PATERSON GROUP

Geotechnical Investigation

Proposed Development

1867 Alta Vista Drive
Ottawa, Ontario

Prepared for TC United

Report PG7401-1 Revision 4 dated August 21, 2026

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1.0 Introduction

Paterson Group (Paterson) was commissioned by TC United to conduct a geotechnical investigation for the proposed development to be located at 1867 Alta Vista Drive in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

This 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 conceptual plan, it is understood that the proposed development at the subject site will consist of two multi-storey buildings with 1 to 2 levels of underground parking. At finished grades, the proposed buildings will be surrounded by asphalt-paved access lanes and landscaped amenity areas.

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

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current field program for the geotechnical investigation was conducted on July 10, 2026, and consisted of advancing 4 boreholes to a maximum depth of 10.6 m below the existing ground surface. The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration the historic borehole locations, site features and underground utilities.

Paterson conducted a previous field investigation on December 20, 2024 which consisted of advancing 3 boreholes to a maximum depth of 7.8 m and 4 test pits to a maximum depth of 3.1 m. Further, a previous investigation at the subject site was conducted by others on September 19 and 20, 2022, and consisted of 8 boreholes advanced to a maximum depth of 6.2 m below the existing ground surface.

The test hole locations are presented on Drawing PG7401-1 – Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

The soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the drill auger flights (AU). Rock cores were obtained using 47.6 mm inside diameter coring equipment. Grab samples (G) were collected at selected intervals from the test pit sidewalls. The samples were initially classified on site, placed in sealed plastic bags or core boxes, and transported to our laboratory.

The depths at which the drill auger, split-spoon, rock core, and grab samples were recovered from the boreholes and test pits are shown as AU, SS, RC and G, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The subsurface conditions observed in the 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.

Groundwater

Groundwater monitoring wells were installed in boreholes BH 2-26 and BH 3-24 to permit long-term groundwater monitoring subsequent to the field investigation. Flexible polyethylene standpipes were installed in the remaining boreholes to permit further groundwater measurement.

Typical monitoring well construction details are described below:

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

The groundwater observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1. Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations, and ground surface elevation at each test hole location, were surveyed by Paterson using a high precision GPS unit and referenced to a geodetic datum. The locations of the test holes are presented on Drawing PG7401-1 – 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. Unconfined compressive strength testing was carried out by Paterson on bedrock samples from borehole BH 1-24. The results of the testing are discussed in Section 4.2 and are provided in Appendix 1.

Soil samples and bedrock cores will be stored for a period of 1 month after this report is completed. They will then be discarded, unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant, but was historically occupied by a 2-storey building which was demolished around 2017. The north and south portions of the site are mainly covered in asphalt pads used for vehicle parking. The ground surface slopes downward gradually from south to north, from approximate geodetic elevation 74.3 m to 72.3 m.

A creek was observed to be running north-south adjacent to the eastern boundary of the site. The creek corridor was observed to be approximately 1 to 2 m wide, 2 to 3 m deep, and surrounded by dense vegetation and mature trees.

This parcel is generally bordered by commercial buildings to the north and south, Alta Vista Drive to the west, and densely vegetated vacant land to the east.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the test holes consists of a layer of fill underlain by glacial till and bedrock. The fill was generally observed to consist of brown silty sand with traces of clay, gravel, brick, and metal fragments, extending to approximate depths ranging between 0.9 to 5 m below the existing ground surface.

The glacial till was encountered immediately underlying the fill, and was observed to consist of dense to very dense, brown silty sand to sandy silt with gravel, cobbles, and boulders.

Refusal to augering was generally encountered at depths of about 4.3 m to 6.2 m below existing ground surface.

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

Bedrock

Bedrock was cored at boreholes BH 1-26, through BH 4-26, BH 1-24 and BH 2-24, and was observed to consist of shale interbedded with limestone, and based on the recovered bedrock core, was generally weathered and of poor quality to

approximate depths ranging from 4.6 to 9.1 m. The bedrock was noted to improve from poor to fair or good in quality at boreholes BH 2-26, BH 3-26, BH 4-26. At boreholes BH 1-26, through BH 4-26, BH 1-24 and BH 2-24, the bedrock was cored to depths ranging from 7.6 to 10.6 m below the existing ground surface.

From available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Carlsbad formation, with an overburden drift thickness of 2 to 3 m.

Unconfined Compressive Strength Testing of Bedrock Core Samples

The bedrock cores in borehole BH 1-24 were tested for their unconfined compressive strength. The results of the testing are summarized in Table 1 below and are also presented on the Unconfined Compressive Strength Testing Results sheets in Appendix 1.

Table 1 – Summary of Unconfined Bedrock Compressive Strength Testing Results

Borehole	Sample	Test Core Depth (m)	Unconfined Compressive Strength (MPa)
BH 1-24	RC2	7.05 - 7.20	27
BH 1-24	RC2	7.20 - 7.35	39

4.3 Groundwater

Groundwater levels for the current investigation were measured on July 16, 2026, in the monitoring wells and flexible standpipe piezometers installed at the borehole locations. The measured groundwater level readings are presented in Table 1 below, and are also shown on the Soil Profile and Test Data sheets in Appendix 2.

Table 2 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-26	73.44	2.43	71.01	July 16, 2026
*BH 2-26	73.25	3.15	70.10	
BH 3-26	74.11	2.22	71.89	
BH 4-26	74.30	4.70	69.60	
*BH 3-24	74.01	1.83	72.18	
		1.70	72.31	January 3, 2025

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

It should be noted that surface water can become trapped within backfilled boreholes which can result in a perched water table, leading 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 an approximate geodetic elevation of 69 to 72 m.

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 building is recommended to be founded on conventional spread footings bearing on the clean, surface sounded bedrock, or on lean concrete which is placed directly over the clean, surface sounded bedrock.

If more than 1 underground parking level is considered for the proposed development, then bedrock removal will likely be required. The above and other considerations are discussed further in the following sections.

Expansive shale bedrock could be present on site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock. This is discussed further in Section 6.7.

The above and other considerations are 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 the building, paved areas, pipe bedding, and other settlement sensitive structures.

Bedrock Removal

If bedrock removal is required, it 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 to 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 the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If non-specified backfill, reviewed and approved by Paterson, is to be placed to build up the subgrade level for areas to

be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 98% of the material's SPMDD.

Expansive shale deteriorates upon exposure to air and is not generally suitable for reuse as an engineered fill. The use of imported granular fill is recommended for this purpose.

Lean Concrete Filled Trenches

Where bedrock is encountered below the underside of footing (USF) elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete (minimum **17 MPa** 28-day compressive strength) up to the founding elevation Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings supported on clean, surface sounded bedrock, or on lean concrete which is placed directly over the clean, surface-sounded bedrock, can be designed using a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **3,000 kPa.** A geotechnical resistance factor of 0.5 was applied to this bearing resistance value.

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, or on lean concrete which is placed directly on the 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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site designation for the proposed multi-storey building in accordance with Ontario Building Code 2024 (OBC 2024). The shear wave velocity testing was completed by Paterson personnel. Two seismic shear wave velocity profiles from the testing are presented in Figure 4 and Figure 5 in Appendix 2 of the present report.

Field Program

The seismic array was located as presented in Drawing PG7401-1 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 1 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer and a hammer trigger switch attached to a 12-pound dead 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 four (4) to eight (8) 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 were 10.0, 2.0 and 1.0 away from the first and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected 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 of soil. 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.

Based on our testing results, the average overburden shear wave velocity is **523 m/s**, while the bedrock shear wave velocity is **2,091 m/s**. Further, the testing indicated the average overburden thickness to be approximately 5.5 m across the tested area.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation provided in the Ontario Building Code 2024, and as presented in the following page.

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{5.5\ m}{523\ m/s} + \frac{24.5\ m}{2{,}091\ m/s}\right)}$$

$$V_{s30=}\ 1{,}349\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity of the upper 30 m profile, Vs_{30}, was calculated to be **1,349 m/s**.

It is understood that the buildings will be founded on conventional footings with an elevation within 3 m from the bedrock surface. Therefore, a **Site Designation $X_{1,349}$** is applicable for design of the proposed structure as per OBC 2024.

The soils underlying the subject site are not considered susceptible to liquefaction or cyclic softening.

5.5 Basement Floor Slab

The existing fill, glacial till, or bedrock will provide a suitable founding medium 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).

Lateral Earth Pressures

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

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.31g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 K_o \cdot \gamma \cdot H^2$, where K = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:
$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

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, 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 about 25 and 40 MPa, which is similar in strength to 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 3 below.

Table 3 – 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 4 below. The factored tensile resistance values given in Table 4 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 4 – Recommended Rock Anchor Lengths – Grouted Rock Anchor				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	0.9	1.1	2.0	210
	1.8	1.2	3.0	420
	4.0	1.2	5.0	900
	5.0	1.0	6.0	1150
125	0.7	1.4	2.1	250
	1.2	1.7	2.9	470
	2.0	1.9	3.9	800
	3.0	2.0	5.0	1150

Other Considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter and should be flushed clean prior to grouting under inspection from geotechnical personnel. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request.

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 of the proposed building consists 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 5 below.

Table 5 - 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.

Pavement Structure Over Podium Deck

The pavement structures presented in Tables 6 and 7 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 6 - Recommended Pavement Structure - Car Only Parking Areas Over 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 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 7 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck	
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 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 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.

Pavement Structure on Overburden Soils

The following pavement structures may be considered for at-grade car only parking and heavy traffic areas, should they be required. The proposed pavement structures are shown in Tables 8 and 9, on the next page.

Table 8 - 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 9 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
450	**SUBBASE** – OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is anticipated that the proposed building foundation walls located below finished grades be blind-poured and placed against a composite drainage board and waterproofing membrane which is fastened to the temporary shoring system and/or vertical bedrock face.

For the portion of the composite drainage board and waterproofing membrane installed against vertical bedrock face, if encountered within the depth of excavation, the following is recommended:

- ❑ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).
- ❑ Mechanically remove bedrock along the foundation walls, up to approximately 150 mm from the finished vertical excavation face.
- ❑ Grind the bedrock surface up to the outer face of the line drilled holes to create a satisfactory surface for the waterproofing membrane and/or composite drainage board.
- ❑ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features of the bedrock surface, as required based on site inspection by Paterson.
- ❑ Place a composite drainage board, such as Delta Drain 6000 or equivalent, against the prepared vertical bedrock surface. The composite drainage layer should extend from the finished grade to the underside of the footing.
- ❑ Place a suitable waterproofing membrane (such as Grace Preprufe 275 or approved equivalent) over the composite drainage board. The waterproofing membrane should extend from the finished grade to the underside of the footing.
- ❑ Pour the foundation wall against the waterproofing membrane and composite drainage board.

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

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

Perimeter and Underslab Drainage System

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

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

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

Foundation Backfill

Where the foundation walls are not poured directly against the temporary shoring system, the exterior of the perimeter foundation walls can be backfilled with the site excavated soils, provided that they are maintained in an unfrozen state and at a suitable moisture content for compaction. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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

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

6.3 Excavation Side Slopes

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

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress. It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides.

