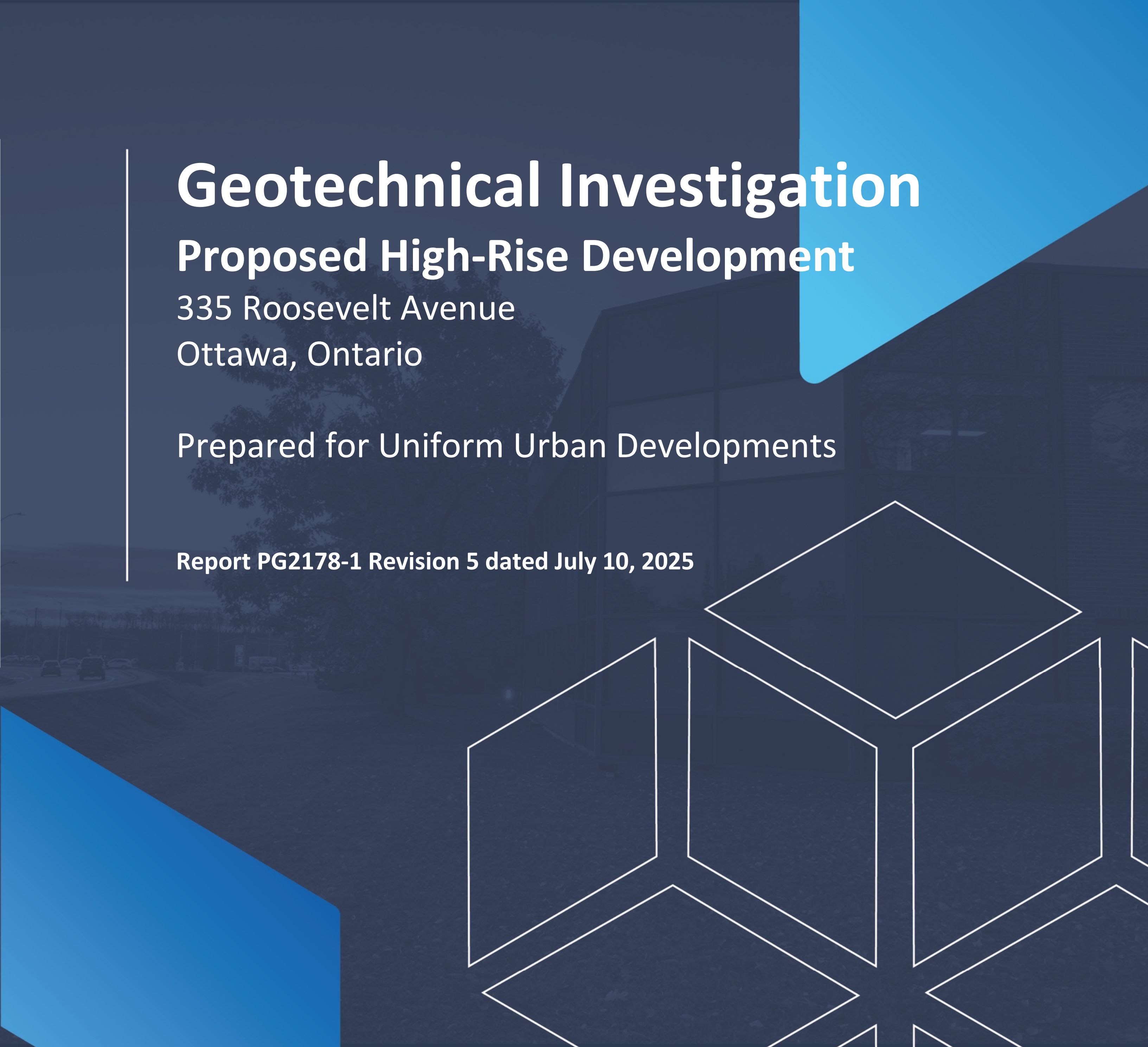


Geotechnical Investigation

Proposed High-Rise Development

335 Roosevelt Avenue
Ottawa, Ontario

Prepared for Uniform Urban Developments

Report PG2178-1 Revision 5 dated July 10, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Uniform Urban Developments to conduct a geotechnical investigation for the proposed development to be located at 335 Roosevelt Avenue in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes, and to

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development at the subject will consist of 2 high-rise buildings. The west building will have 3 underground parking levels, while the east building will have 4 underground parking levels.

At finished grades, the proposed buildings will be surrounded by asphalt-paved walkways and parking areas with paver walkways and landscaped margins. It is also expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on April 1 and 4, 2025. At that time, 6 boreholes and 14 test pits were advanced to maximum depths of 15.1 and 3.0 m below the existing ground surface, respectively. A previous geotechnical investigation conducted at subject site by Paterson in November 2010 included 5 boreholes advanced to a maximum depth of 9.5 m below the existing ground surface.

The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG2178-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a truck-mounted auger drill rig operated by a two-person crew and test pits were completed by a hydraulic excavator. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden. In addition, bedrock was cored at each borehole location using diamond drilling procedures.

Sampling and In Situ Testing

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

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

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

Diamond drilling was carried out at borehole BH 1-25 through BH 6-25 to assess the bedrock depth and quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data Sheets in Appendix 1.

The recovery value is the ratio shown, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

Groundwater

A groundwater monitoring well was installed in all boreholes to permit the monitoring of the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations, and the ground surface elevation at each test hole location for the current investigation, were surveyed by Paterson using a handheld GPS unit with respect to a geodetic datum. The ground surface elevation at each borehole locations for the previous investigation was referenced to a temporary benchmark (TBM), consisting of a magnetic nail in a utility pole. A geodetic elevation of 67.30 m has been provided for the TBM by Annis O’Sullivan Vollebekk Ltd. The location of the TBM and boreholes, as well as the ground surface elevation at each borehole, are presented on Drawing PG2178-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples from the current investigation will be stored in the laboratory for 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 samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

At the time of the field program, two existing residential buildings were located along the southwest boundary of the subject site. The remainder of the site was surfaced with gravel and fill.

The site is bordered to the north by the transitway, to the west by Roosevelt Avenue, to the south by Winston Avenue and Wilmont Avenue, and to the east by a 7-storey residential building. The western-most building was noted to be approximately 0.6 m below Roosevelt Avenue.

A sewer (West Nepean Collector) and watermain are located just to the north of the subject site, with inverts at approximate elevations of 50 m and 63 m, respectively. Additionally, the transit-way located north of the subject site was noted to be approximately 6 m below the elevation of 335 Roosevelt Avenue. The subject site is relatively flat.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of either asphaltic concrete or fill overlying native silty sand or silty clay. Bedrock was encountered at depths between about 0.5 and 3 m.

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

Based on the results of the bedrock coring, the bedrock consists of limestone with layers of black shale which is generally poor to fair in quality in the upper 2 m, becoming to good to excellent in quality with depth.

Available geological mapping indicates that the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation, with drift thicknesses varying between 1 and 2 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in all boreholes on November 16, 2010. The measured GWL readings are presented in Table 1 below. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 – Summary of Groundwater Levels

Test hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1	66.39	4.88	61.51	November 16, 2010
BH 2	66.37	6.53	59.84	
BH 3	66.43	Dry	--	
BH 4	66.64	3.84	62.80	
BH 5	66.50	4.97	61.53	
BH 1-25	66.26	3.57	62.69	April 11, 2025
BH 2-25	66.29	3.90	62.39	
BH 3-25	66.36	4.46	61.9	
BH 4-25	66.63	4.13	62.5	
BH 5-25	66.25	4.28	61.97	
BH 6-25	66.13	3.10	63.03	

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

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

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

5.0 Discussion

5.1 Geotechnical Assessment

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

Considering that the site is underlain by shallow bedrock (within about 1 m below the surface), shoring may not be necessary if the excavation of the overburden soils can be stepped back from the bedrock excavation face.

Bedrock removal will be required to complete the 3 levels of underground parking. Temporary rock bolts may be required to stabilize the walls of the excavation through bedrock.

A sewer (West Nepean Collector) and watermain run along the north property boundary, in close proximity to the subject site. It is expected that the adjacent sewer and watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent sewer and watermain, a vibration monitoring and control program is recommended to be undertaken during the blasting and excavation work required for the proposed building excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures. However, due to the depth of bedrock and the anticipated founding level for the proposed buildings, it is anticipated that all existing overburden material will be excavated from within the proposed building footprints.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

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

An existing watermain alignment is located approximately 2.5 m north of the subject site’s north property line. Blasting can be used for most of the bedrock removal up to a minimum horizontal distance of 2 m from the northern property line, along the existing watermain. Blasting operations will be reviewed and the 2 m minimum distance from the watermain may be increased if vibrations from the blasting operation are questionable.

