46	24	2	59.03
		RC	1	35		3	58.03
						4	57.03
BEDROCK: Excellent quality, grey limestone	15.16	RC	2	100	93	5	56.03
		RC	3	100	100	6	55.03
						7	54.03
		RC	4	100	100	8	53.03
						9	52.03
		RC	5	100	100	10	51.03
		RC	6	100	100	11	50.03
						12	49.03
		RC	7	100	100	13	48.03
		RC	8	100	100	14	47.03
						15	46.03
End of Borehole							
(GWL not available - Mar. 8, 2023)							

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

Supplemental Geotechnical Investigation
137 & 141 George Street and 110 York Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** February 28, 2023

FILE NO. PG2733

HOLE NO. BH 6-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.08
Asphaltic concrete 0.06		AU	1				
FILL: Dark brown sand with crushed stone		SS	2	54	18	1	61.08
		SS	3	58	26	2	60.08
		SS	4	58	19		
3.58		SS	5	70	50+	3	59.08
Reinforced concrete slab (former crane base)		RC	1	100		4	58.08
5.18						5	57.08
BEDROCK: Excellent quality, grey limestone		RC	2	100	95	6	56.08
7.47		RC	3	100	100	7	55.08
End of Borehole							
(GWL @ 2.77m - March 8, 2023)							

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
110 York St., 321 Dalhousie St. & 167-141 George St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 8, 2012

FILE NO. PG2733

HOLE NO. BH 7

SOIL 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	61.02		
Asphaltic concrete 0.10									
		AU	1						
		SS	2	17	11	1	60.02		
FILL: Brown silty sand with gravel, cobbles, concrete, asphalt, trace organics		SS	3	25	17	2	59.02		
		SS	4	75	26				
3.20						3	58.02		
		SS	5	21	21				
GLACIAL TILL: Grey-brown silty clay with sand, gravel, cobbles		SS	6	29	23	4	57.02		
4.70		SS	7	80	50+				
End of Borehole Practical refusal to augering at 4.70m depth									

Water Content %: 20 40 60 80

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
110 York St., 321 Dalhousie St. & 167-141 George St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 8, 2012

FILE NO. PG2733

HOLE NO. BH 8

SOIL 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	61.20		
Asphaltic concrete 0.10		AU	1						
FILL: Brown silty sand, trace clay and gravel		SS	2	29	11	1	60.20		
1.83		SS	3	58	30	2	59.20		
GLACIAL TILL: Brown to grey silty clay with sand, gravel, cobbles		SS	4	46	69	3	58.20		
3.53		SS	5	52	59				
End of Borehole Practical refusal to augering at 3.53m depth									

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

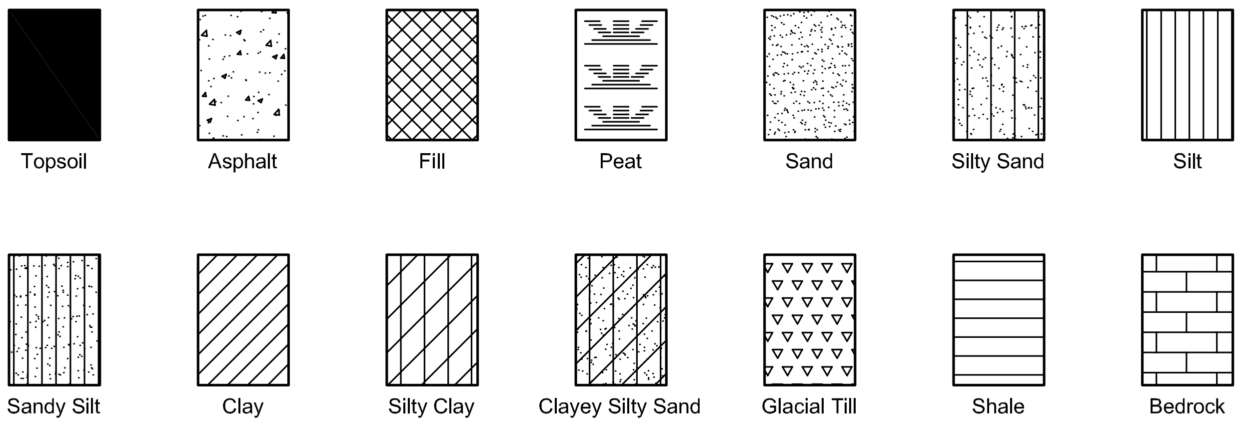
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

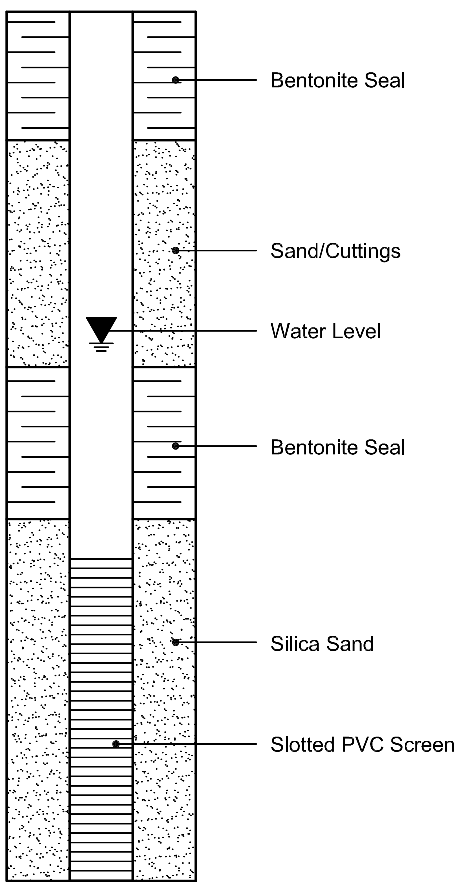
SYMBOLS AND TERMS (continued)