It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Bedrock Stabilization

While the bedrock quality was observed to be variable across the subject site, it is expected that the upper 1 to 3 m of the bedrock consists of weathered, poor quality shale.

It is expected that bedrock stabilization measures will be needed for the upper portion of the shale, which would likely consist of fibre-reinforced shotcrete with rock bolts at an approximate 2 m by 2 m spacing. Details of the bedrock stabilization measures can be provided at the time of construction when the bedrock can be inspected on-site.

Dependent on the depth of excavation, the lower portion of the shale bedrocl is expected to be of good quality. Therefore, bedrock stabilization measures may not be required for this portion of the bedrock, although this will need to be reviewed and confirmed at the time of construction. Horizontal anchors maybe required at specific location to prevent pop-outs of the bedrock, especially in areas where bedrock fractures and weak bedding planes are conductive to the failure of the bedrock surface.

In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors. A minimum 1 m horizontal ledge should remain between the unsupported excavation and sound bedrock surface. Where sufficient space for the horizontal ledge is not available, it is recommended that a temporary concrete block retaining wall be used to retain the overburden soils.

Temporary Shoring

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

Inspections and approval of the temporary system will also be the responsibility of the designer. The geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should consider the impact of a significant precipitation event and designate design measures to ensure that precipitation will not negatively impact the shoring system or soils supported by the system.

Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

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

The earth pressures acting on the temporary shoring system may be calculated with the parameters presented in Table 10, presented below.

Table 10 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level. The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated to full weight, with no hydrostatic groundwater pressure component.

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

Bedrock Stabilization

The excavation side slopes in sound bedrock can be carried out using 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.

The requirement for bedrock stabilization measures such as temporary chainlink fencing, shotcrete, and/or rock bolts, particularly for the upper weathered bedrock, should be evaluated during the excavation operations 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.

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

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

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

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

6.5 Groundwater Control

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

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three 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. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Adverse Effects of Dewatering on Adjacent Properties

Given the glacial till and bedrock soils present at the subject site and its vicinity, the neighbouring structures are expected to be founded on the glacial till or bedrock surface, which are not susceptible to settlement from dewatering. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Assessment

As part of the ongoing geotechnical investigation, Paterson has conducted a review of the slope along the creek adjacent to the eastern boundary of the site.

Based on the current field investigation and available topographic information, a cross-section was analyzed for slope stability using SLIDE, a computer program for two-dimensional slope stability analysis. Overall, 1 slope cross-section throughout the above-noted locations was analyzed as part of the slope stability analysis.

Slope Stability Analysis

The slope stability analysis was modelled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure.
Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.18 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading. It should be noted that only the figures with the lowest factor of safety are presented and considered the governing factors.

One (1) slope cross-section (Section A-A') was studied as the worst case scenario. The cross-section location is presented in Drawing PG7401-1 – Test Hole Location Plan in Appendix 2. It should be noted that details of the slope height and slope angle at the cross-section locations are presented in Figures 2 and 3 in Appendix 2, based on the topographic data obtained during the field investigation, as well as the available topographic data.

The effective and total strength soil parameters used for the static and seismic analyses were chosen based on the subsoil information recovered during the site investigation. The effective and total strength soil parameters used for the analyses are presented in Table 11 below.

Table 11 – Effective and Total Stress Soil Parameters

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	28	5
Fill	20	35	-
Glacial Till	20	38	-
Bedrock	22	-	1,000+

Stable Slope Allowance

The result of the static analysis, shown on Figure 2 in Appendix 2, indicates a factor of safety of 1.32. The result of the seismic analysis, shown on Figure 3, indicates a factor of safety of 0.72, respectively. As a result, a **stable slope allowance of 1.8 m** is required from the top of the slope at this site, which extends to the required factors of safety of 1.5 and 1.1 under static and seismic analysis conditions, respectively.

Toe Erosion Allowance

Since the slopes are in direct contact with an active watercourse, erosion of the toe of slope is considered to be a notable factor contributing to the stability of the subject slopes.

The toe erosion allowance is based on the nature of the soils, the observed current erosion activities, and the width and location of the current watercourse, and has been determined in accordance with the City of Ottawa's Slope Stability Guidelines for Development Application in the City of Ottawa (2004) and the Ontario Ministry of Natural Resource Technical Guide – River and Stream Systems: Erosion Hazard Limit (2002).

Generally, subsurface conditions at the toe of slope were observed to consist of a layer of glacial till underlain by bedrock. Based on this and the consistency of the subsoil materials encountered throughout our investigation and field review along the watercourse, a **toe erosion allowance of 1 m** is recommended to be considered for the associated Limit of Hazard Lands.

Erosion Access Allowance

Based on our review and current guidelines, a **6 m erosion access allowance** is recommended from the top of the stable slope for the slopes to allow for future maintenance of the slope.

Limit of Hazard Lands

Based on these observations and the results of the slope stability analysis, Paterson has established a Limit of Hazard Lands setback from the observed top of the slope. The Limit of Hazard Lands is determined by a combination of stable slope allowance, toe erosion allowance, and erosion access allowance.

As a result, a Limit of Hazard Lands designation of **8.8 m** is required from the top of the slope along the creek at the subject site. This Limit of Hazard Land setback is shown on the attached Drawing PG7401-1 - Test Hole Location Plan, located in Appendix 2.

It is recommended that the existing vegetation on the slope faces not be removed, as it plays a vital role in maintaining slope stability and mitigating erosion.

7.0 Recommendations

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

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

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

- ❑ Review and inspection of the installation of the foundation drainage and waterproofing systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant. All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided 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 TC United, 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.



Jacob Priest, B.Eng.

Kevin A. Pickard, P.Eng.

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- ❑ TC United
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370436.85 **NORTHING:** 5029250.48 **ELEVATION:** 73.44

PROJECT: Proposed Residential Development

ADVANCED BY: CME 55 Low-Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** July 6, 2026

FILE NO. : PG7401

HOLE NO. : BH 1-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.10m [73.34m]									73
FILL: Brown silty sand, trace to some gravel and clay		1	SS 1	25	0-4-10-8 14	6			
			SS 2	62	24-9-5-5 14	9			72
2.21m [71.23m]		2	SS 3	50	2-2-4-7 6	16			
Dense, brown **SILTY SAND** to **SANDY SILT**			SS 4	83	8-20-22-30 42	17		2.43 m 2026-07-16	71
2.97m [70.47m]		3	SS 5	0	50-/-/-/ 50/0.08				
GLACIAL TILL: Dense, grey silty sand, trace to some clay, cobbles, gravel and shale fragments		4	SS 6	62	10-20-25-36 45	7			70
		5	SS 7	25	50-/-/-/ 50/0.1				69
5.51m [67.93m]			SS 8	85	50-/-/-/ 50/0.03				68
BEDROCK: Fair to good quality shale bedrock		6	RC 1	100	RQD 77				67
		7	RC 2	100	RQD 69				66
		8	RC 3	100	RQD 79				65
9.02m [64.42m]		9							
End of Borehole									64
(GWL at 2.43 m depth - July 16, 2026)		10							63
		11							62
		12							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370461.84 **NORTHING:** 5029218.89 **ELEVATION:** 73.25

PROJECT: Proposed Residential Development

ADVANCED BY: CME 55 Low-Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** July 6, 2026

FILE NO. : PG7401

HOLE NO. : BH 2-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
FILL: Brown silty sand, with gravel - trace topsoil to 0.15 m depth			SS 1	83	1-5-8-6 13	12			73
		1	SS 2	17	3-5-2-3 7	12			72
1.98m [71.27m]			SS 3	21	6-8-6-5 14				
GLACIAL TILL: Dense to very dense, grey silty sand, with cobbles and boulders, trace clay and shale fragments		2	SS 4	17	4-9-11-9 20	17			71
		3	SS 5	83	5-14-20-19 34			3.15 m 2026-07-16	70
		4	SS 6	83	14-23-17-50 40	13			69
4.83m [68.42m]			SS 7	30	40-50-/-/ 50/0.08	9			
BEDROCK: Poor to good quality shale bedrock		5	RC 1	100	RQD 29				68
		6						6.20m	67
		7	RC 2	100	RQD 64				66
		8							65
9.25m [64.00m]		9	RC 3	100	RQD 81			9.25m	64
End of Borehole (GWL at 3.15 m depth - July 16, 2026)		10							63
		11							62
		12							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370468.09 **NORTHING:** 5029176.47 **ELEVATION:** 74.11

PROJECT: Proposed Residential Development

ADVANCED BY: CME 55 Low-Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** July 6, 2026

FILE NO. : PG7401

HOLE NO. : BH 3-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.10m [74.01m]									74
FILL: Brown silty sand and gravel, trace to some clay		1	SS 1	40	1-4-9-12 13	5	○		
			SS 2	83	4-4-4-5 8	11	○		73
1.83m [72.28m]		2	SS 3	83	3-7-12-15 19	11	○	1.83 m 2026-07-16	
GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, trace clay and shale fragments - Grey by 2.2 m depth			SS 4	50	5-12-27-49 39	7	○		72
		3	SS 5	71	17-50-50-/ 100/0.28	6	○		71
4.29m [69.82m]		4	SS 6	55	30-50-/-/ 50/0.08	5	○		70
BEDROCK: Poor to good quality shale bedrock			RC 1	100	RQD 30				
		5	RC 2	100	RQD 75				69
		6							68
7.62m [66.49m]		7	RC 3	100	RQD 80				67
End of Borehole									
(GWL at 1.83 m depth - July 16, 2026)		8							66
		9							65
		10							64
		11							63
		12							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370496.87 **NORTHING:** 5029175.71 **ELEVATION:** 74.30

PROJECT: Proposed Residential Development

ADVANCED BY: CME 55 Low-Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** July 7, 2026

FILE NO. : PG7401

HOLE NO. : BH 4-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.08m [74.22m]									74
FILL: Brown silty sand, with gravel and crushed stone		1	SS 1	72	3-9-9-5 18	5	○		
1.45m [72.85m]			SS 2	25	4-9-9-9 18	10	○		73
FILL: Grey silty sand, with gravel		2	SS 3	25	4-9-9-9 18				
			SS 4	17	4-8-8-7 16	8	○		72
		3	SS 5	50	5-5-8-10 13	6	○		71
3.96m [70.34m]		4	SS 6	62	7-14-15-20 29	8	○		70
GLACIAL TILL: Dense to very dense, grey silty sand, with gravel, cobbles and shale fragments			SS 7		12-34-50-/ 84/0.28	7	○	4.70 m ▼ 2026-07-16	
5.21m [69.09m]		5							69
BEDROCK: Poor to fair quality shale bedrock		6	RC 1	100	RQD 63				68
		7	RC 2	100	RQD 54				67
		8	RC 3	100	RQD 44				66
		9							65
		10	RC 4	100	RQD 64				64
10.62m [63.68m]									
End of Borehole		11							63
(GWL at 4.70 m depth - July 16, 2026)		12							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370447.07 **NORTHING:** 5029287.66 **ELEVATION:** 72.63