Vibration monitors should be installed to measure the vibrations and to ensure that the vibration levels stay below 25 and 15 mm/s at the property boundary and watermain, respectively.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

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

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

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

Vibration Monitoring and Control Plan

Due to the presence of the existing sewer and watermain near the northern property boundary, a vibration monitoring and control plan (VMCP) is recommended during the excavation program. The purpose of the vibration monitoring and control plan is to provide measures to be implemented by the contractor to manage excavation operations and any other vibration sources during the construction for the proposed development. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

The monitoring program will incorporate real time results at the existing sewer and watermain segment adjacent to the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. At least two vibration monitoring devices should be placed adjacent to the existing watermain.

It is recommended that the vibration monitoring devices be installed at obvert level of the existing watermain, and be periodically inspected during the construction program.

This report, which includes the VMCP, should be provided to all parties involved with the construction for review. A meeting between Paterson and site contractor should be conducted prior to any excavation or construction of the subject site to review the following:

☐ The pre-condition/pre-construction survey.
☐ Control measures (i.e vibrations, noise).
☐ Monitoring locations.
☐ Tracking and reporting of excavation progress, and.
☐ Procedure for exceedances (i.e vibrations, noise), complaints, evaluation and corrective measures.

When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in the weekly report. The following table outlines the vibration limits for the adjacent watermain segment.

Table 2 - Structure Vibration Limits for adjacent Watermain Segment

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)	Event	Description of Event
<10	all	none	no action required
<40	>10	trigger level	Warning e-mail sent to contractor.
<40	≥15	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.
>40	>15	trigger level	Warning e-mail sent to contractor.
>40	≥20	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.

The monitoring protocol should include the following information:

Trigger Level Event

- ☐ Paterson will review all vibrations over the established warning level, and
- ☐ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance levels.

Exceedance Level Event

- ☐ Paterson will notify all the relevant stakeholders via email,
- ☐ Ensure monitors are functioning, and
- ☐ Isse the vibration exceedance results.

Fill Placement

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

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

If excavated bedrock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where this fill material is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

Lean Concrete Filled Trenches

Where rock overbreak occurs at the underside of footing (USF) elevation, lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate the subgrade from the bedrock surface to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured will suffice in providing a direct transfer of the footing load to the underlying bedrock.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on the clean, surface sounded bedrock surface can be designed using a bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **5,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at SLS and ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

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

Field Program

The seismic array was located within the proposed mid-rise building footprint at the subject site and as presented in Drawing PG2178-1 – 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 4 to 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 1, 2 and 10.5 m away from the first and last geophone, and at the centre of the seismic 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 profile, immediately below the proposed foundation of the buildings. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

It is understood that the footings of the proposed building are to be founded directly on the bedrock surface. From the testing results, the average bedrock shear wave velocity is **2,220 m/s**.

The V_{s30} was calculated using the standard equation for average shear wave velocity provided in OBC 2024 and as presented below:

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

$$V_{s30=}\frac{30\,m}{\left(\frac{30\,m}{2{,}220\,m/s}\right)}$$

$$V_{s30=}\,2{,}220\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed buildings founded on bedrock is **2,220 m/s**. Therefore, as per the OBC 2024, a **Seismic Site Designation X_{2220}** is applicable for the design of proposed buildings. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

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

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

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level slab of the proposed building. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3 (effective unit weight 13 kN/m^3).

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

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 0.303g according to 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 $P_o = 0.5\ K_o \cdot ɣ \cdot H^2$, where K = 0.5 for the soil conditions noted above.

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

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

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

5.7 Rock Anchor Design

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 limestone bedrock ranges between about 60 and 120 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 69** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 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 Limestone Hoek and Brown Parameters	69 m=0.575 and s=0.00293
Unconfined Compressive Strength – Shale Bedrock	60 MPa
Unit weight – Submerged Bedrock	15 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 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	
125	1.1	0.5	1.6	300
	1.5	0.7	2.2	500
	2.6	1	3.6	1000

Other Considerations

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

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

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed building consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 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.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas Over Podium

Thickness (mm)	Material Description
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

Table 7 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Loading Areas Over Podium

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the existing pavement structures. For this transition, a 5H:1V is recommended between the two subgrade surfaces.

Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

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

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

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.

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
400	**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	

Other Considerations

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

For the installation of the composite drainage board against the vertical bedrock face, the following is recommended:

☐ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).

☐ Mechanically remove bedrock along the foundation walls, up to approximately 150 mm from the finished vertical excavation face.

☐ Grind the bedrock surface up to the outer face of the line drilled holes to create a satisfactory surface for the composite drainage board.

☐ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features of the bedrock surface, as required based on site inspection by Paterson.

☐ Place a composite drainage board, such as Delta Drain 6000 or equivalent, against the prepared vertical bedrock surface. The composite drainage layer should extend from finished grade to underside of footing level. A waterproofing membrane should then be installed over the composite drainage board.

☐ Pour foundation wall against the composite drainage board and waterproofing membrane.

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 from the composite drainage board to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

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

Perimeter and Underslab Drainage System

The perimeter and underslab drainage system is recommended to control water infiltration below the underground parking level slab and to re-direct water from the building's foundation drainage system to the building's sump pit(s). For preliminary design purposes, it is recommended that 100 mm diameter perforated pipes be placed at approximate 6 m centres underlying the lowest level slab. The underslab drainage pipes should also be provided with a geosock and surrounded on all sides by a minimum 100 mm thick layer of 19 mm clear crushed stone.

The perimeter drainage system should be mechanically connected to the 100 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.

Elevator (and Sump) Pit Waterproofing

All elevator shaft exterior foundation walls and floor slabs should be waterproofed to avoid any infiltration into the elevator pit. The underside of the elevator pit slab should be waterproofed using a membrane such as Colphene BSW H for horizontal applications (or approved equivalent). It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved equivalent), is applied to the exterior of the elevator shaft foundation wall. The membrane should extend to the top of the footing in accordance with the manufacturer's specifications.

A continuous PVC waterstop, such as Southern Waterstop 14RCB (or approved equivalent), should be installed within the interface between the concrete base slab below the elevator pit sidewalls. An outlet for any trapped water should be installed through the elevator pit wall with a gravity connection to the underfloor drainage system or directly to the sump pit.

A protection board should be placed over the waterproofing membrane to protect the membrane from damage during the backfilling operations.

Consideration should also be given to waterproofing the sump pit(s). If chosen, the above-noted waterproofing methodology will also be applicable to sump pit waterproofing.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

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

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

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

6.3 Excavation Side Slopes

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

Unsupported Side Slopes

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

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

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

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

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

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

Temporary Shoring

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

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

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

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

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

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.

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

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

6.4 Pipe Bedding and Backfill

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

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

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

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

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

6.5 Groundwater Control

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

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

Permit to Take Water

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons as stipulated under O.Reg. 63/16.

Adverse Effects of Dewatering on Adjacent Properties

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

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Protection of Existing Watermain

During the bedrock removal for the proposed development, the existing watermain located just beyond the north property boundary of the subject site will require monitoring.

Bedrock Condition

Based on our existing information, the bedrock is expected at approximate elevation +/-65.5 m. The upper portion of the bedrock is weathered and the bedrock quality improves with depth. The bedrock quality is generally fair to good based on the rock quality designation (RQD) findings below upper 1 to 2 m of weathered bedrock.

Paterson undertook a test pit excavation program on the subject property along the northern boundary on September 13, 2010. Three test pits were excavated using a rubber-tired backhoe and our findings can be summarized as follows:

Subsurface Conditions	Test Pit 1	Test Pit 2	Test Pit 3
Pavement structure overlying sandy silt deposit thickness	810 mm	810 mm	710 mm
Weathered bedrock thickness	100 mm	none	none
South bedrock depth	910 mm	810 mm	710 mm

The approximate locations of the test pits are shown on Drawing PG2178-1 - Test Hole Location Plan in Appendix 2.

Bedrock Removal along the Northern Boundary

The bedrock removal for the subject site will be carried out using a combination of blasting and hoe-ramming techniques, especially along the northern boundary where the existing watermain is located. The bedrock removal along the northern boundary will be carried out as follows:

☐ Blasting can be used for most of the bedrock removal up to a minimum horizontal distance of 2 m from the northern property line. A minimum line drilling spacing of 300 mm c/c will be required at the 2 m blasting boundary.

☐ The blasting contractor will control the blasting operation to keep peak particle velocities below 25 mm/s at the property boundary. It is expected that the blasting contractor will commence the blasting operation at the opposite end of the site so that blasting patterns and vibrations can be monitored and verified prior to attempting any blasting along the northern boundary adjacent to the existing watermain. This approach will allow the blasting contractor to adjust and control the blasting operation.

☐ Blasting operations will be reviewed and the 2 m minimum distance from the watermain may be increased if vibrations from the blasting operation are questionable.

☐ Within the minimum 2 m distance from the watermain, the bedrock will be removed using hoe-ramming or grinding techniques. Blasting will not be permitted. Line drilling spacing will be decreased to 200 mm c/c along the proposed excavation boundary. Similar to the blasting operations, hoe-ramming or grinding operations will be governed by the vibrations they produce along the property boundary adjacent to the watermain.

Monitoring and Reporting

☐ Two seismographs will be installed directly on the bedrock along the northern property line to monitor vibrations. Each blasting event will be reviewed and reported to the blasting contractor and the site superintendent.