STRATA PLOT

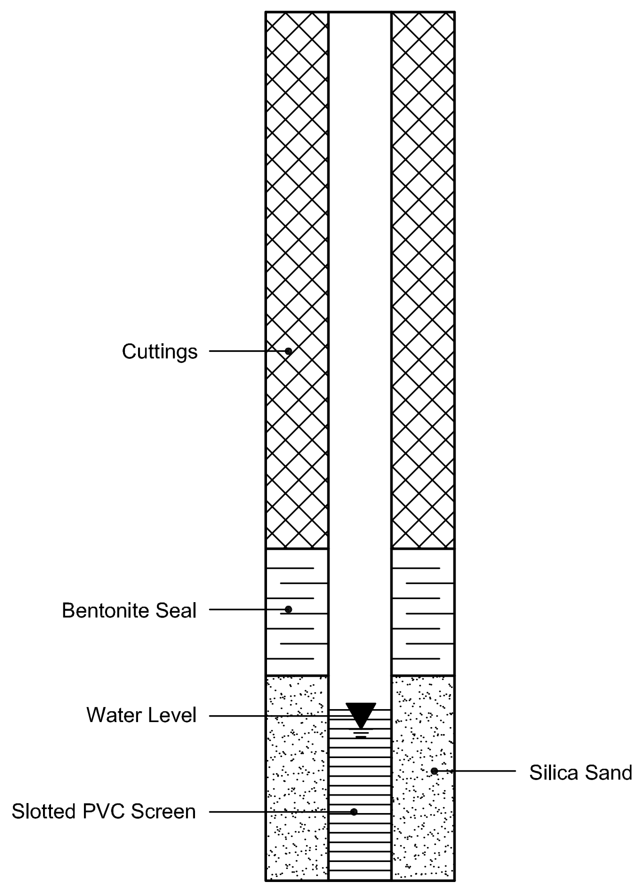


MONITORING WELL AND PIEZOMETER CONSTRUCTION

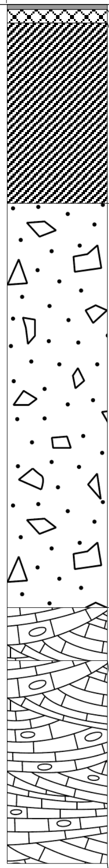
MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Project:	High Rise Building		Yuri Mendez Engineering
Location: 116 York St.	Client: Byview Ottawa Holdings Ltd.		Test Hole No.: BH1 of 4
Job No.: 36-BOHL	Test Hole Type: 8" hollow stem augers		Date: June 23 and 24, 2018
Standard SPT and 50mm rock cores	SPT Hammer Type: Safety auto hammer		Logged By: Yuri Mendez



Depth (m)	Elevation (m) 100.53	Material Description	Samples or Blows/Ft	Water	Shear Strength (kPa)	Moisture Content (%)	Rock Quality RQD %	Other Lab Tests
0 – 0.25	100.5	Asphalt						
0.25 – 0.5		Granular Fill						
0.5 – 1.5	100 – 99	Fill: brown silt-sand-clay-gravel-cobbles.						
1.5 – 4.5	99 – 96	Glacial Till: brown sand-silt-clay-boulders-cobble. Friction of 34 degrees and density of 18.5kN/m3.	39 8 9 8					
4.5 – 5.8	96 – 94.7	Limestone bedrock: 66% Rock Quality Designated (RQD) and thus of fair quality, blocky and seamy, fractured	#8				66	
5.8 – 6.48	94.7 – 94	Limestone bedrock: 88% Rock Quality Designated (RQD) and thus of good quality, massive, moderately jointed or sound	#9				88	
6.48		End of corehole at 6.48 m depth						

Depth scale (m): 0, 0.25, 0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 2.25, 2.5, 2.75, 3, 3.25, 3.5, 3.75, 4, 4.25, 4.5, 4.75, 5, 5.25, 5.5, 5.75, 6, 6.25, 6.5, 6.75, 7, 7.25, 7.5, 7.75, 8, 8.25, 8.5, 8.75, 9, 9.25, 9.5, 9.75, 10, 10.25, 10.5, 10.75, 11

Elevation scale (m): 100.5, 100, 99.5, 99, 98.5, 98, 97.5, 97, 96.5, 96, 95.5, 95, 94.5, 94, 93.5, 93, 92.5, 92, 91.5, 91, 90.5, 90