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Track Mounted Drill Rig

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 — PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							72
ASPHALT 0.08m [72.55m]			AU 1						
FILL: Compact, brown silty sand, with gravel and crushed stone 0.69m [71.94m]									
FILL: Stiff, brown silty clay, with sand, gravel and cobbles		1	SS 2	71	4-23-7-7 30				
1.45m [71.18m]									71
FILL: Dense, brown silty sand, with gravel and cobbles		2	SS 3	67	12-18-17-17 35				
			SS 4	87	10-15-32-27 47				70
2.97m [69.66m]		3							
GLACIAL TILL: Very dense, grey silty sand to sandy silt, with gravel, cobbles and boulders			SS 5	92	20-30-30-22 60				69
		4	SS 6	75	13-15-15-19 30				
			SS 7	100	18-50-/-/ 50/0.15				68
		5	SS 8	51	50-/-/-/ 50/0.05				
5.49m [67.14m]									67
BEDROCK: Weathered		6	SS 9	0	50-/-/-/ 50/0.03				
6.10m [66.53m]									
BEDROCK: Poor to fair quality shale, interbedded with limestone			RC 1	100	RQD 47				66
		7							
			RC 2	92	RQD 32				65
7.72m [64.91m]									
End of Borehole		8							64
		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370459.66 **NORTHING:** 5029249.36 **ELEVATION:** 73.61

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Track Mounted Drill Rig

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 — PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							73
TOPSOIL and organics — 0.08m [73.53m]			AU 1						
FILL: Loose to compact, brown silty sand, with gravel, cobbles and crushed stone		1	SS 2	50	4-4-3-2 7				72
		2	SS 3	46	4-4-3-8 7				71
2.59m [71.02m]									
GLACIAL TILL: Dense, brown silty sand to sandy silt, some gravel and cobbels		3	SS 4	100	14-21-23-22 44				70
4.11m [69.50m]		4							
GLACIAL TILL: Very dense, grey silty sand to sandy silt, with gravel, cobbles and boulders		5	SS 5	98	39-54-/-/ 54/0.15				69
- Clay at 5.94 m depth		6	SS 6	51	50-/-/-/ 50/0.05				68
5.99m [67.62m]									
BEDROCK: Poor quality shale, interbedded wtith limestone		7	RC 1	91	RQD 15				67
			RC 2	92	RQD 41				66
7.80m [65.81m]		8							65
End of Borehole		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370496.33 **NORTHING:** 5029207.70 **ELEVATION:** 74.01

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Track Mounted Drill Rig

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : BH 3-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) / PID (ppm) / PID (% LEL)	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0					50 100 150 200 / 20 40 60 80		74
FILL: Compact to dense, brown silty sand, with gravel and cruhsed stone			AU 1						
		1	SS 2	50	18-11-22-13 33				73
		2	SS 3	46	6-11-21-12 32			1.7 m – 2025-01-03	72
2.59m [71.42m]									
GLACIAL TILL: Very dense grey silty sand to sandy silt, with gravel, cobbles and boulders		3	SS 4	83	15-25-27-21 52				71
		4							70
		5	SS 5	83	11-18-28-24 46				69
6.15m [67.86m]		6	SS 6	102	50-/-/-/ 50/0.05				68
End of Borehole Pracical refusal to augering on inferred bedrock at 6.15 m depth (GWL at 1.70 m depth - January 3, 2025)		7							67
		8							66
		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370460.54 **NORTHING:** 5029194.69 **ELEVATION:** 74.00

PROJECT: Proposed Residential Development

BORINGS BY: Backhoe

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : TP 1-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 / PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							74
FILL: Compact, brown silty sand, with gravel, some shale fragments, trace brick			G 1						
0.50m [73.50m]			G 2						
FILL: Very dense, brown silty sand, with crushed stone, gravel and cobbles, trace concrete, metal and trash		1	G 3						73
			G 4						
2.70m [71.30m]		2	G 5						72
End of Test Pit		3							71
Groundwater infiltration was observed at 0.60 m depth		4							70
		5							69
		6							68
		7							67
		8							66
		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370447.93 **NORTHING:** 5029176.78 **ELEVATION:** 74.30

PROJECT: Proposed Residential Development

BORINGS BY: Backhoe

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : TP 2-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 / PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							74
FILL: Compact, brown silty sand, some crushed stone and gravel, trace clay and topsoil			G 1						
0.80m [73.50m]		1	G 2						73
FILL: Dense, brown silty sand, with crushed stone, gravel and cobbles, some trash and topsoil			G 3						
2.30m [72.00m]		2	G 4						72
GLACIAL TILL: Compact, grey silty clay, some gravel and cobbles 2.50m [71.80m]									
End of Test Pit		3							71
Groundwater infiltration was observed at 1.05 m depth		4							70
		5							69
		6							68
		7							67
		8							66
		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370517.04 **NORTHING:** 5029185.02 **ELEVATION:** 74.66

PROJECT: Proposed Residential Development

BORINGS BY: Backhoe

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : TP 3-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 / PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							74
FILL: Compact, brown silty sand, some gravel and cobbles, trace concrete and topsoil			G 1						
0.90m [73.75m]		1	G 2						73
Compact to loose, brown **SILTY SAND**, trace gravel and cobbles to 1.50 m depth		2	G 3						72
3.10m [71.56m]		3	G 4 G 5						71
End of Test Pit		4							70
Groundwater infiltration was observed at 2.80 m depth		5							69
		6							68
		7							67
		8							66
		9							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1867 Alta Vista Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 370473.15 **NORTHING:** 5029261.10 **ELEVATION:** 73.25

PROJECT: Proposed Residential Development

BORINGS BY: Backhoe

REMARKS:

DATE: December 20, 2024

FILE NO. : PG7401

HOLE NO. : TP 4-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: ANALYTICAL TESTS	GASTECH (ppm) / GASTECH (% LEL) 50 100 150 200 / PID (ppm) / PID (% LEL) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							73
FILL: Loose to compact, brown silty sand, some gravel and cobbles, metal, crushed stone, topsoil, asphalt, trace brick			G 1						
		1	G 2						72
		2	G 3						71
2.60m [70.65m]			G 4						
End of Test Pit Groundwater Infiltration was observed at 2.10 m depth		3							70
		4							69
		5							68
		6							67
		7							66
		8							65
		9							

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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

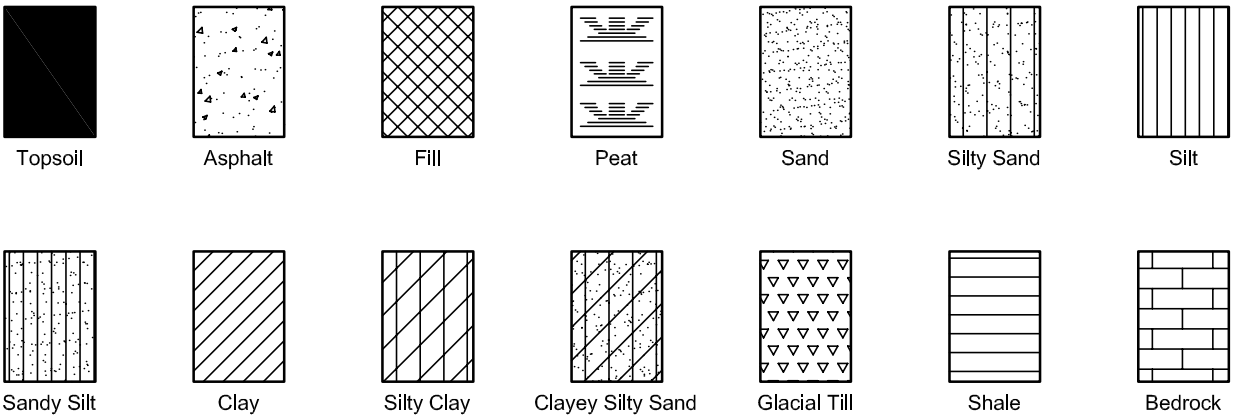
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

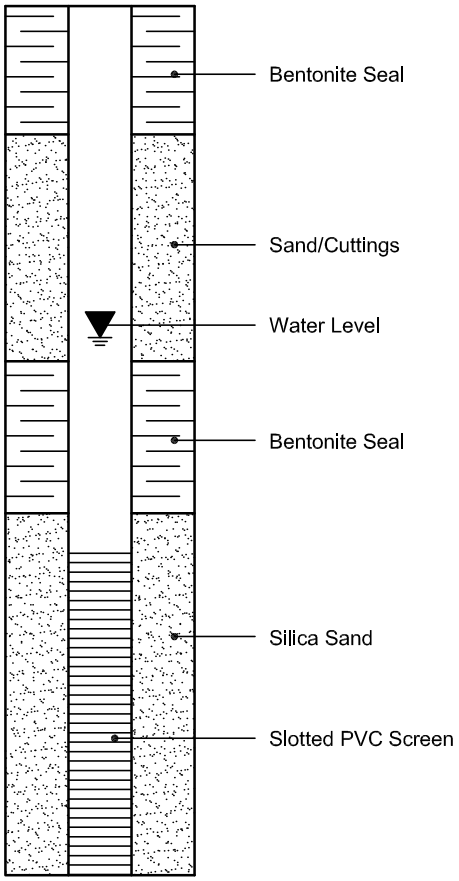
SYMBOLS AND TERMS (continued)