☐ A weekly summary report will be issued presenting our findings and observations. Any concerns identified during the monitoring will be immediately reported, as discussed in Section 5.2, and the rock removal operations in the immediate area will be temporarily halted to address the concern.

6.8 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

☐ Review preliminary and detailed grading, servicing and landscaping plans, from a geotechnical perspective.

☐ Review of the geotechnical aspects of the foundation drainage systems prior to construction, if applicable.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.

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

☐ Review and inspection of the installation of the foundation drainage and waterproofing systems.

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

☐ Sampling and testing of the concrete and fill materials.

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

☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.

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

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Urban Developments, 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.

Deepak K Rajendran, E.I.T.

LICENSED PROFESSIONAL ENGINEER
July 10, 2025
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Uniform Urban Development
- ☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362959.59 **NORTHING:** 5028485.36 **ELEVATION:** 66.26

PROJECT:

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 1, 2025

FILE NO. : PG2178

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Compact, brown silty sand and gravel, with crushed stone, trace clay

0.64m [65.62m]

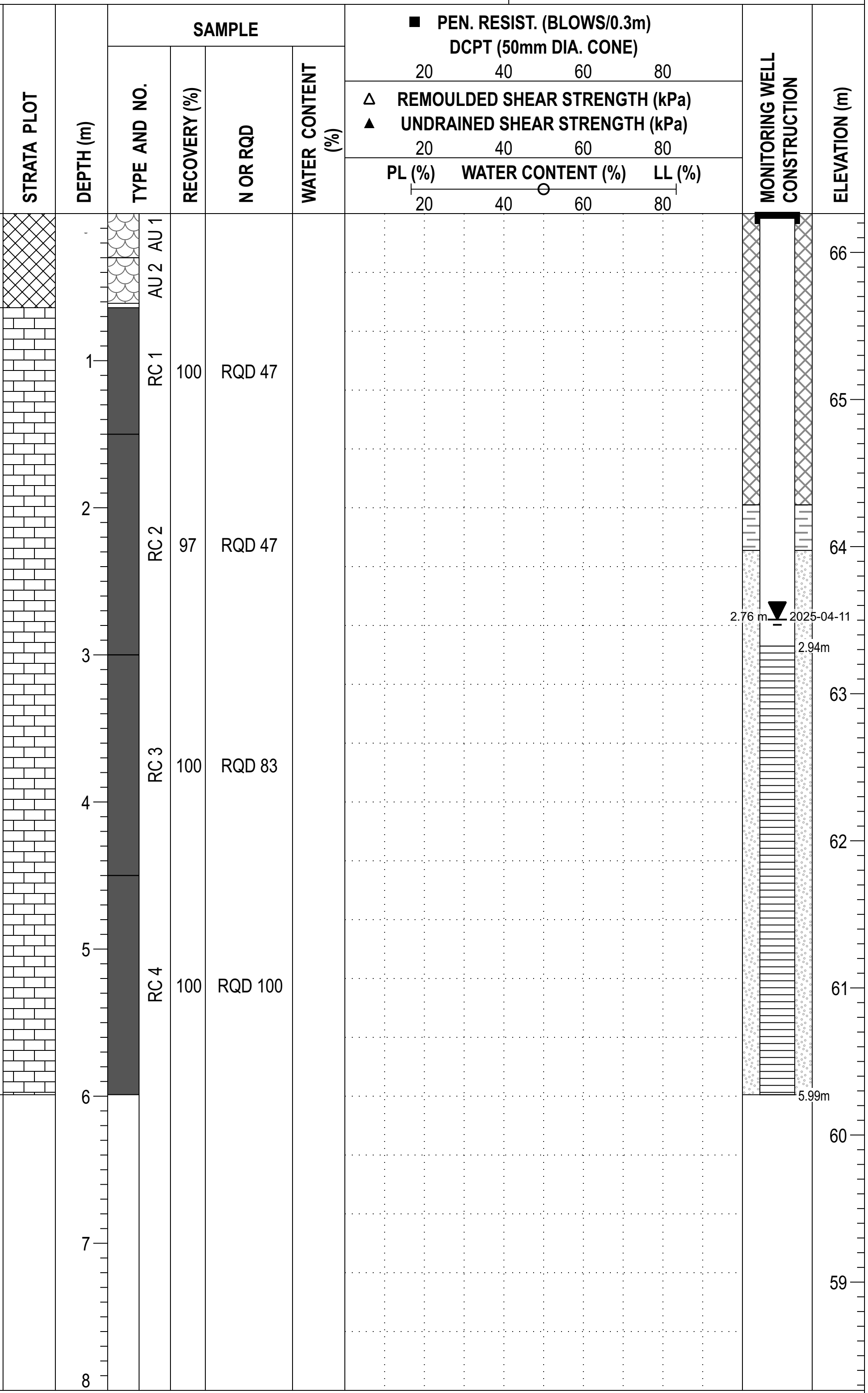
BEDROCK: Poor quality bedrock, limestone and dolomite interbedded

- Good to excellent quality bedrock below 3.0 m depth

5.99m [60.27m]

End of Borehole

(GWL at 2.76 m depth - April 11, 2025)



TYPE AND NO.	RECOVERY (%)	N OR RQD
AU 1		
AU 2		
RC 1	100	RQD 47
RC 2	97	RQD 47
RC 3	100	RQD 83
RC 4	100	RQD 100

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362948.36 **NORTHING:** 5028436.56 **ELEVATION:** 66.29

PROJECT:

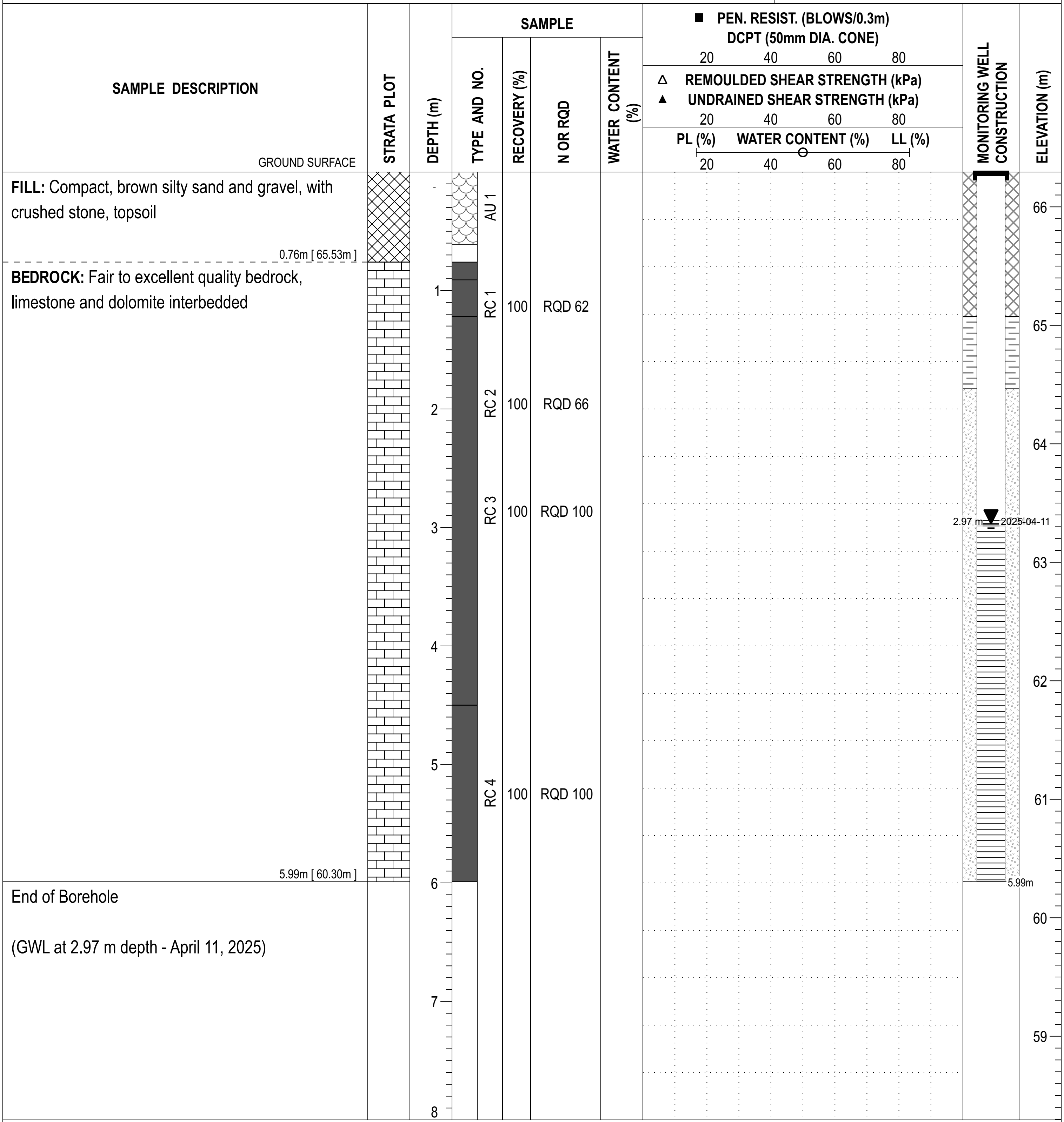
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 1, 2025