Laboratory Tests

S = Sample for lab review and moisture content

▼ Measured water level. June 30, 2018, 10:00 am

Project: High Rise Building			Yuri Mendez Engineering
Location: 116 York St.	Client: Byview Ottawa Holdings Ltd.		Test Hole No.: BH2 of 4
Job No.: 36-BOHL	Test Hole Type: 8" hollow stem augers		Date: June 23 and 24, 2018
Standard SPT and 50mm rock cores	SPT Hammer Type: Safety auto hammer		Logged By: Yuri Mendez

Depth (m)	Elevation (m) 100.31	Lithology and color	Material Description	Samples or Blows/Ft	Water	Elevation (m) 100.31	Depth (m)	Shear Strength (kPa)	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Rock Quality RQD %	Laboratory Tests: Other Lab Tests
0 – 0.25	100		Asphalt			100	0 – 0.25				
0.25 – 0.5			Granular Fill				0.25 – 0.5				
0.5 – 2.4	99.5 – 98		Fill: brown silt-sand-clay-gravel-cobbles.	7		99.5 – 98	0.5 – 2.4				
2.4 – 4.5	98 – 96		Glacial Till: brown sand-silt-clay-boulders-cobble (0.46 m core extracted from boulder). Friction of 34 degrees and density of 18.5kN/m3.	24; >100	▼ (approx. 2.85 m)	98 – 96	2.4 – 4.5				
4.5 – 8.7	96 – 91.5		Limestone bedrock: 100% Rock Quality Designated (RQD) and thus of Excellent quality, intact very sound.	#6; #7; #8		96 – 91.5	4.5 – 8.7			100; 100; 100	
8.7 – 9.47	91.5 – 90.84		As above: Unconfined Compressive Strength (UCS) = 142 and 107 MPa	#9		91.5 – 90.84	8.7 – 9.47			100	
9.47			End of corehole at 9.47 m depth				9.47				

S = Sample for lab review and moisture content

▼ Measured water level. June 30, 2018, 10:00 am

Project:	High Rise Building		Yuri Mendez Engineering
Location: 116 York St.	Client: Byview Ottawa Holdings Ltd.		Test Hole No.: BH3 of 4
Job No.: 36-BOHL	Test Hole Type: 8" hollow stem augers		Date: June 23 and 24, 2018
Standard SPT and 50mm rock cores	SPT Hammer Type: Safety auto hammer		Logged By: Yuri Mendez

Depth (m)	Elevation (m) 100.9	Lithology and color	Material Description	Samples or Blows/Ft	Water	Elevation (m) 100.9	Depth (m)	Shear Strength (kPa)	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Rock Quality RQD %	Laboratory Tests: Other Lab Tests
0 – 0.25	100.9		Asphalt			100.6	0.25				
0.25 – 0.5	100.5		Granular Fill				0.5				
0.5 – 2.25	100 – 99		Fill: brown silt-sand-clay-gravel-cobbles.	3		100.1 – 99.1	0.75 – 2				
2.25 – 5	98.5 – 96		Glacial Till: brown sand-silt-clay-boulders-cobble. gray at 4.8 m depth. Friction of 34 degrees and density of 18.5kN/m3.	16, 21, 19	▼	98.6 – 96.1	2.25 – 5				
5 – 6.25	95.5 – 95		Limestone bedrock: 94% Rock Quality Designated (RQD) and thus of Excellent quality, intact very sound.	#8		95.6 – 95.1	5.25 – 6			94	
6.25 – 7.06	94.5 – 94		Limestone bedrock: 100% Rock Quality Designated (RQD) and thus of Excellent quality, intact very sound.	#9		94.6 – 93.6	6.25 – 7.25			100	
			End of corehole at 7.06 m depth								

S = Sample for lab review and moisture content

▼ Measured water level. June 30, 2018, 10:00 am

Project:	High Rise Building		Yuri Mendez Engineering
Location: 116 York St.	Client: Byview Ottawa Holdings Ltd.		Test Hole No.: BH4 of 4
Job No.: 36-BOHL	Test Hole Type: 8" hollow stem augers		Date: June 23 and 24, 2018
Standard SPT and 50mm rock cores	SPT Hammer Type: Safety auto hammer		Logged By: Yuri Mendez

Depth (m)	Elevation (m) 101.12	Material Description	Samples or Blows/Ft	Water	Shear Strength (kPa)	Moisture Content (%)	Rock Quality RQD %	Other Lab Tests
0 – 0.25	101.12 – 100.87	Asphalt						
0.25 – 0.5		Granular Fill						
0.5 – 2		Fill: brown silt-sand-clay-gravel-cobbles.	26					
2 – 5		Glacial Till: brown sand-silt-clay-boulders-cobble. gray at 4.9 m depth. Friction of 34 degrees and density of 18.5kN/m3.	19 41 6					
5 – 6.5		Limestone bedrock: 94% Rock Quality Designated (RQD) and thus of Excellent quality, intact very sound.	#8				94	
6.5 – 9.6		Limestone bedrock: 100% Rock Quality Designated (RQD) and thus of Excellent quality, intact very sound.	#9 #10				100 100	
9.6 – 11		As above: Unconfined Compressive Strength (UCS) = 132 and 108 MPa	#11				100	

End of corehole at 11 m

S = Sample for lab review and moisture content

▼ Measured water level. June 30, 2018, 10:00 am

Certificate of Analysis

Client: Yuri Mendez

Client PO:

Report Date: 28-Jun-2018

Order Date: 25-Jun-2018

Project Description: 116 York

Client ID:		BH3-SS6	-	-	-
Sample Date:		06/24/2018 09:00	-	-	-
Sample ID:		1826064-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.95	-	-	-
Resistivity	0.10 Ohm.m	49.5	-	-	-
Anions					
Chloride	5 ug/g dry	282	-	-	-
Sulphate	5 ug/g dry	151	-	-	-

APPENDIX 2

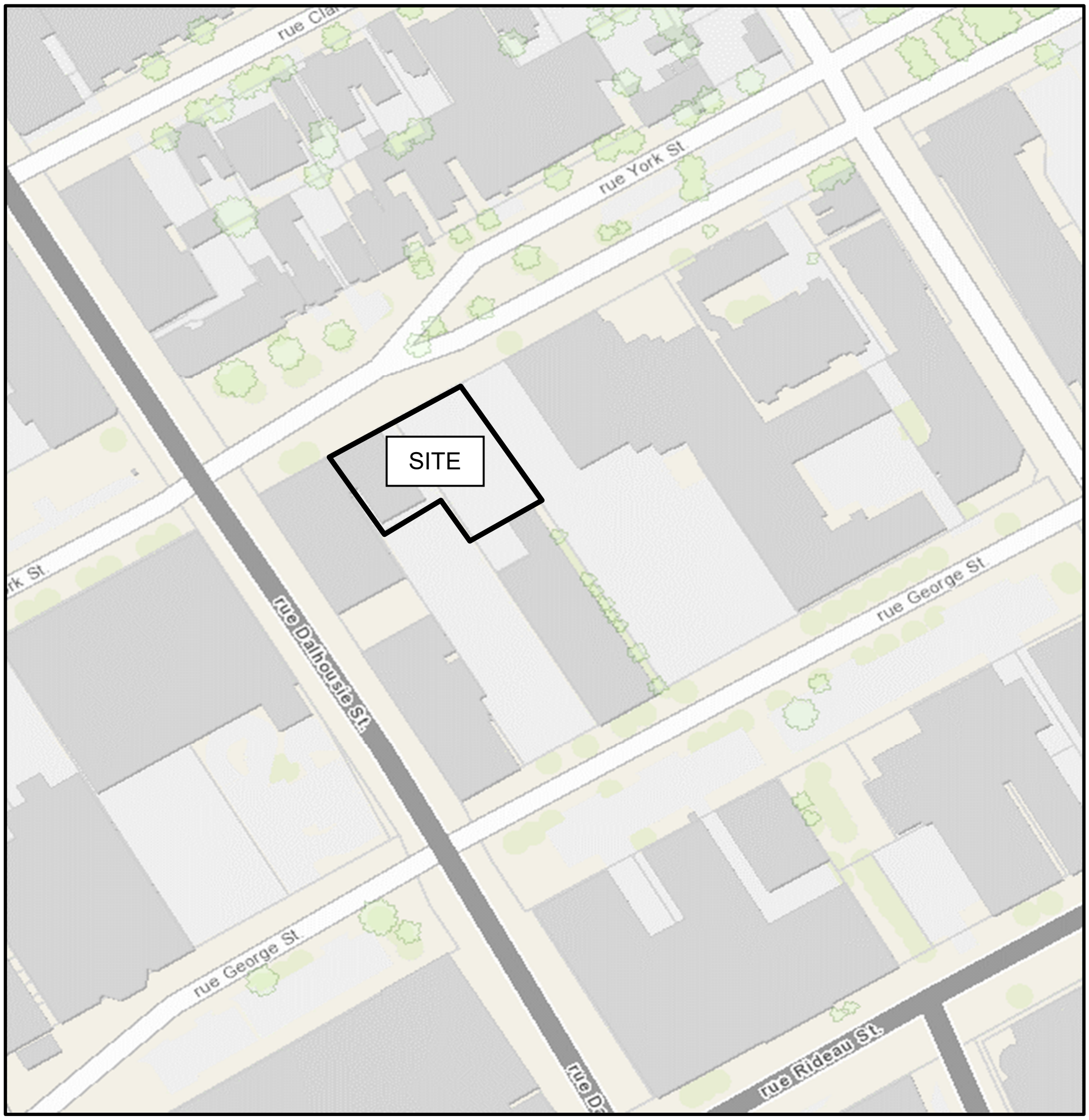


FIGURE 1

KEY PLAN