STRATA PLOT

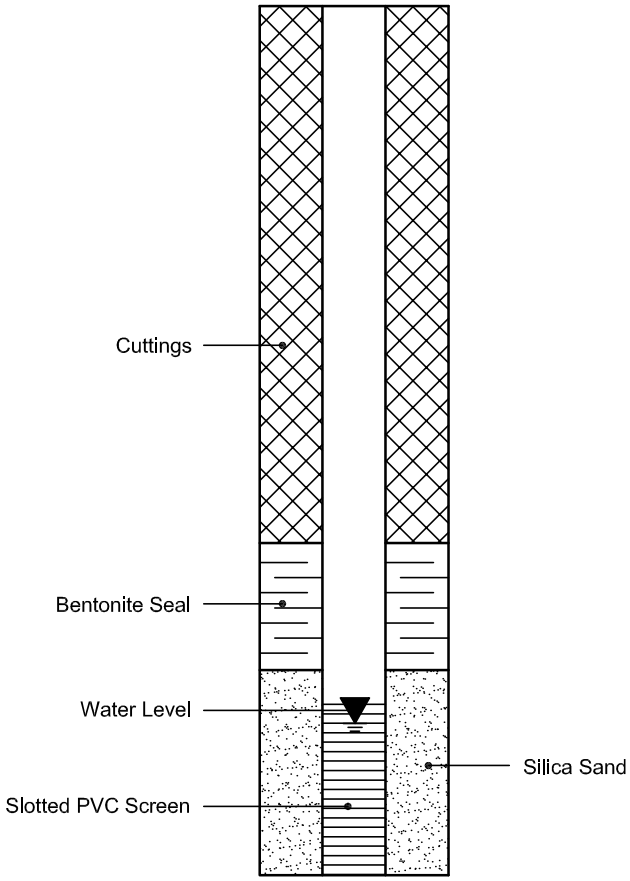


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



LOG OF BOREHOLE

NO. BH1(MW) SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

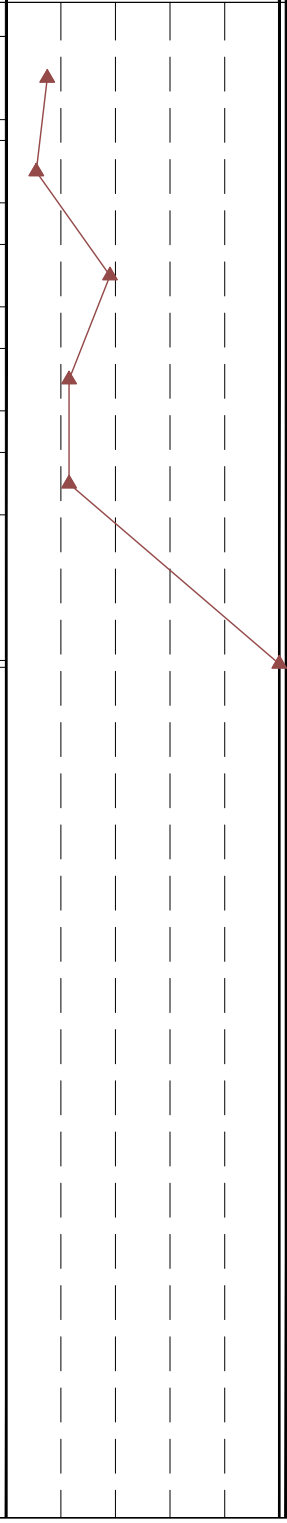
PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

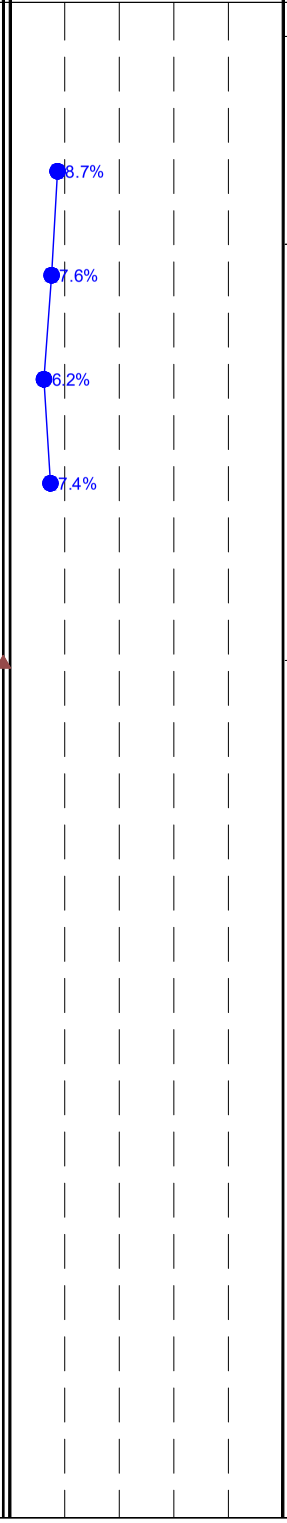
LOCATION: 1867 Alta Vista Drive, Ottawa, ON

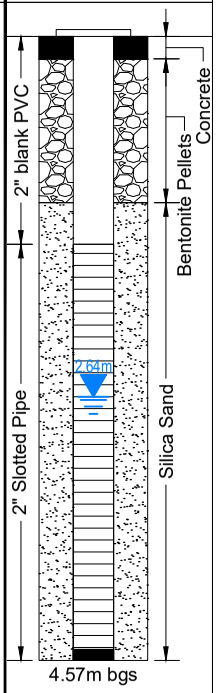
DRILLING METHOD: Truck, Soild Stem

DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				72.29			
0	0	~2" ASPHALT ~3" GRANULAR MATERIAL		0.13 72.16			
2	1	FILL: Greenish brown silty sand to silt & sand, trace gravel, mixed soil, cobbles, moist to wet, compact to dense				SS-1	15
4	1					SS-2	11
6	2					SS-3	38
8	2	Wet @ 2.31m				SS-4	23
10	3					SS-5	23
12	4						
14	4			4.57 67.72			
16	5	CLAYEY SILT TO SILTY CLAY: Grey, dry, hard		4.62 67.67		SS-6	100+
18	5	End of borehole at refusal to auger on suspected bedrock at 4.62m					
20	6						
22	7						
24	7						
26	8						
28	8						
30	9						
32	10						
34	10						







Groundwater Depth (m): on completion: 3.66m; on 7 October, 2022: 2.64m

DRAWN: D.C. | LOGGED: J.Y. | CHECKED: C.W.

FISHER ENGINEERING

LOG OF BOREHOLE

NO. BH2 SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 20 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				72.86			
0	0	~3" TOPSOIL					
		FILL: Brown sand, some silt, trace clay & gravel, moist, loose to dense				SS-1	6
2	1					SS-2	7
4							
6	2					SS-3	8
8						SS-4	6
10	3					SS-5	31
				3.51 / 69.35			
12	4	BEDROCK: Grey, dry, hard					
14		Auger refusal @ 4.62m		4.62 / 68.24		SS-6	100+
		End of borehole at 4.62m					
16	5						
18							
20	6						
22	7						
24							
26	8						
28							
30	9						
32							
34	10						

PENETRATION TESTING (SPT) ▲ 20 40 60 80

SHEAR STRENGTH (Kpa) ✚ 40 80 120 160

VAPOUR READING (ppm) □ 20 40 60 80

MOISTURE CONTENT (%) ● 10 20 30 40

PIEZOMETER OR WELL CONSTRUCTION

Groundwater Depth (m): on completion: Dry

DRAWN: D.C. LOGGED: J.Y. CHECKED: C.W.

LOG OF BOREHOLE

NO. BH3(MW) SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				73.47			
0	0	FILL: Brown silty sand, trace gravel, moist, compact				SS-1	15
				0.79 / 72.68			
4	1	SILTY GRAVELLY SAND (POSSIBLY FILL): Brown to grey, moist, compact				SS-2	16
6	2					SS-3	10
8						SS-4	16
10	3					SS-5	21
12	4						
14				4.62 / 68.85			
16	5	SILTY SAND (POSSIBLY FILL): Grey, trace gravel & clay, moist, very dense		5.03 / 68.44		SS-6	62
		End of borehole at 5.03m					
18							
20	6						
22							
24	7						
26	8						
28							
30	9						
32							
34	10						

PENETRATION TESTING (SPT) ▲ 20 40 60 80

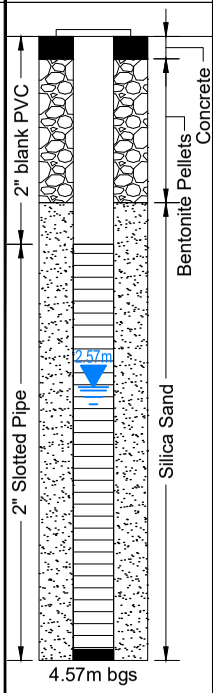
SHEAR STRENGTH (Kpa) ✚ 40 80 120 160

VAPOUR READING (ppm) □ 20 40 60 80

MOISTURE CONTENT (%) ● 10 20 30 40

● 12.9%

PIEZOMETER OR WELL CONSTRUCTION



Groundwater Depth (m): on completion: 2.41m; on 7 October, 2022: 2.57m

DRAWN: D.C. LOGGED: J.Y. CHECKED: C.W.

FISHER ENGINEERING

LOG OF BOREHOLE

NO. BH4 SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

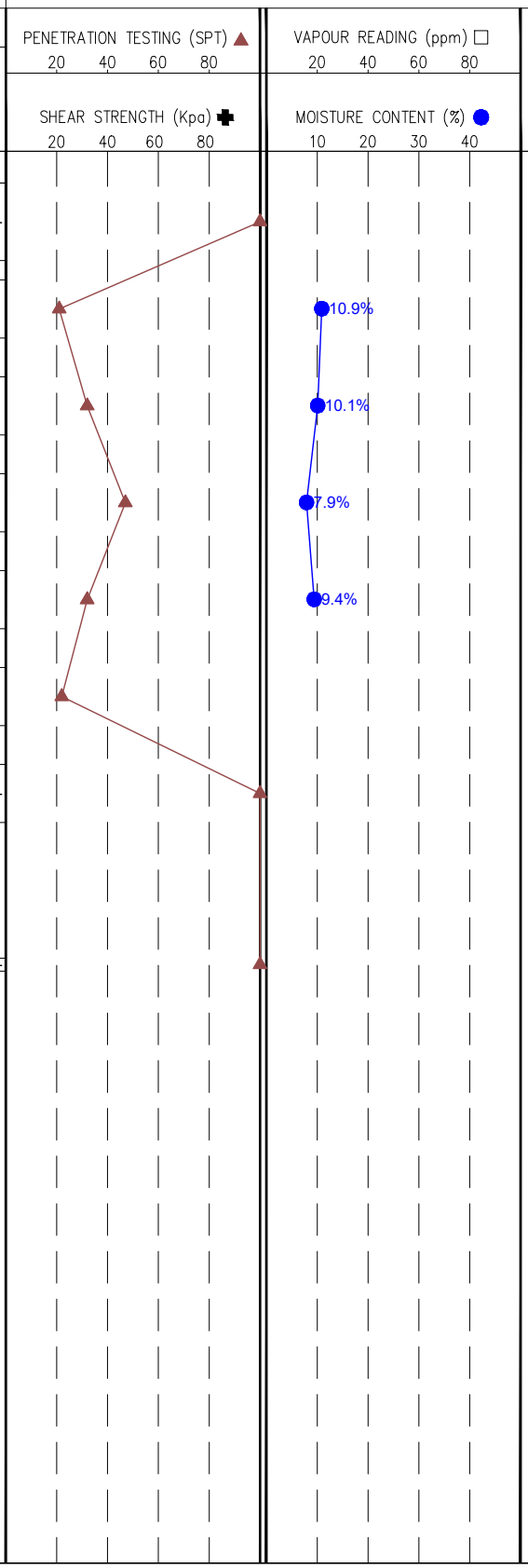
PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				73.48			
0	0	~2" ASPHALT ~3" GRANULAR MATERIAL		0.13 73.35			
		FILL: Brown to grey silty sand to sand, trace gravel, mixed soil, moist, very dense to compact				SS-1	100+
2	1					SS-2	21
4							
6	2					SS-3	32
8						SS-4	47
10	3					SS-5	32
12							
	4					SS-6	22
14							
16	5			4.97 68.51		SS-7	100+
		CLAYEY SILT TO SILTY CLAY: Grey, dry, hard					
18							
20	6					SS-8	100+
		End of borehole at refusal to auger on suspected bedrock at 6.20m		6.20 67.28			
22	7						
24							
26	8						
28							
30	9						
32							
34	10						



PIEZOMETER OR WELL CONSTRUCTION

Groundwater Depth (m): on completion: Dry

DRAWN: D.C. | LOGGED: J.Y. | CHECKED: C.W.