FILE NO. : PG2178

HOLE NO. : BH 2-25



FILL: Compact, brown silty sand and gravel, with crushed stone, topsoil

0.76m [65.53m]

BEDROCK: Fair to excellent quality bedrock, limestone and dolomite interbedded

5.99m [60.30m]

End of Borehole

(GWL at 2.97 m depth - April 11, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD
AU 1		
RC 1	100	RQD 62
RC 2	100	RQD 66
RC 3	100	RQD 100
RC 4	100	RQD 100

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362970.93 **NORTHING:** 5028470.25 **ELEVATION:** 66.36

PROJECT:

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 1, 2025

FILE NO. : PG2178

HOLE NO. : BH 3-25

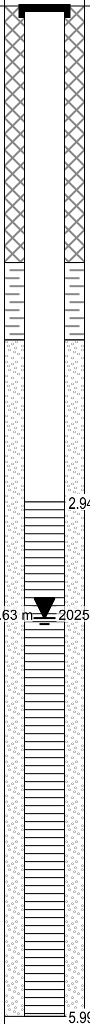
■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							
FILL: Compact, brown silty sand and gravel, with crushed stone, trace clay	▨		AU 1				66
0.86m [65.50m]							
BEDROCK: Fair to excellent quality bedrock, limestone and dolomite interbedded	▦	1	RC 1	84	RQD 56		65
		2	RC 2	100	RQD 66		64
		3	RC 3	100	RQD 100		63
			RC 4	100	RQD 90		
		4					62
		5					61
5.99m [60.37m]							
End of Borehole (GWL at 3.63 m depth - April 11, 2025)		6					60
		7					59
		8					

MONITORING WELL CONSTRUCTION



P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-25, 17:01 Paterson_Template KS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362955.80 **NORTHING:** 5028427.53 **ELEVATION:** 66.63

PROJECT:

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 2, 2025

FILE NO. : **PG2178**

HOLE NO. : **BH 4-25**

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Compact to dense, brown silty sand and gravel, with crushed stone, rock fragments

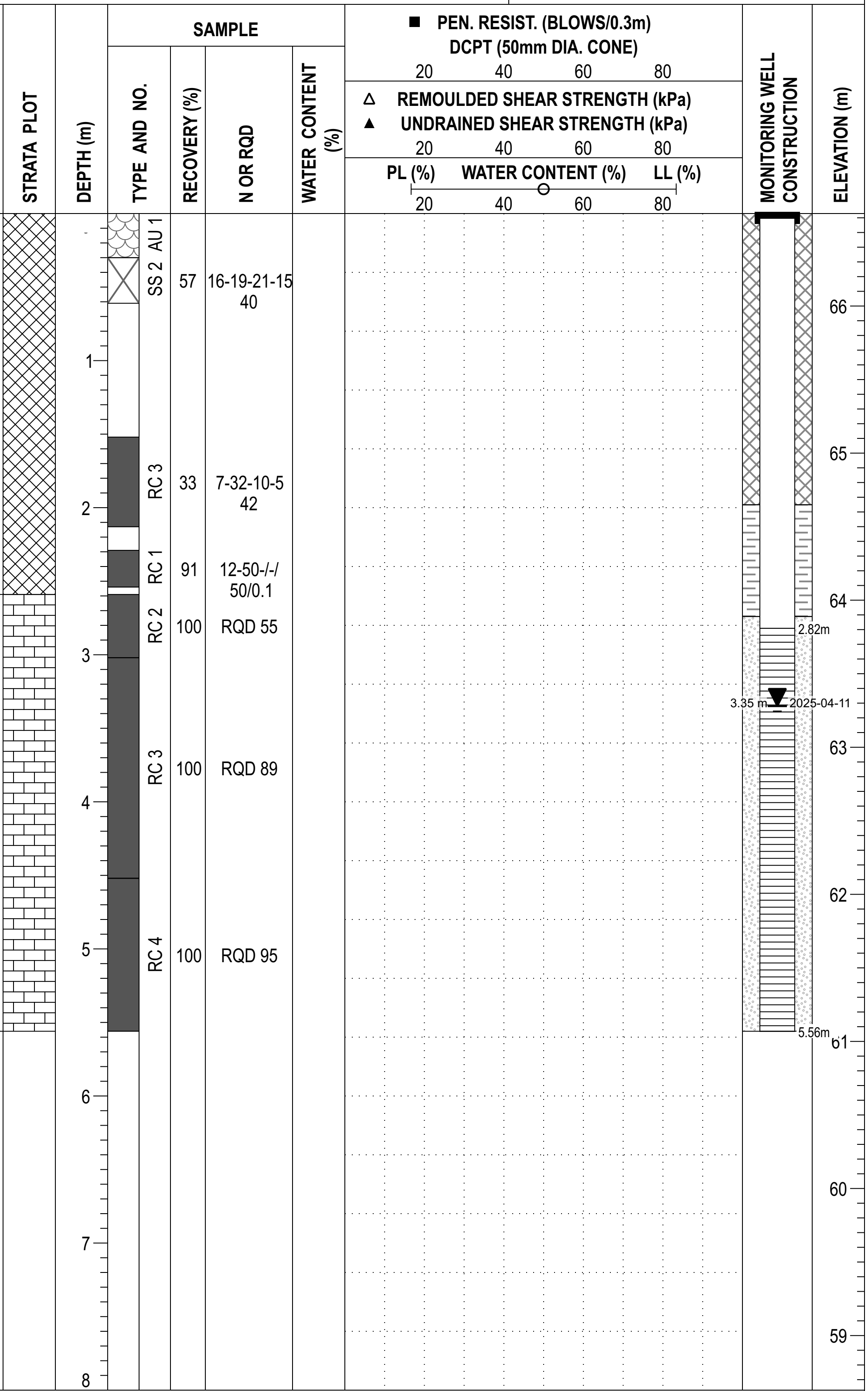
2.59m [64.04m]

BEDROCK: Fair to excellent quality bedrock, limestone and dolomite interbedded

5.56m [61.07m]

End of Borehole

(GWL at 3.35 m depth - April 11, 2025)



TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
SS 2	57	16-19-21-15 40	
RC 3	33	7-32-10-5 42	
RC 1	91	12-50-/-/ 50/0.1	
RC 2	100	RQD 55	
RC 3	100	RQD 89	
RC 4	100	RQD 95	

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-25, 17:01 Paterson_Template KS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362891.67 **NORTHING:** 5028416.89 **ELEVATION:** 66.25

PROJECT:

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 2, 2025

FILE NO. : PG2178

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Compact to dense, brown silty sand and gravel, with crushed stone, topsoil

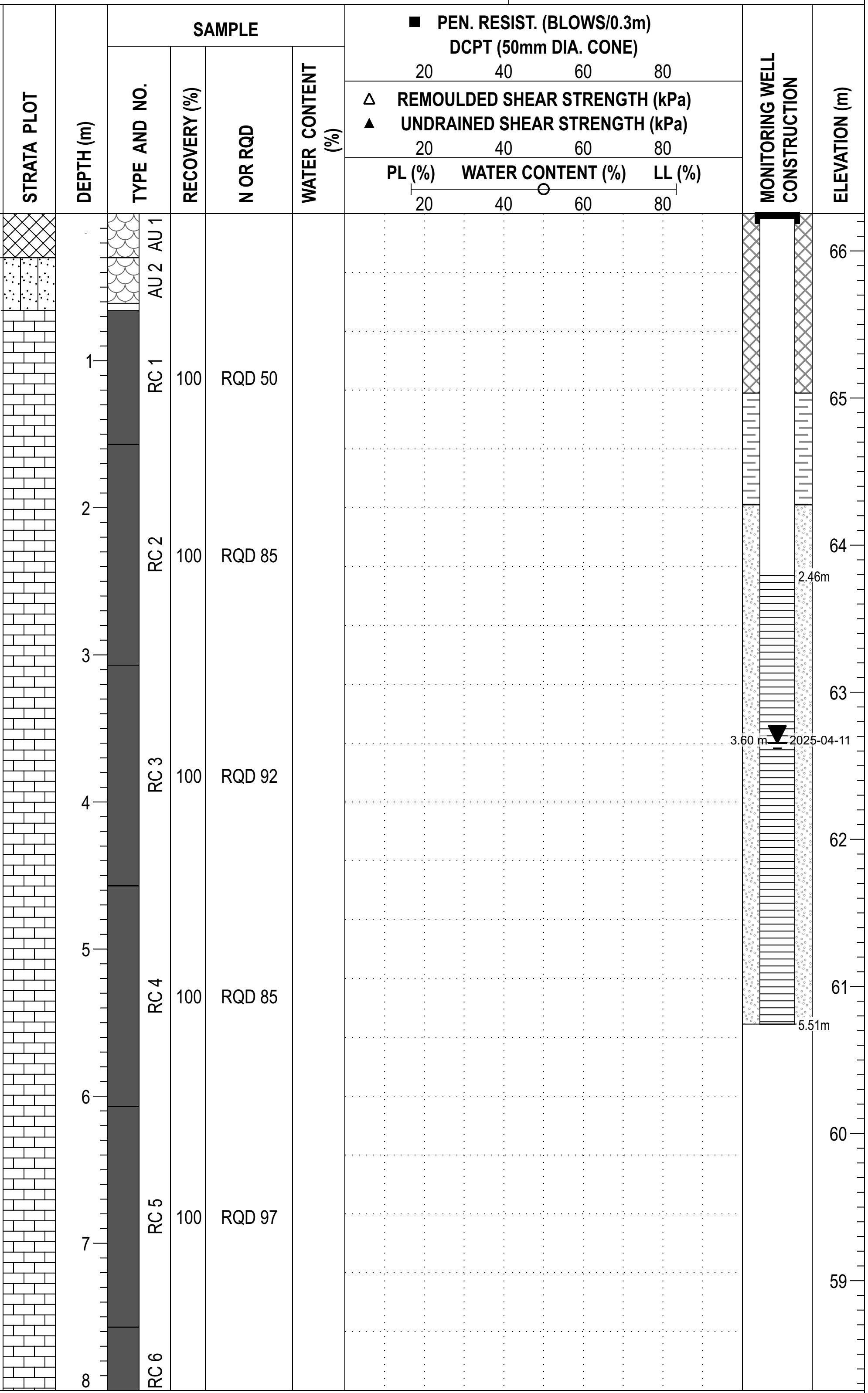
0.30m [65.95m]