PATERSON GROUP

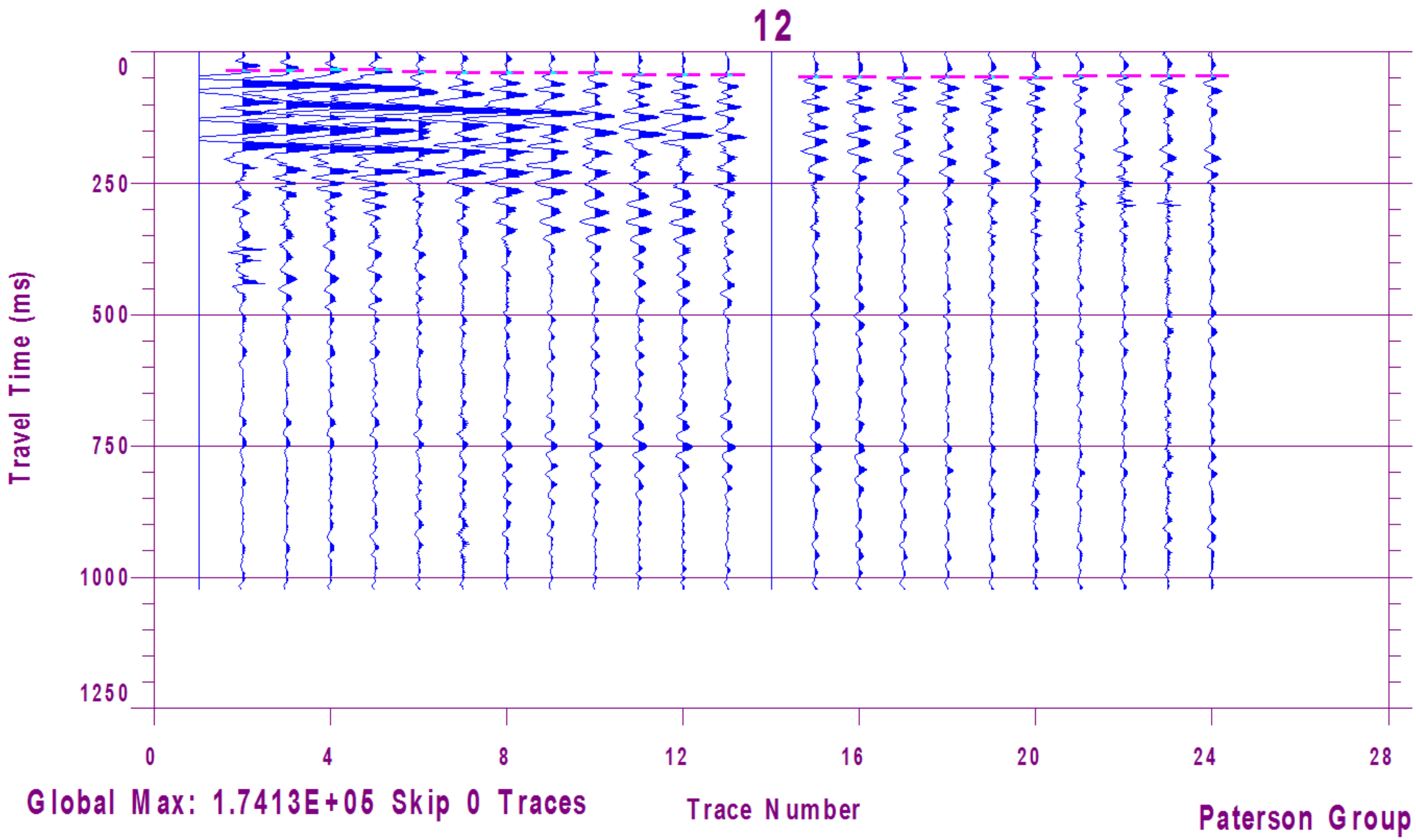


Figure 2 – Shear Wave Velocity Profile at Shot Location – 16.5 m

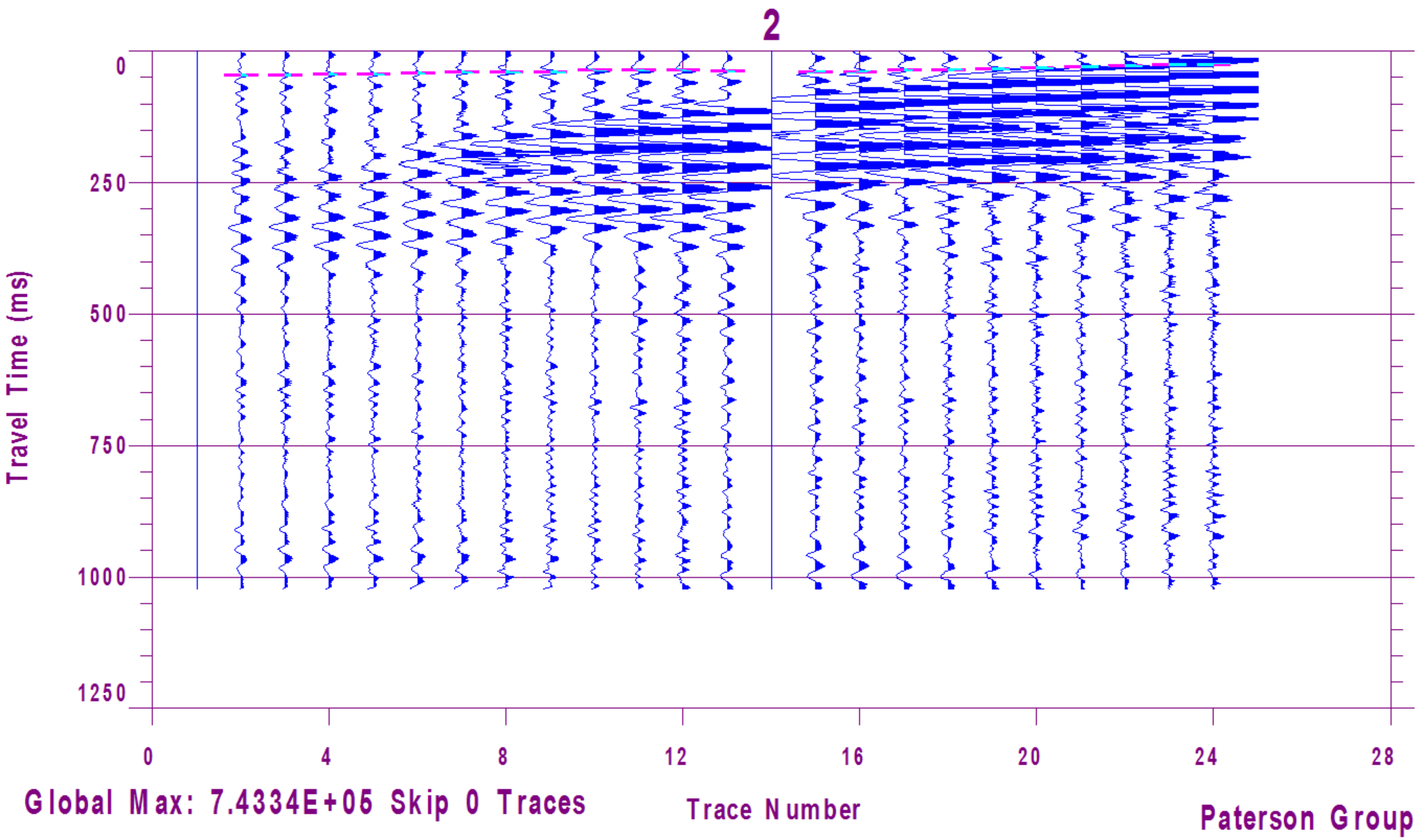
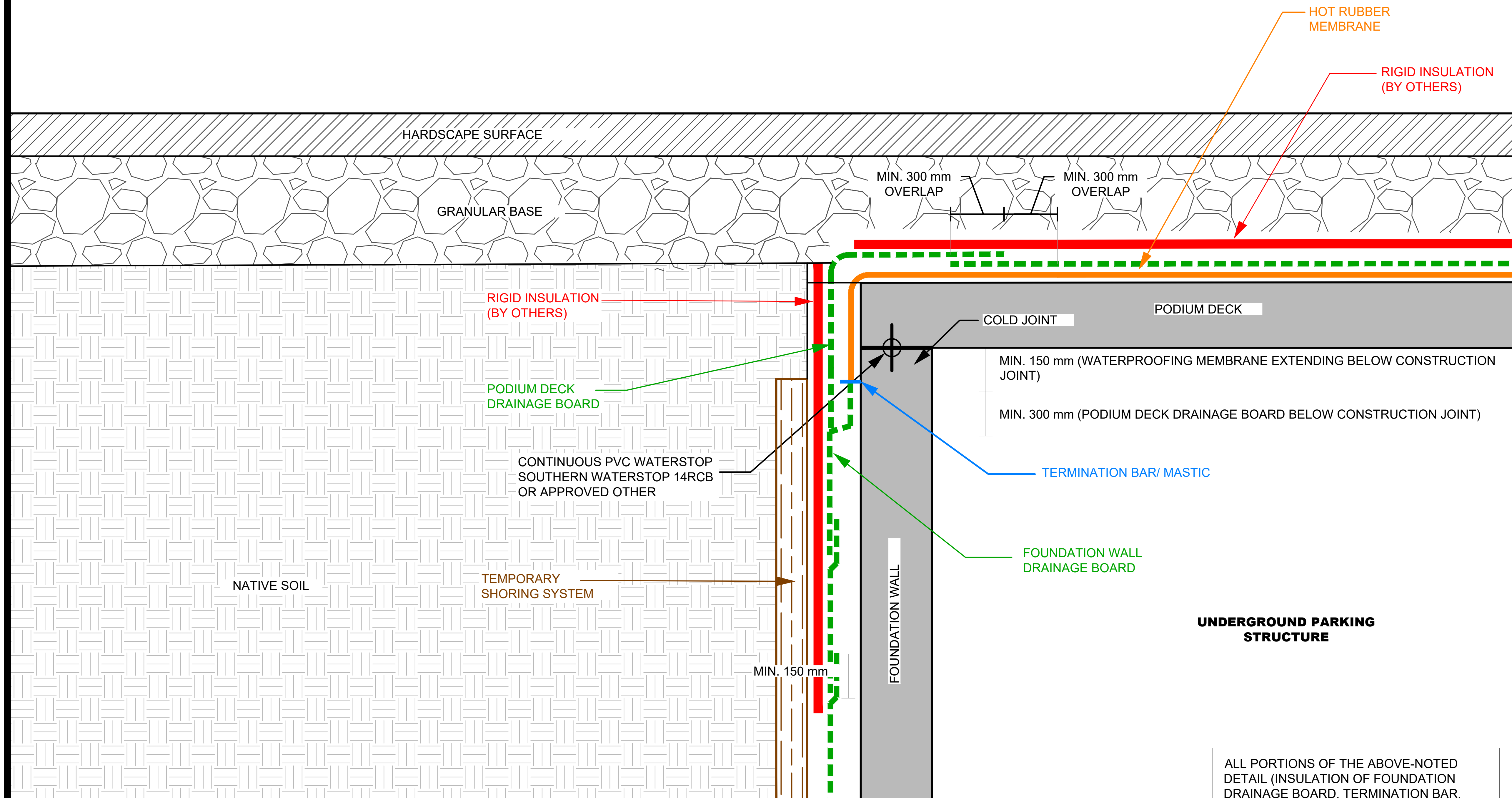