FISHER ENGINEERING

LOG OF BOREHOLE

NO. BH5 SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 20 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				73.99			
0	0	~2" TOPSOIL					
		FILL: Grey/brown silt & sand to sand, with gravel, trace silt, mixed soil, moist to wet, compact to dense				SS-1	29
2	1					SS-2	24
4							
6	2					SS-3	25
		Wet @ 2.44m					
8						SS-4	41
10	3					SS-5	26
12							
14	4			4.60 / 69.39			
		End of borehole at refusal to auger on suspected bedrock at 4.60m				SS-6	100+
16	5						
18							
20	6						
22							
24	7						
26	8						
28							
30	9						
32							
34	10						

PENETRATION TESTING (SPT) ▲ 20 40 60 80

SHEAR STRENGTH (Kpa) ✚ 40 80 120 160

VAPOUR READING (ppm) □ 20 40 60 80

MOISTURE CONTENT (%) ● 10 20 30 40

8.7%

PIEZOMETER OR WELL CONSTRUCTION

Groundwater Depth (m): on completion: 2.44m

DRAWN: D.C. LOGGED: J.Y. CHECKED: C.W.

LOG OF BOREHOLE

NO. BH6(MW) SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

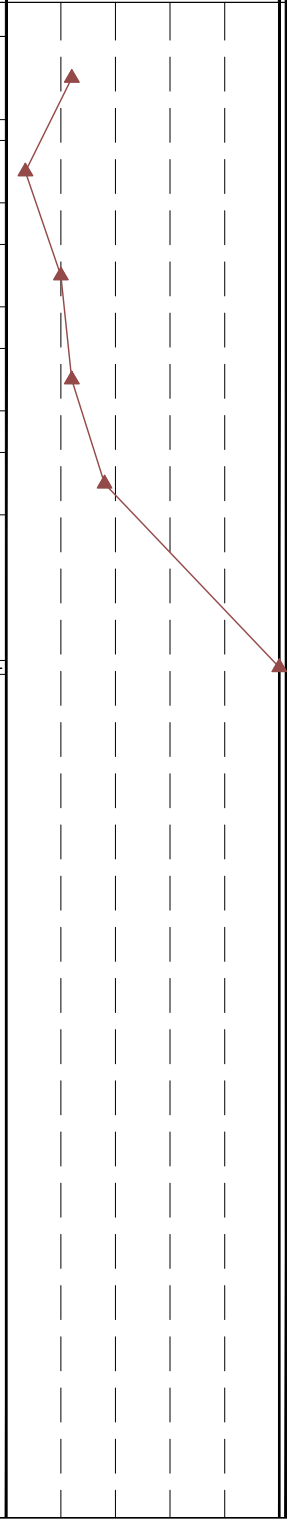
PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

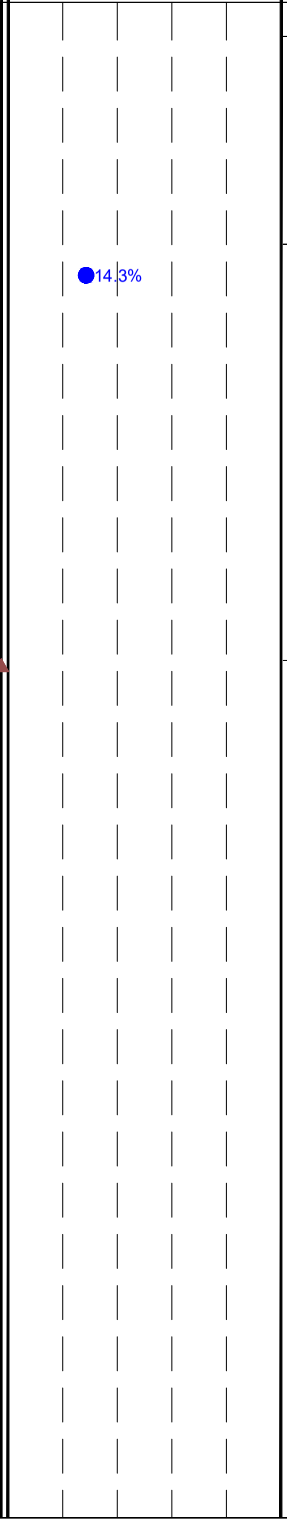
LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

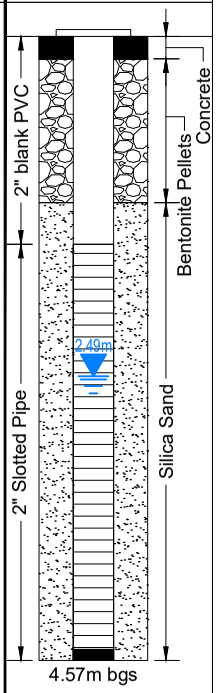
DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				72.82			
0	0	~3" ASPHALT		0.15			
		~3" GRANULAR MATERIAL		72.67			
2	1	FILL: Greenish brown, silt and sand, trace gravel & clay, some mixed soil, moist to wet, loose to dense				SS-1	24
4		Wet @ 1.52m				SS-2	7
6	2					SS-3	20
8						SS-4	24
10	3					SS-5	36
12	4						
14				4.60			
				68.22		SS-6	100+
		CLAYEY SILT TO SILTY CLAY: Grey, dry, hard		4.67			
16	5	End of borehole at refusal to auger on suspected bedrock at 4.67m		68.15			
18							
20	6						
22							
24	7						
26	8						
28							
30	9						
32							
34	10						





PIEZOMETER OR WELL CONSTRUCTION



Groundwater Depth (m): on completion: 4.57m; on 7 October, 2022: 2.49m

DRAWN: D.C. LOGGED: J.Y. CHECKED: C.W.

FISHER ENGINEERING

LOG OF BOREHOLE

NO. BH7 SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

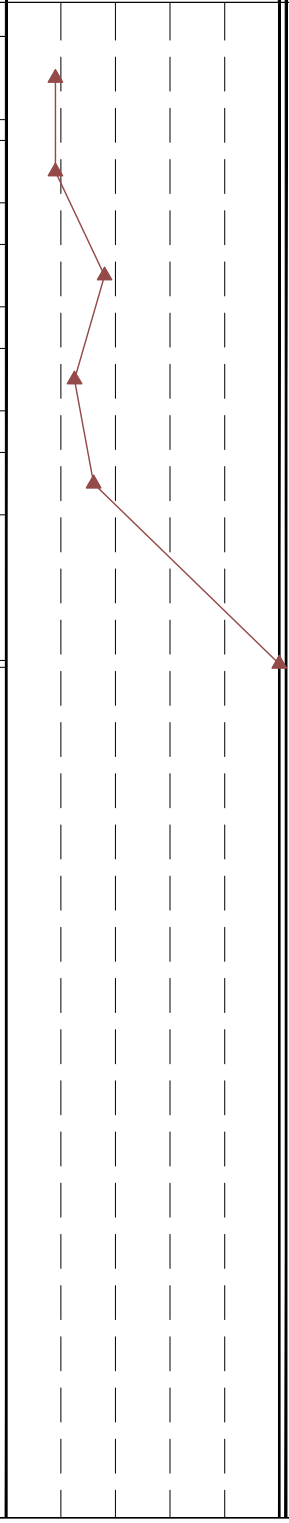
PROJECT NAME: GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

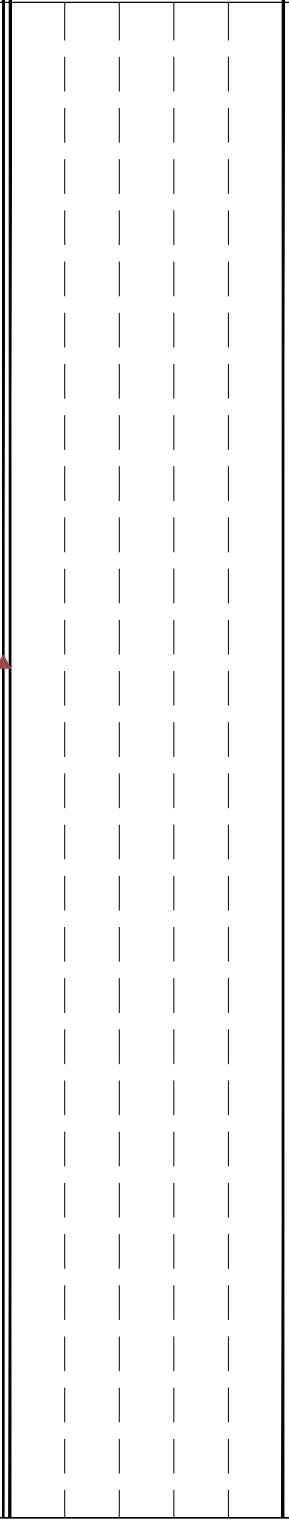
LOCATION: 1867 Alta Vista Drive, Ottawa, ON

DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				73.76			
0	0	~6" ASPHALT & GRANULAR MATERIAL		0.15 / 73.61			
		FILL: Greyish brown silty sand to silt & sand, trace gravel & clay, moist, compact to dense				SS-1	18
4	1					SS-2	18
6	2					SS-3	36
8						SS-4	25
10	3					SS-5	32
12	4						
14				4.60 / 69.16			
16	5	CLAYEY SILT TO SILTY CLAY: Grey, dry, hard		4.62 / 69.14		SS-6	100+
18		End of borehole at refusal to auger on suspected bedrock at 4.62m					
20	6						
22	7						
24							
26	8						
28							
30	9						
32	10						
34							





PIEZOMETER OR WELL CONSTRUCTION

Groundwater Depth (m): on completion: Dry

DRAWN: D.C. | LOGGED: J.Y. | CHECKED: C.W.