Compact, brown **SILTY SAND** with gravel

0.66m [65.59m]

BEDROCK: Fair to excellent quality bedrock, limestone and dolomite interbedded

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
AU 2			
RC 1	100	RQD 50	
RC 2	100	RQD 85	
RC 3	100	RQD 92	
RC 4	100	RQD 85	
RC 5	100	RQD 97	
RC 6			



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362891.67 **NORTHING:** 5028416.89 **ELEVATION:** 66.25

PROJECT:

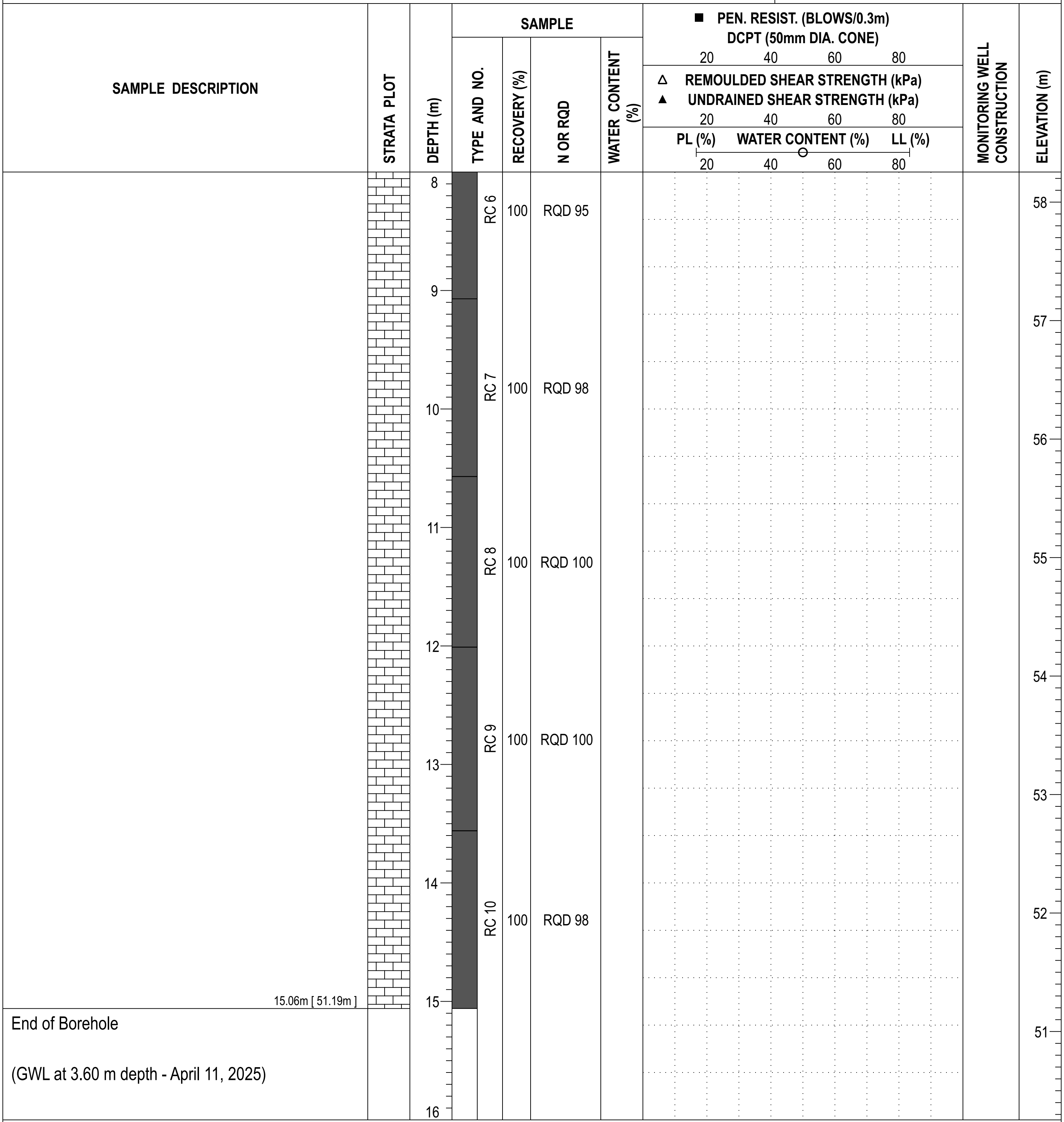
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 2, 2025

FILE NO. : PG2178

HOLE NO. : BH 5-25



TYPE AND NO.	RECOVERY (%)	N OR RQD
RC 6	100	RQD 95
RC 7	100	RQD 98
RC 8	100	RQD 100
RC 9	100	RQD 100
RC 10	100	RQD 98

15.06m [51.19m]

End of Borehole

(GWL at 3.60 m depth - April 11, 2025)

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362907.01 **NORTHING:** 5028402.46 **ELEVATION:** 66.13

PROJECT:

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: April 2, 2025

FILE NO. : PG2178

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL: with organics 0.18m [65.95m]

Compact, brown **SILTY SAND** with gravel 0.56m [65.57m]

BEDROCK: Fair to excellent quality bedrock, limestone and dolomite interbedded

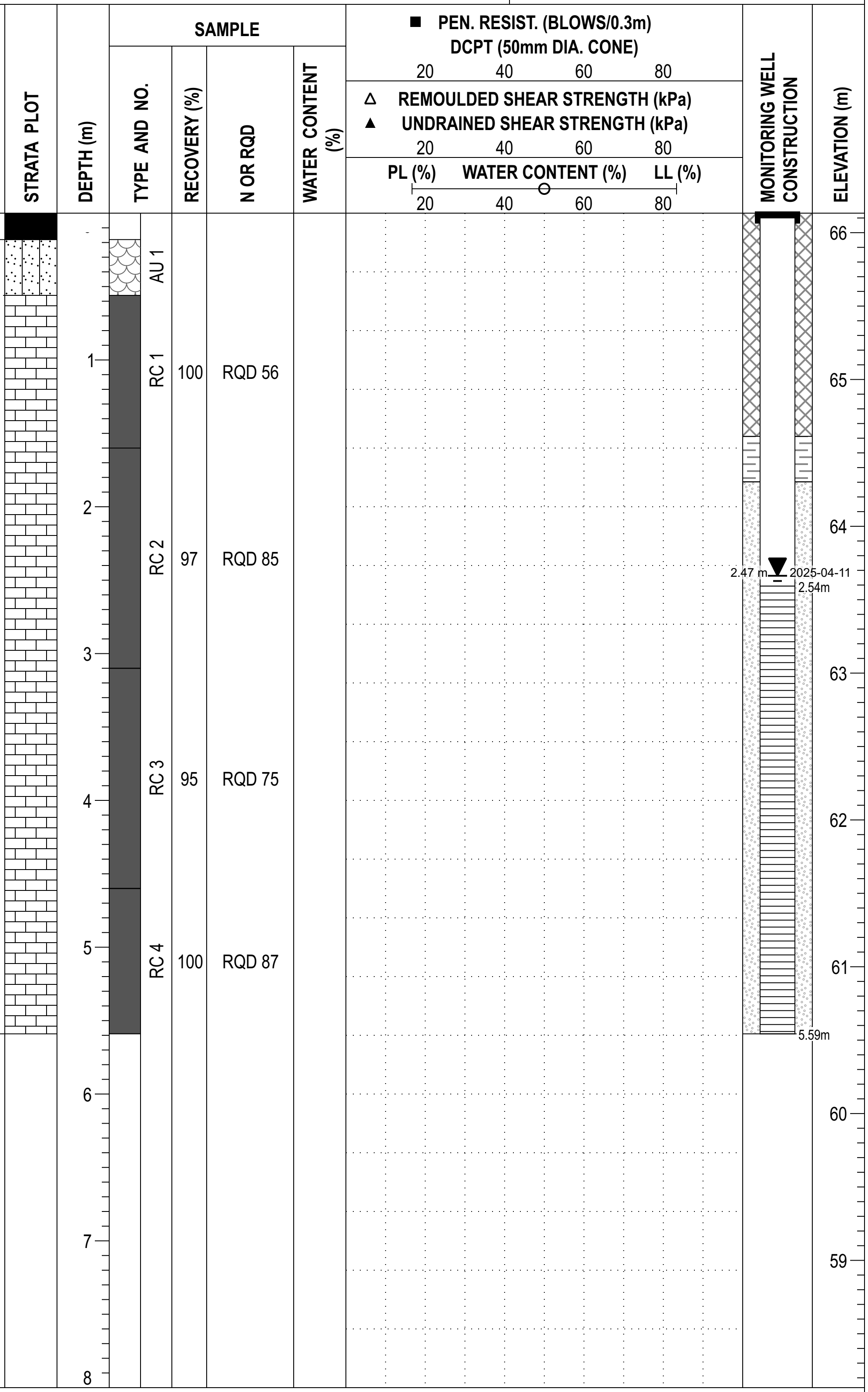
- Vertical fracture from 3.1 to 3.51 m depth