Figure 3 – Shear Wave Velocity Profile at Shot Location 58 m



ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB AND FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER, AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

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

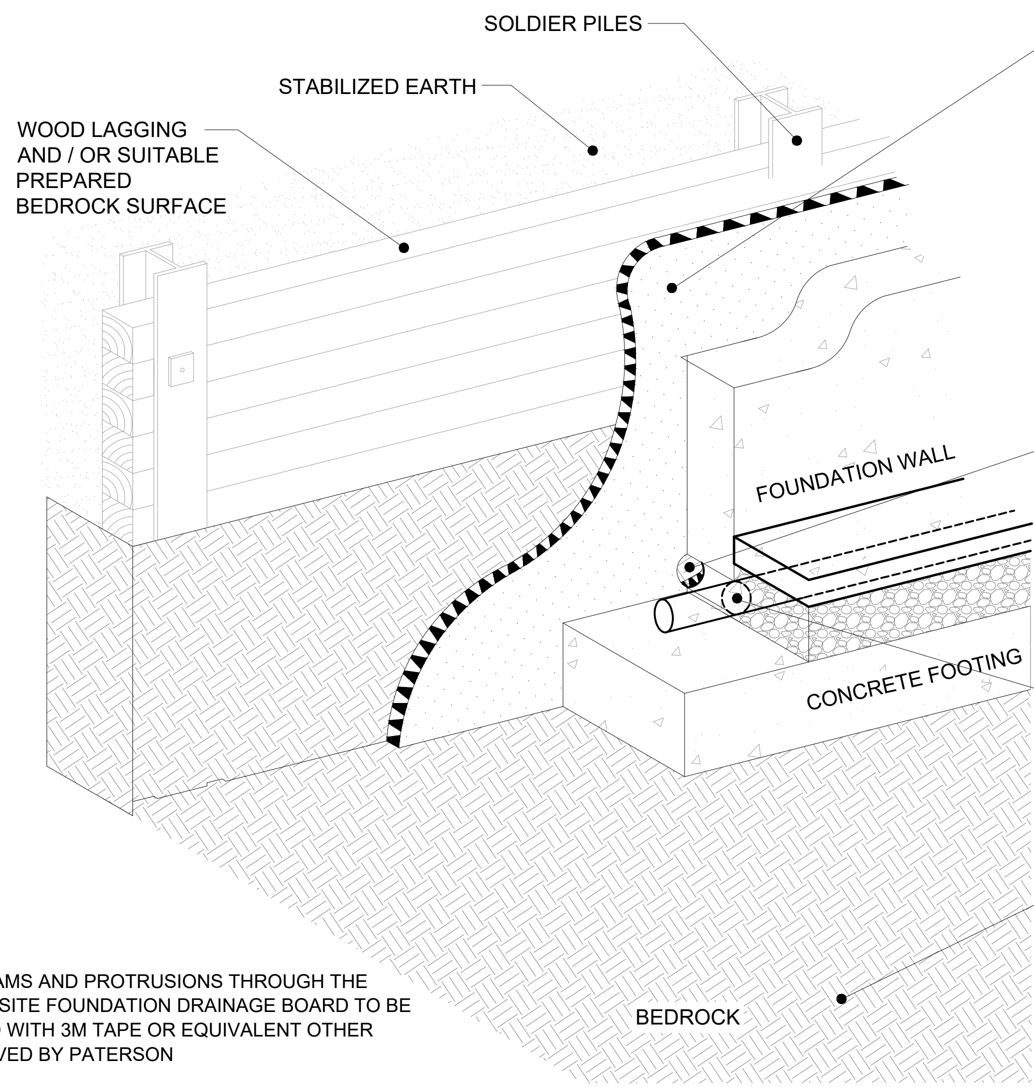
NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDING
116, 110 YORK STREET
OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale:	N.T.S	Date:	06/2024
Drawn by:	GK	Report No.:	PG2733-4
Checked by:	SD	Dwg. No.:	FIGURE 4
Approved by:	SD	Revision No.:	

p:\autocad drawings\geotechnical\pg27xx\pg2733\pg6263-fig 4-podium drainage tie-in detail.dwg



COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE.

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEAOTECHNICAL CONSULTANT.