LOG OF BOREHOLE

NO. BH8(MW) SHEET. 1 of 1

PROJECT NO.: FE-P# 22-12467/68

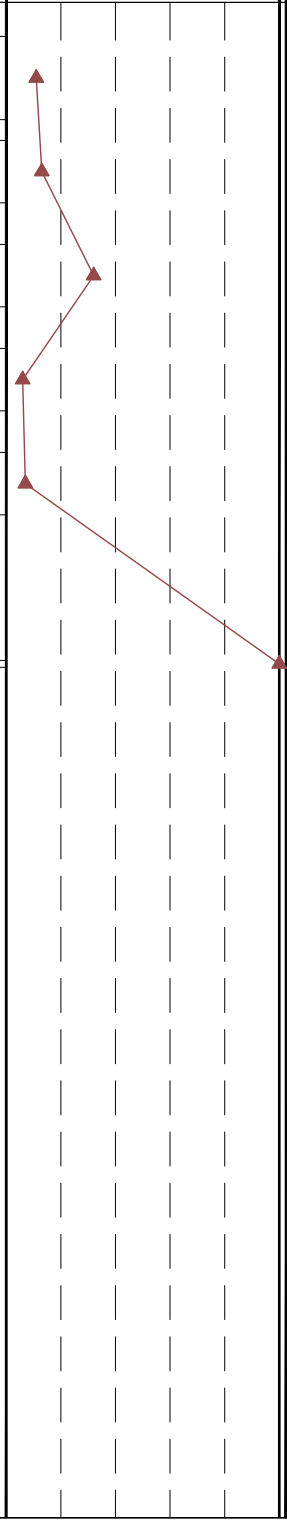
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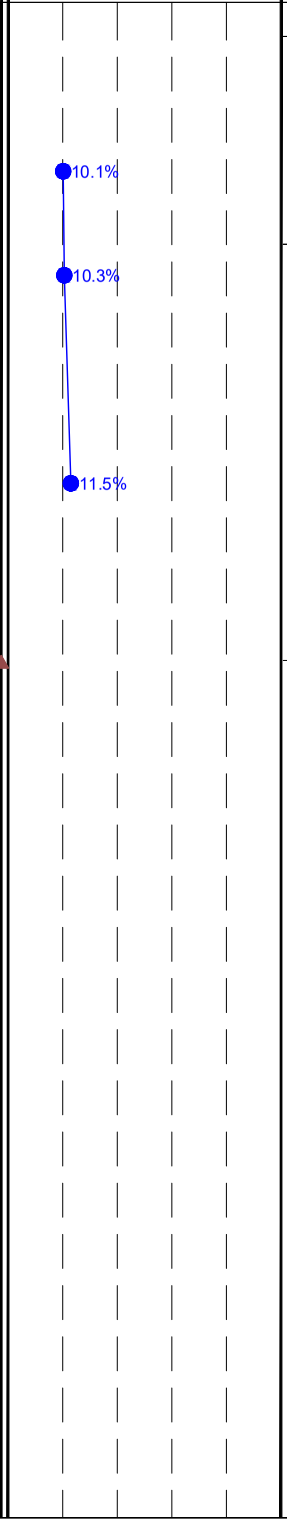
LOCATION: 1867 Alta Vista Drive, Ottawa, ON

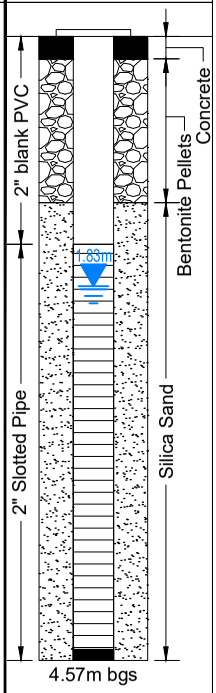
DRILLING METHOD: Truck, Solid Stem

DRILLING DATE: 19 September, 2022

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	LAB ID	Type NO.	"N" VALUE
				74.23			
0	0	~4" TOPSOIL					
		FILL: Brown sand, some silt, trace gravel, mixed soil, trace clay, moist, dense to loose				SS-1	11
2	1					SS-2	13
4							
6	2					SS-3	32
8						SS-4	6
10	3			3.11			
		SILTY SAND (POSSIBLY FILL): Grey, some clay, trace gravel, wet, loose		71.12		SS-5	7
12	4						
14				4.60			
				69.63			
		CLAYEY SILT TO SILTY CLAY: Grey, dry, hard		4.62		SS-6	100+
16	5			69.61			
		End of borehole at refusal to auger on suspected bedrock at 4.62m					
18							
20	6						
22							
24	7						
26	8						
28							
30	9						
32							
34	10						







Groundwater Depth (m): on completion: 2.44m; on 7 October, 2022: 1.83m

DRAWN: D.C. | LOGGED: J.Y. | CHECKED: C.W.

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**Claridge Homes**		FILE No.: **PG7401**
PROJECT:	1867 Alta Vista Drive		REPORT No.: 1
SITE ADDRESS:	**1867 Alta Vista Drive**		DATE REPT'D: 3-Jan-25
STRUCTURE TYPE & LOCATION:	Rock Core		
SAMPLE INFORAMTION			
LAB NO.:	**58811 Top**	**58811 Bottom**	
SAMPLE NO.:	BH1-24 RC2	BH1-24 RC2	
LOCATION:	23'5" - 23'11"	23'5" - 23'11"	
SAMPLE DATES			
DATE CAST	-	-	
DATE CORED	-	-	
DATE RECEIVED	23-Dec-24	23-Dec-24	
DATE TESTED	3-Jan-25	3-Jan-25	
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00	47.00	
HEIGHT (mm)	51.00	67.00	
WEIGHT (g)	230	320	
AREA (mm^2)	1735	1735	
VOLUME (cm^3)	88	116	
UNIT WEIGHT (kg/m^3)	**2599**	**2753**	
TEST RESULTS			
H / D RATIO	1.09	1.43	
CORRECTION FACTOR	0.901	0.951	
LOAD (lbs)	11700	16000	
GROSS Mpa	30.0	41.0	
MPa CORRECTED	**27.0**	**39.0**	
FORM OF BREAK	**Type A**	**Type A**	
DIRECTION OF LOADING	PARALLEL	PARALLEL	
CURING CONDITIONS	SITE→→→

Photo 1:

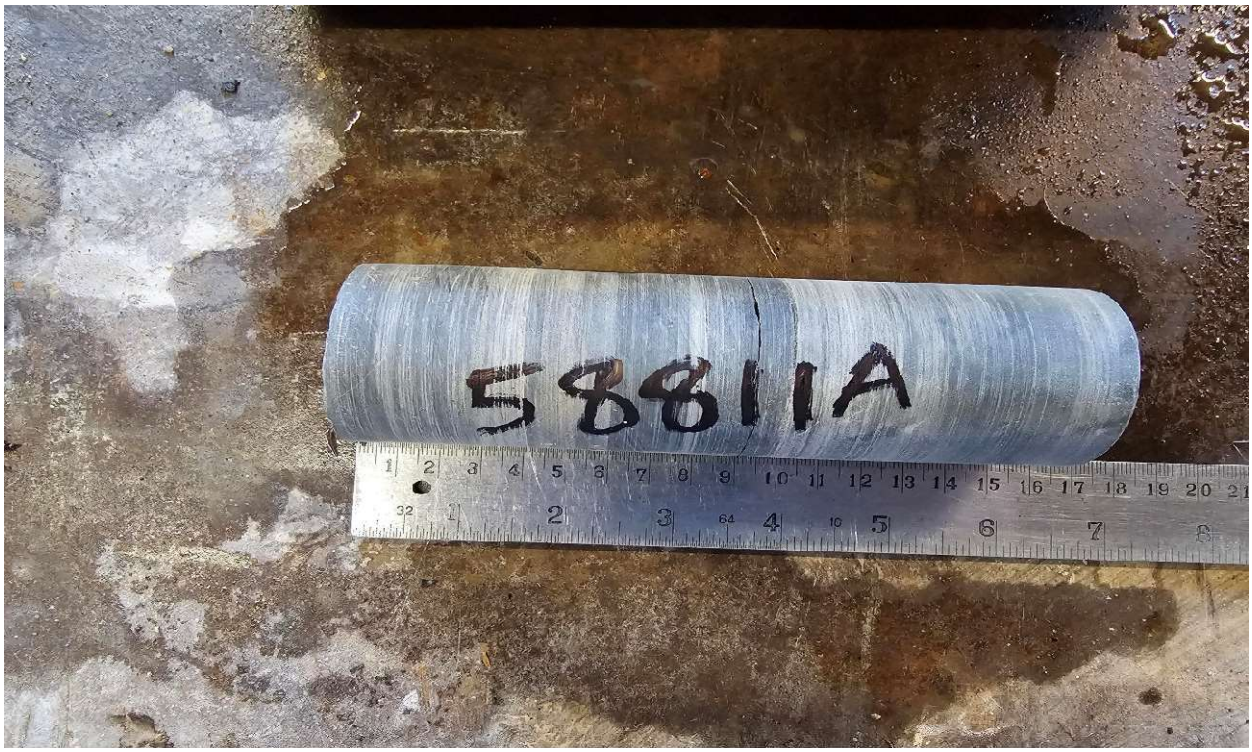


Photo 2:

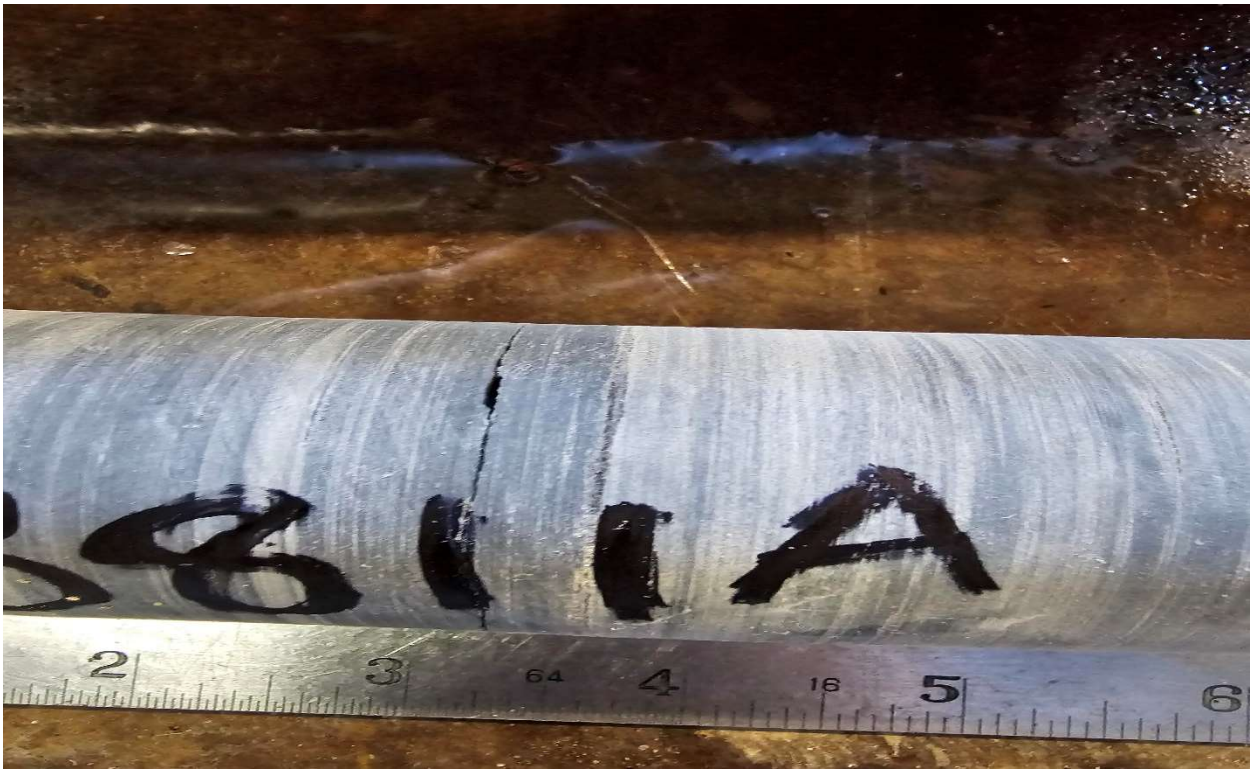
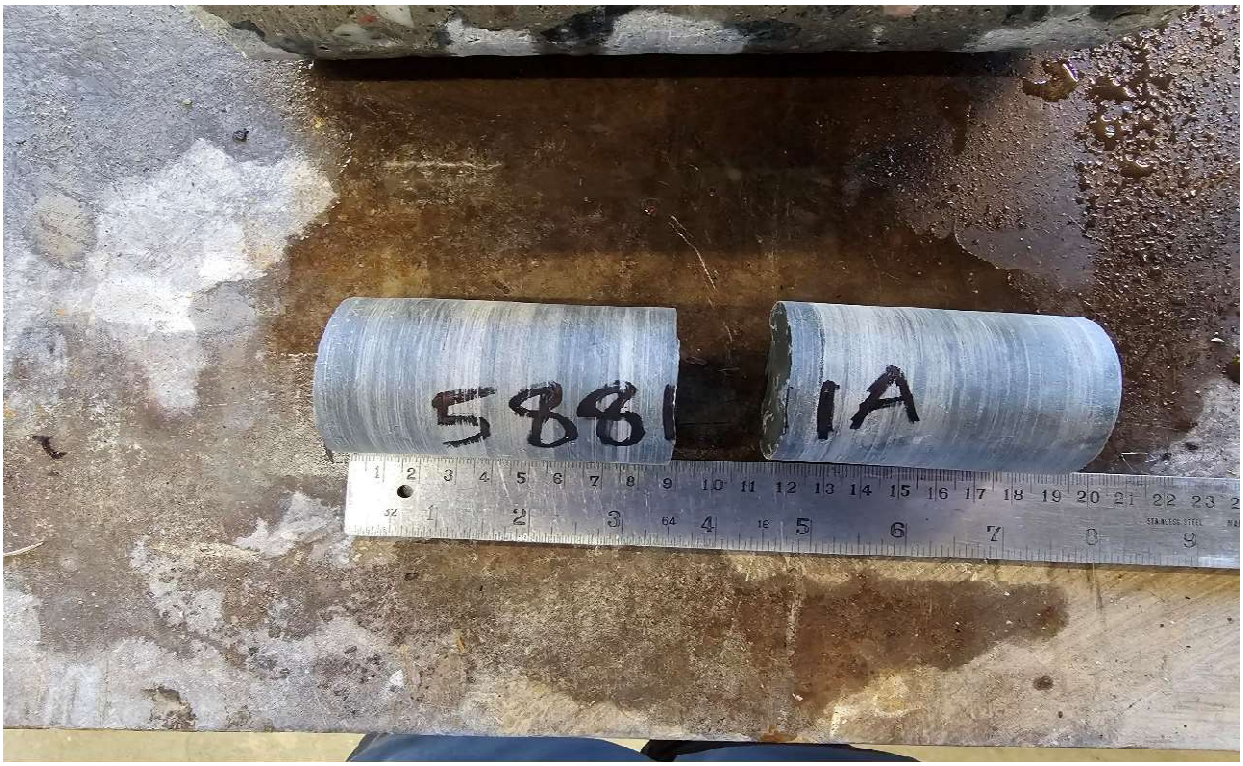


Photo 3:



PARACEL LABORATORIES LTD.

Order #: 2452099

Certificate of Analysis

Report Date: 02-Jan-2025

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 24-Dec-2024

Client PO: 62083

Project Description: PE6830

	Client ID:	BH1-24 SS4	-	-	-		
	Sample Date:	20-Dec-24 09:00	-	-	-	-	-
	Sample ID:	2452099-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	91.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.16	-	-	-	-	-
Resistivity	0.1 Ohm.m	4.8	-	-	-	-	-
Anions							
Chloride	10 ug/g	260	-	-	-	-	-
Sulphate	10 ug/g	84	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SLOPE STABILITY ANALYSIS CROSS-SECTIONS

FIGURES 4 & 5 - SEISMIC SHEAR WAVE VELOCITY PROFILES

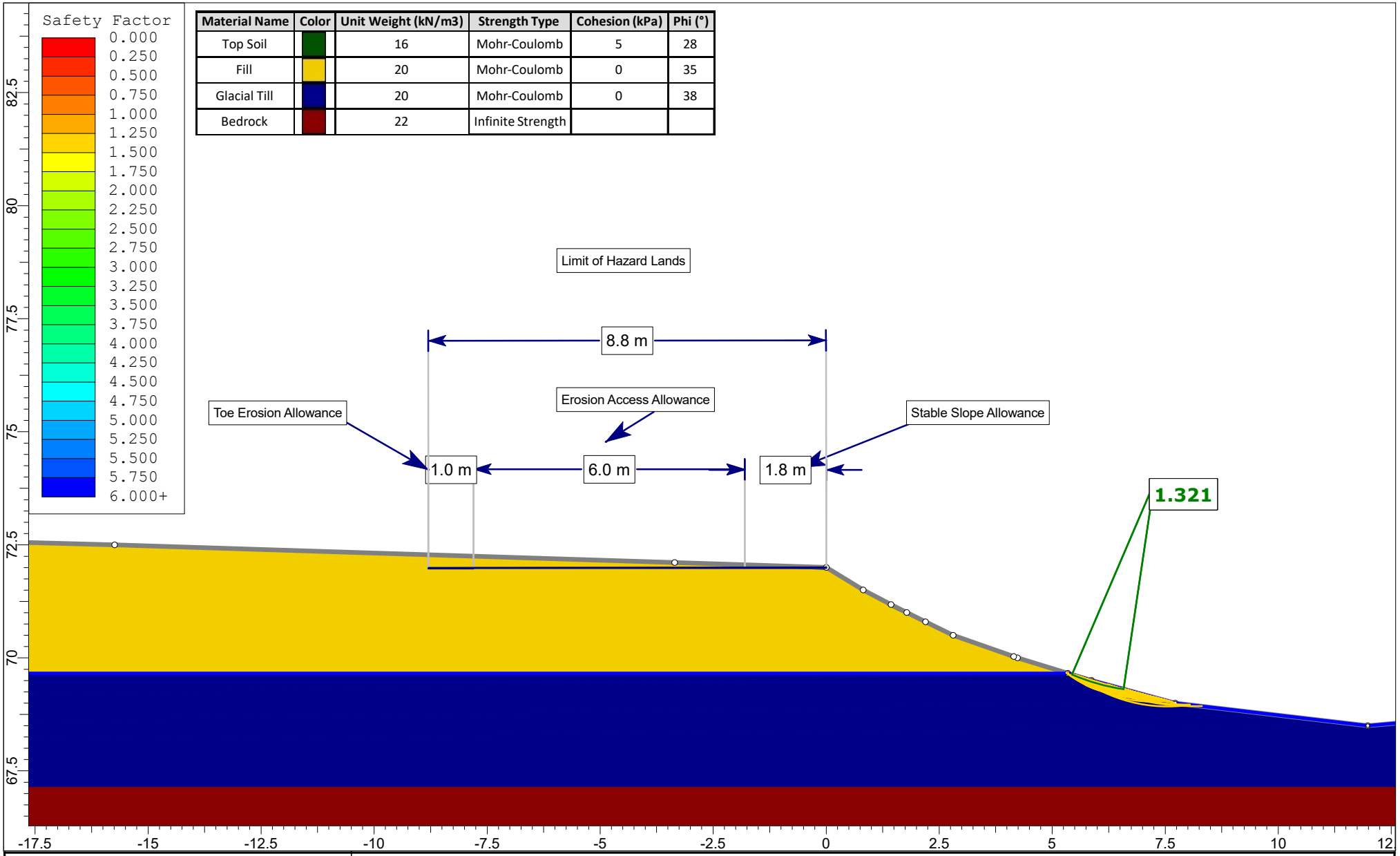
DRAWING PG7401-1 – TEST HOLE LOCATION PLAN



FIGURE 1

KEY PLAN

PATERSON GROUP

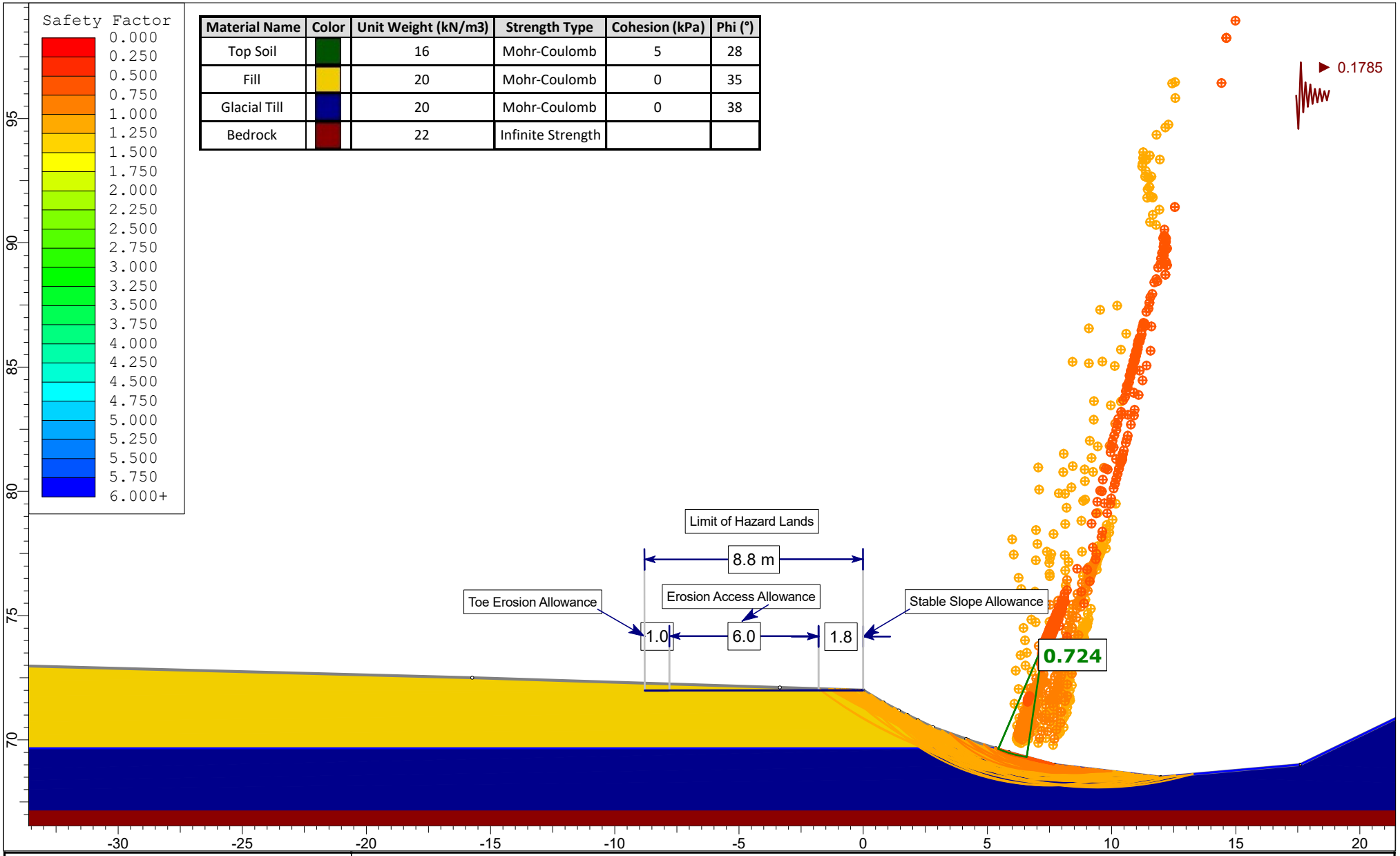


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Top Soil		16	Mohr-Coulomb	5	28
Fill		20	Mohr-Coulomb	0	35
Glacial Till		20	Mohr-Coulomb	0	38
Bedrock		22	Infinite Strength		