5.59m [60.54m]

End of Borehole

(GWL at 2.47 m depth - April 11, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
RC 1	100	RQD 56	
RC 2	97	RQD 85	
RC 3	95	RQD 75	
RC 4	100	RQD 87	



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362969.22 **NORTHING:** 5028491.31 **ELEVATION:** 66.42

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 1-25

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									
FILL: Compact, crushed stone 0.10m [66.32m]	▨								
FILL: Compact, brown fine to medium sand and clay with some gravel 0.30m [66.12m]	▨		G 1 G 2						
FILL: Compact, dark brown, fine to medium sand with clay, trace topsoil, organics 0.45m [65.97m]	▨								66
FILL: Compact, brown, silty fine sand with some gravel, trace clay 0.85m [65.57m]	▨		G 3						
End of Test Pit Practical refusal on bedrock at 0.85 m depth		1							65
		2							64
		3							63
		4							

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362957.73 **NORTHING:** 5028475.73 **ELEVATION:** 66.21

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 2-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Loose to compact, crushed stone

0.10m [66.11m]

FILL: Compact, brown medium sand with silt and gravel, crushed stone

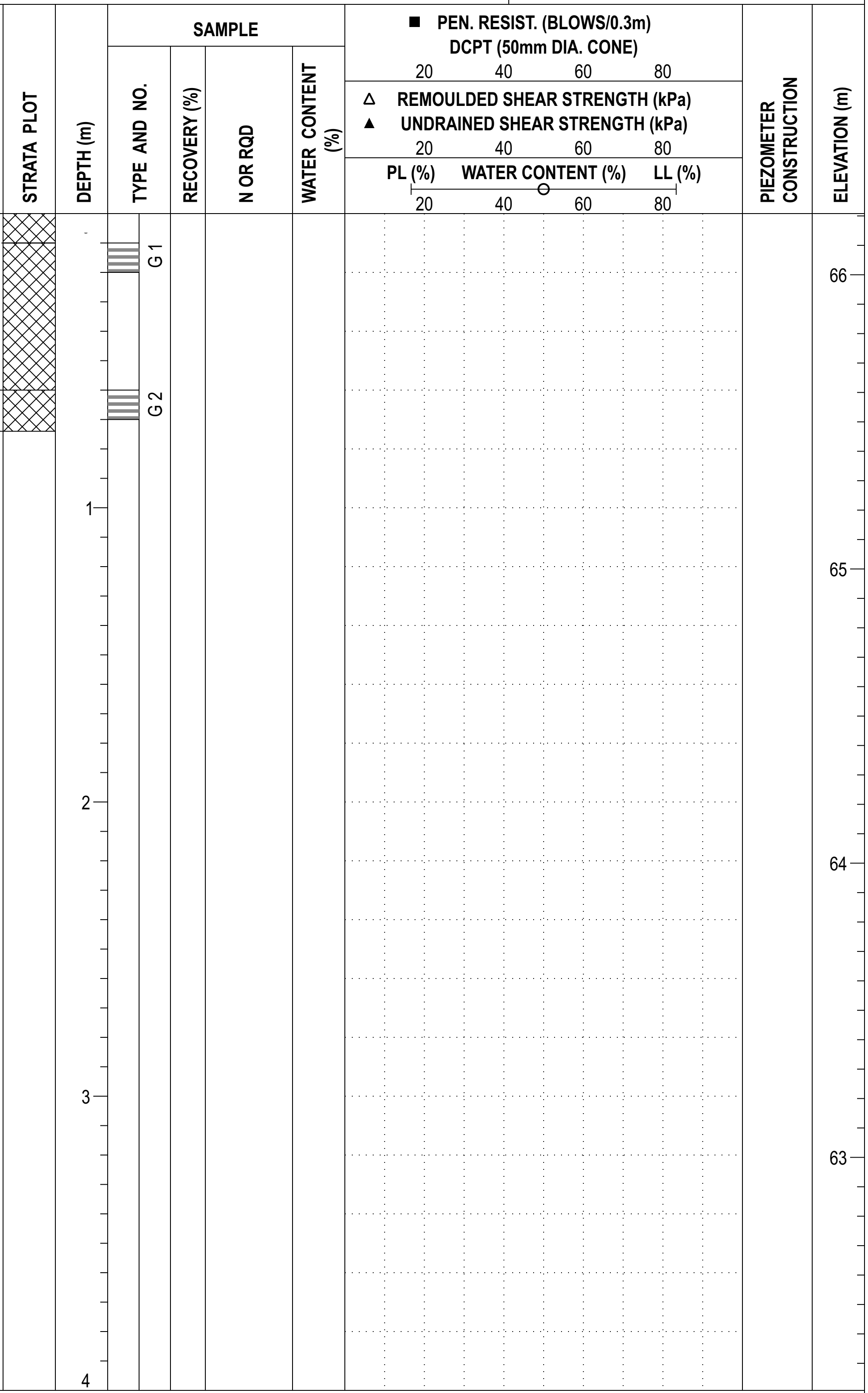
0.60m [65.61m]

FILL: Compact, light brown, fine silty sand with gravel, cobbles

0.74m [65.47m]

End of Test Pit

Practical refusal on bedrock at 0.74 m depth



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362960.01 **NORTHING:** 5028455.22 **ELEVATION:** 66.27

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 3-25

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE 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)
FILL: Compact, crushed stone 0.10m [66.17m]									
FILL: Compact, brown medium to coarse sand with gravel, crushed stone and occasional cobbles 0.25m [66.02m]			G 1						
FILL: Compact, dark brown, fine to medium sand with silt, gravel and crushed stone, trace topsoil 0.65m [65.62m]			G 2						
Compact, light brown **SILTY SAND** with gravel and cobbles 0.82m [65.45m]			G 3						
End of Test Pit Practical refusal on bedrock at 0.82 m depth		1							66
		2							65
		3							64
		4							63

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-10, 09:38 Paterson_Template KS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362964.39 **NORTHING:** 5028431.66 **ELEVATION:** 66.78

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 4-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE 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									
FILL: Compact, crushed stone 0.24m [66.54m]	▨								
FILL: Compact, dark brown medium sand with some gravel, crushed stone 0.95m [65.83m]	▨		G 1 G 2						66
End of Test Pit Practical refusal on bedrock at 0.95 m depth		1							
									65
		2							
									64
		3							
									63
		4							

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362958.70 **NORTHING:** 5028422.84 **ELEVATION:** 66.68

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 5-25

SAMPLE DESCRIPTION	DEPTH (m)	ELEVATION (m)
GROUND SURFACE		
FILL: Compact, crushed stone	0.10m	[66.58m]
FILL: Compact, blasted rock with dark brown silty fine to medium sand, gravel, cobbles and crushed stone	0.25m	[66.43m]
FILL: Compact, crushed stone	0.50m	[66.18m]
FILL: Compact, blasted rock with dark brown silty fine to medium sand, gravel, cobbles and crushed stone, trace clay	3.00m	[63.68m]

Samples (TYPE AND NO.): G 1, G 2, G 3, G 4, G 5

Column headings: STRATA PLOT; DEPTH (m); SAMPLE: TYPE AND NO., RECOVERY (%), N OR RQD, WATER CONTENT (%); PIEZOMETER CONSTRUCTION; ELEVATION (m)

■ 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

Depth scale: 1, 2, 3, 4

Elevation scale: 66, 65, 64, 63

End of Test Pit

Test pit terminated at 3.0m depth due to water infiltration and side walls collapsing into test pit