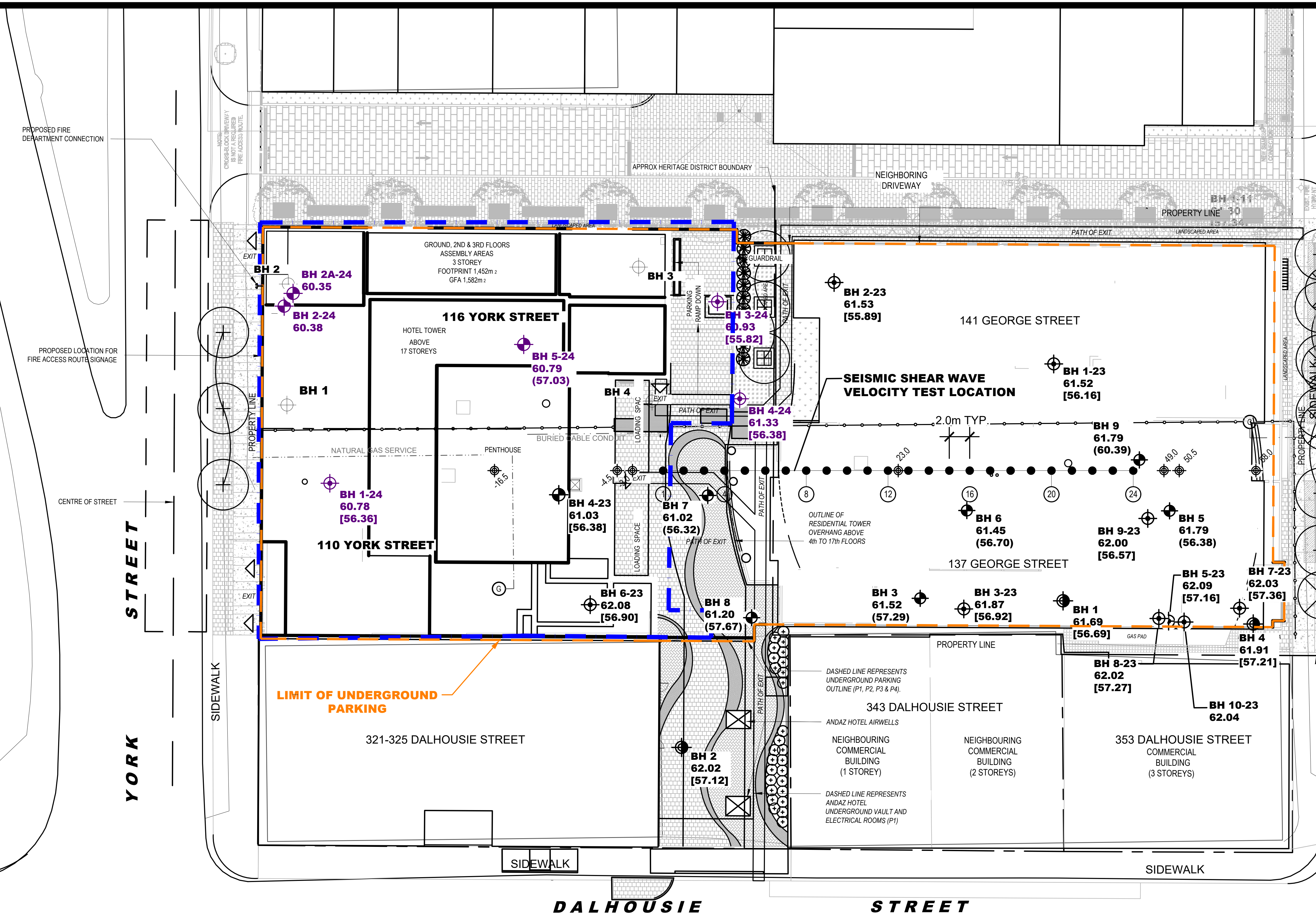
ALL SEAMS AND PROTRUSIONS THROUGH THE COMPOSITE FOUNDATION DRAINAGE BOARD TO BE SEALED WITH 3M TAPE OR EQUIVALENT OTHER APPROVED BY PATERSON

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

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPSOED HIGH-RISE BUILDING
116, 110 YORK STREET
OTTAWA, ONTARIO

Title:
GROUNDWATER INFILTRATION CONTROL SYSTEM

Scale: NTS	Date: 06/2024
Drawn by: YA	Report No.: PG2733-4
Checked by: SD	Drawing No.: FIG.5
Approved by: SD	Revision No.:



LEGEND:

- BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION, PREVIOUS INVESTIGATION, 2023
- BOREHOLE LOCATION PREVIOUS INVESTIGATION, 2023
- BOREHOLE WITH MONITORING WELL LOCATION, PREVIOUS INVESTIGATION, 2012
- BOREHOLE WITH MONITORING WELL LOCATION, PREVIOUS INVESTIGATION, 2011
- BOREHOLE LOCATION, BY OTHERS, 2018

61.52 GROUND SURFACE ELEVATION (m)

[56.16] BEDROCK SURFACE ELEVATION (m)

(57.29) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

GEOPHONE LOCATIONS

20 GEOPHONE NUMBER

3.0 SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECT(E)S.

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

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
2	UPDATED CONCEPTUAL PLAN	16/07/2025	PB
1	UPDATED CONCEPTUAL PLAN	04/07/2025	PB

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDINGS
110 AND 116 YORK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 09/2024
Drawn by: GK	Report No.: PG2733-4
Checked by: PB	Dwg. No.: PG2733-1
Approved by: SD	Revision No.: 2

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11x17