PATERSON GROUP

SLIDEINTERPRET 9.031

Project	1867 Alta Vista Drive - Ottawa, Ontario		
Group	Cross Section A-A - Figure 2	*Scenario*	Static Condition
Drawn By	EA	*Client*	Claridge Homes
Date	1/15/2025	*File Name*	PG7401-1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Top Soil		16	Mohr-Coulomb	5	28
Fill		20	Mohr-Coulomb	0	35
Glacial Till		20	Mohr-Coulomb	0	38
Bedrock		22	Infinite Strength		

PATERSON GROUP

SLIDEINTERPRET 9.031

Project	1867 Alta Vista Drive - Ottawa, Ontario		
Group	Cross Section A-A - Figure 3	Scenario	Seismic Condition
Drawn By	EA	Client	Claridge Homes
Date	1/15/2025	File Name	PG7401-1



Figure 4 – Shear Wave Velocity Profile at Shot Location -2 m

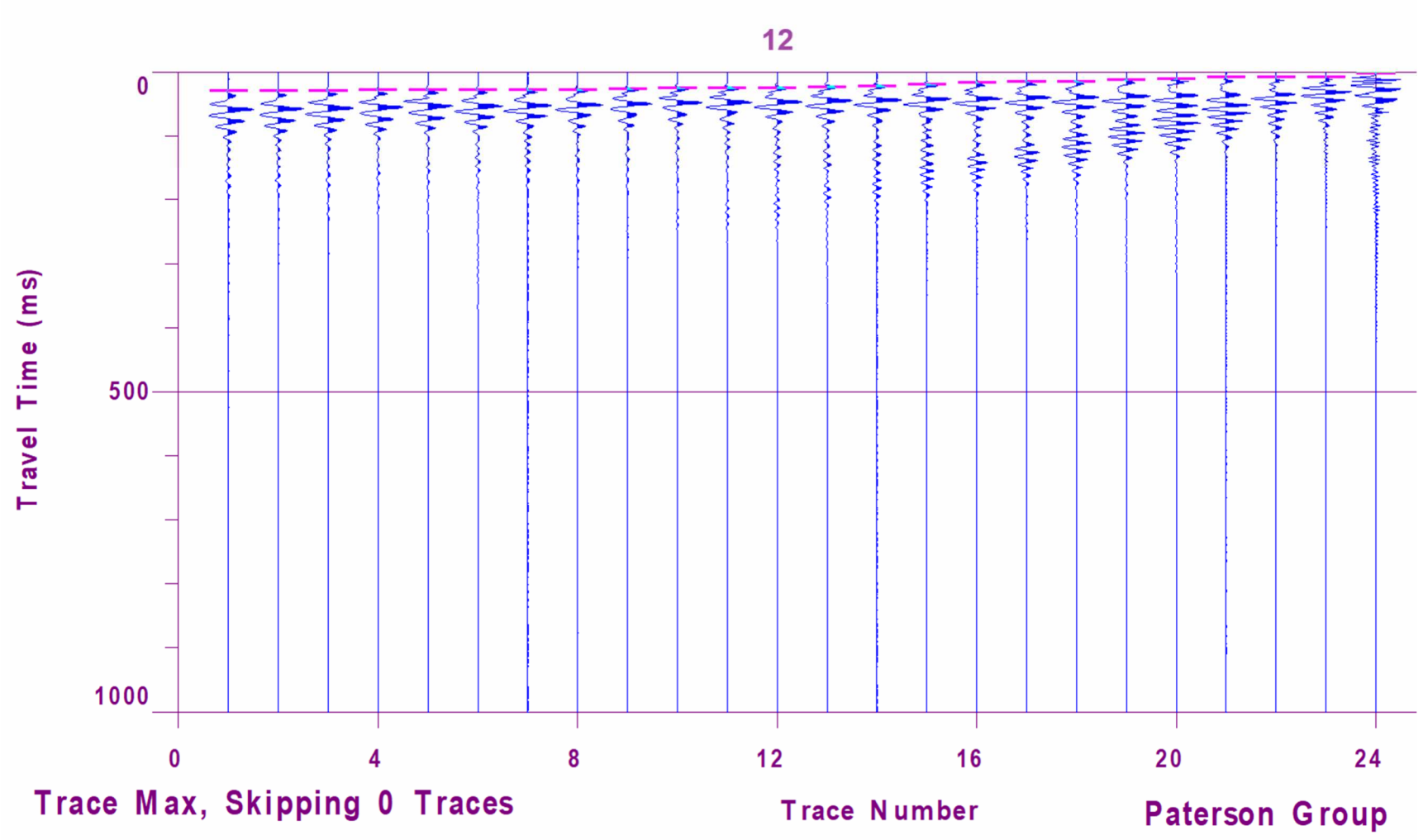
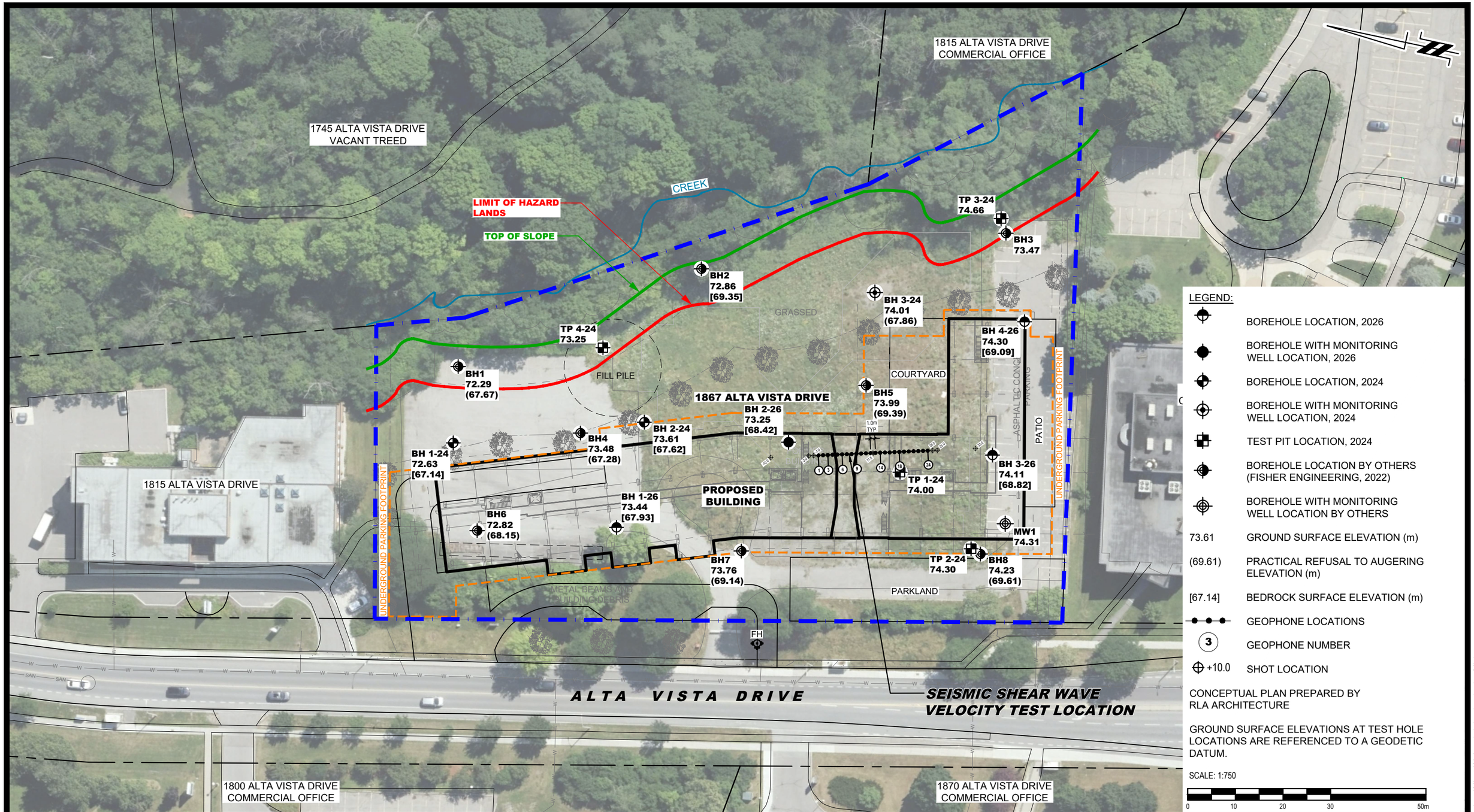


Figure 5 – Shear Wave Velocity Profile at Shot Location 24 m



LEGEND:

- BOREHOLE LOCATION, 2026
- BOREHOLE WITH MONITORING WELL LOCATION, 2026
- BOREHOLE LOCATION, 2024
- BOREHOLE WITH MONITORING WELL LOCATION, 2024
- TEST PIT LOCATION, 2024
- BOREHOLE LOCATION BY OTHERS (FISHER ENGINEERING, 2022)
- BOREHOLE WITH MONITORING WELL LOCATION BY OTHERS
- 73.61 GROUND SURFACE ELEVATION (m)
- (69.61) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
- [67.14] BEDROCK SURFACE ELEVATION (m)
- GEOPHONE LOCATIONS
- 3 GEOPHONE NUMBER
- +10.0 SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY RLA ARCHITECTURE

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

SCALE: 1:750

0 10 20 30 50m

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

NO.	REVISIONS	DD/MMYYYY	INITIAL
4	ADDED SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	12/08/2026	FC
3	ADDED 2026 BOREHOLE LOCATIONS	21/07/2026	KP
2	ADDED NEW CONCEPTUAL PLAN	04/12/2025	KP
1	ADDED CONCEPTUAL PLAN	21/04/2025	PB

TC UNITED
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
1867 ALTA VISTA DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 01/2025
Drawn by: GK	Report No.: PG7401-1
Checked by: PB	Dwg. No.: PG7401-1
Approved by: SD	Revision No.: 4

11x17

p:\autocad drawings\geotechnical\pg7401\pg7401-1-test hole location plan (rev.04).dwg