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362946.53 **NORTHING:** 5028427.77 **ELEVATION:** 66.36

PROJECT:

ADVANCED BY: Back Hoe

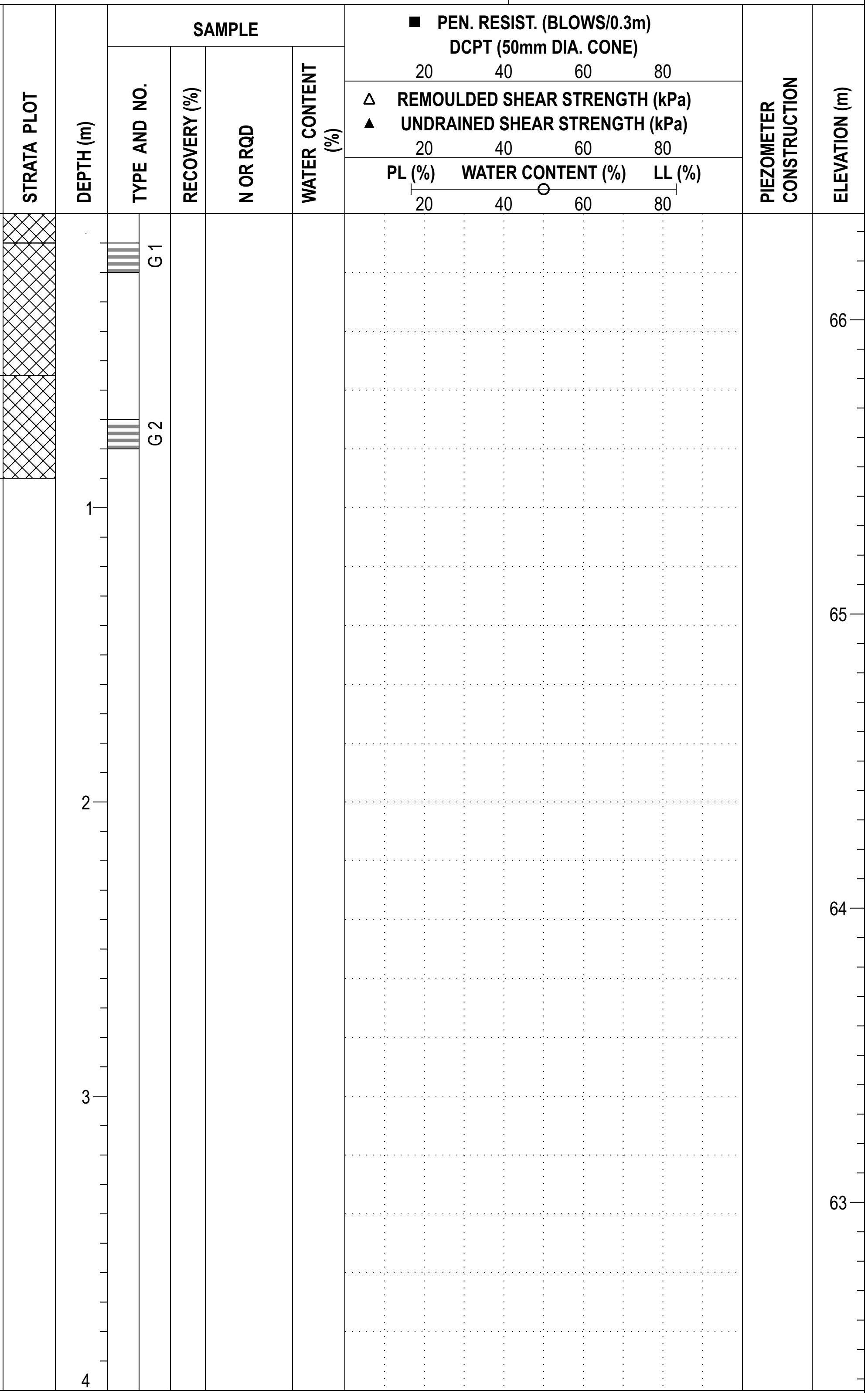
REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 6-25

SAMPLE DESCRIPTION	
GROUND SURFACE	
FILL: Compact, crushed stone	0.10m [66.26m]
FILL: Compact, brown medium sand with some gravel and cobbles	0.55m [65.81m]
FILL: Compact, light brown, silty fine sand with some gravel and cobbles	0.90m [65.46m]
End of Test Pit Practical refusal on bedrock at 0.9 m depth	



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.

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-10, 09:38 Paterson_Template KS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362931.86 **NORTHING:** 5028422.09 **ELEVATION:** 66.33

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 7-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE 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									
FILL: Compact, crushed stone 0.10m [66.23m]									
FILL: Compact, dark brown medium sand with some silt and gravel 0.60m [65.73m]			G 1						66
Compact, light brown **SILTY** fine **SAND** with gravel and cobbles 0.90m [65.43m]			G 2						
End of Test Pit Practical refusal on bedrock at 0.9 m depth		1 2 3 4							65 64 63

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362936.14 **NORTHING:** 5028459.77 **ELEVATION:** 66.18

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 8-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Compact, crushed stone

0.15m [66.03m]

FILL: Compact, topsoil, some gravel and crushed stone

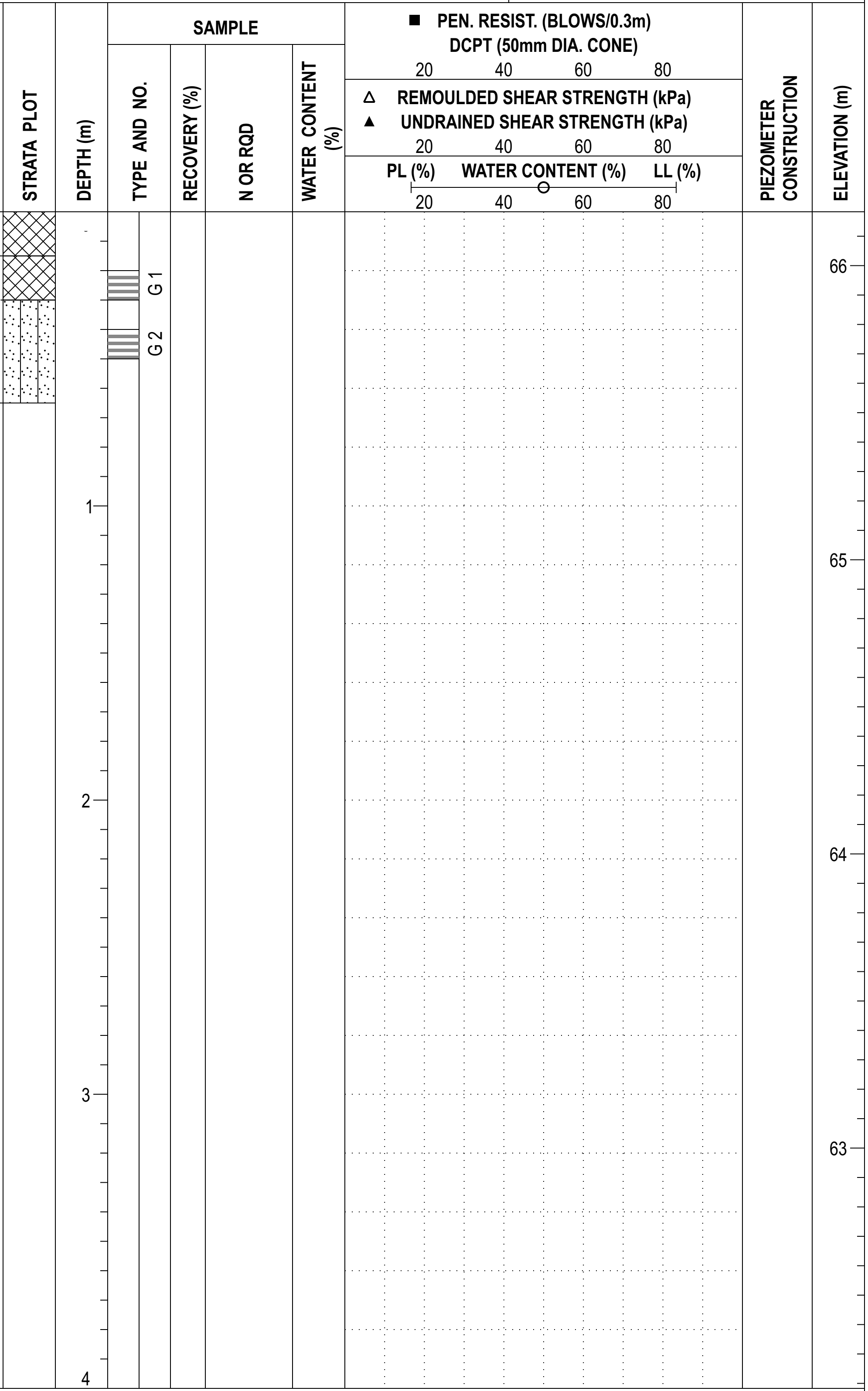
0.30m [65.88m]

Compact, light brown **SILTY** fine **SAND** with some cobbles and gravel

0.65m [65.53m]

End of Test Pit

Practical refusal on bedrock at 0.65 m depth



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362944.77 **NORTHING:** 5028445.96 **ELEVATION:** 66.25

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP 9-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Loose to compact, crushed stone

0.15m [66.10m]

FILL: Compact, brown silty fine to medium sand, gravel, trace clay

0.45m [65.80m]

TOPSOIL: trace gravel and clay

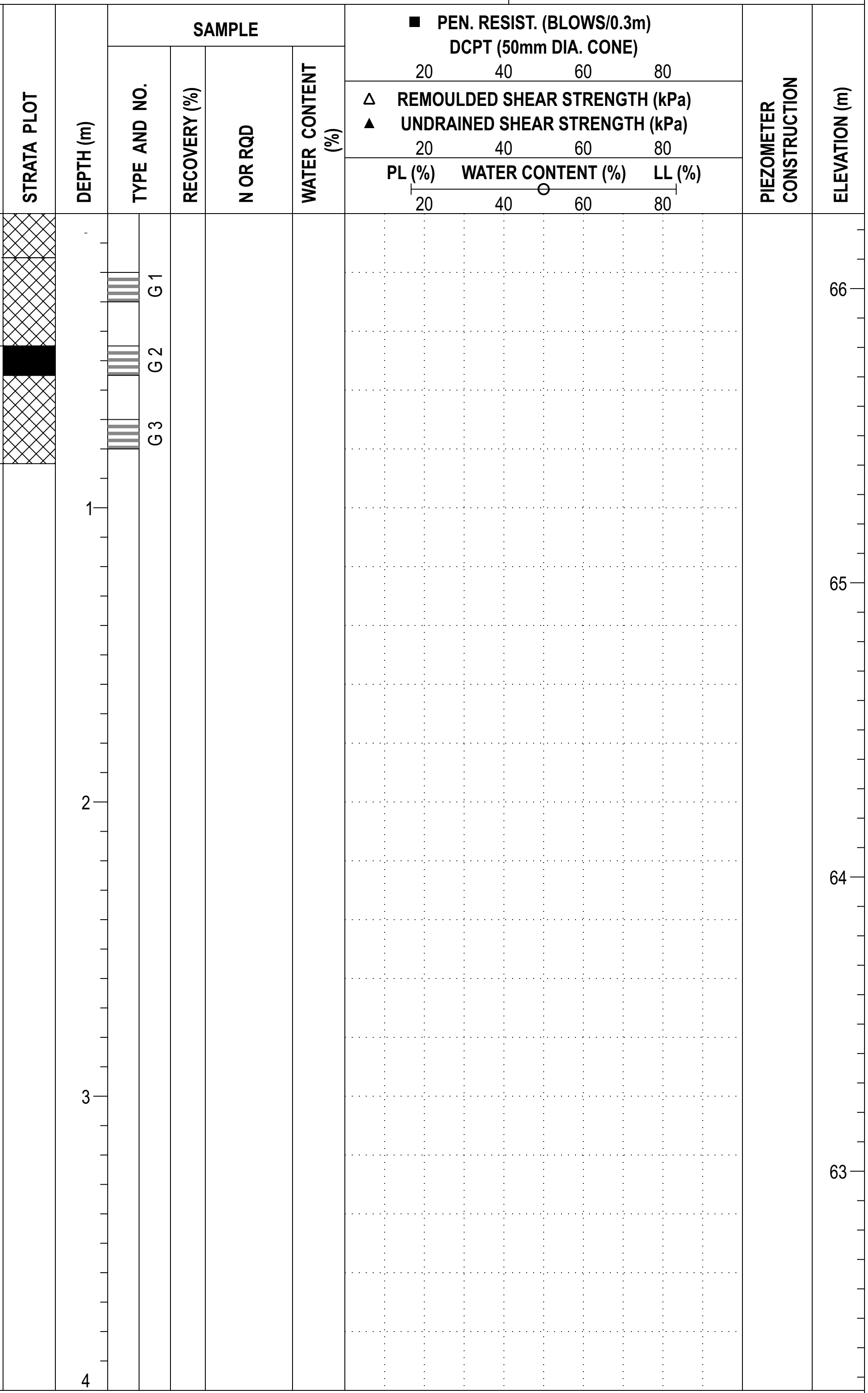
0.55m [65.70m]

FILL: Compact, light brown silty fine to medium sand, gravel, cobbles and crushed stone, trace clay

0.85m [65.41m]

End of Test Pit

Practical refusal on bedrock at 0.85 m depth



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.

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362918.26 **NORTHING:** 5028443.88 **ELEVATION:** 66.18

PROJECT:

ADVANCED BY: Back Hoe

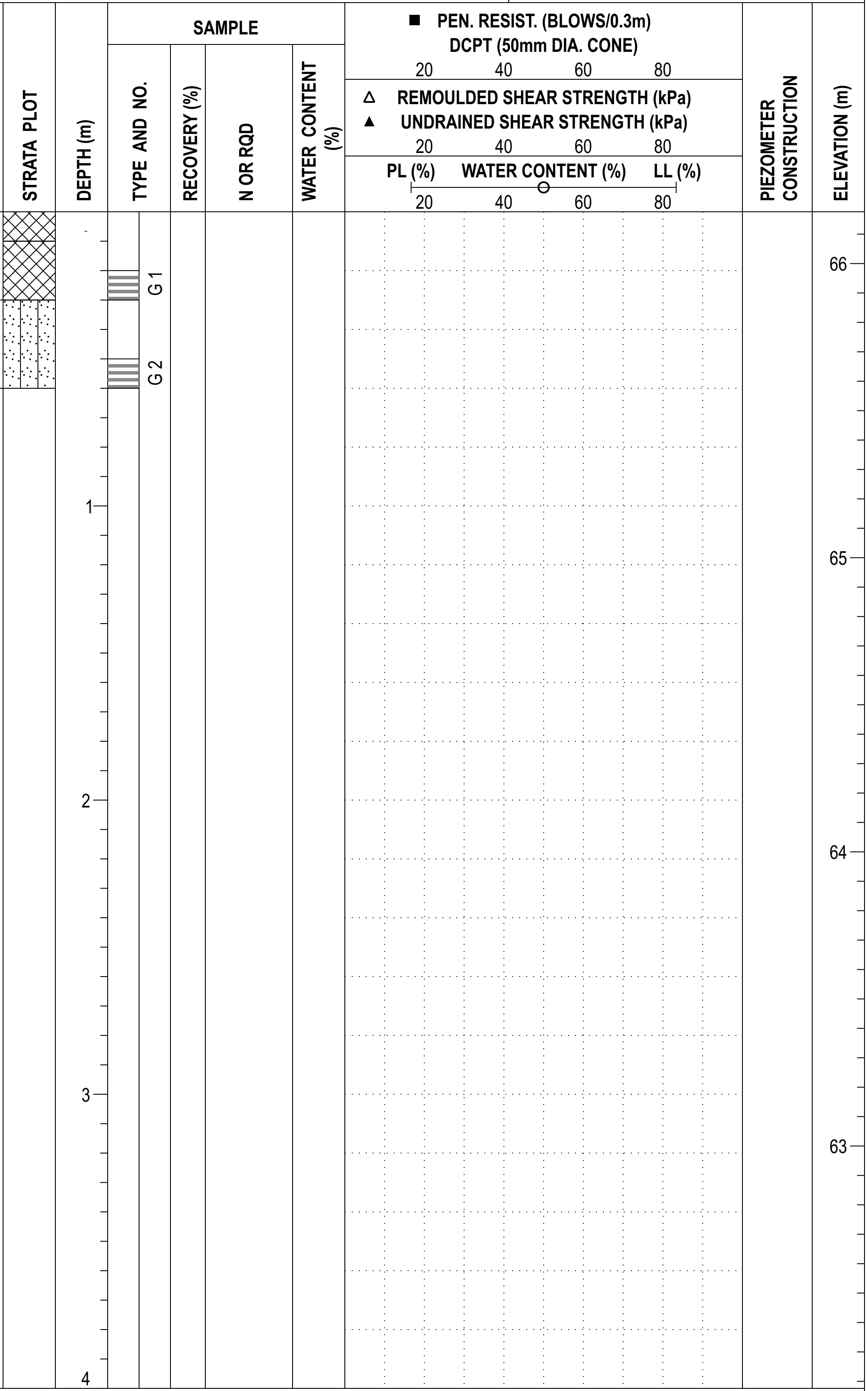
REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP10-25

SAMPLE DESCRIPTION						
GROUND SURFACE						
FILL: Compact, crushed stone	0.10m [66.08m]					
FILL: Compact, dark brown silty clay with topsoil, sand, gravel, crushed stone	0.30m [65.88m]	G 1				
Compact, light brown **SILTY** fine **SAND** with gravel	0.60m [65.58m]	G 2				
End of Test Pit Practical refusal on bedrock at 0.6 m depth						



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.

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-10, 09:38 Paterson_Template KS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362906.53 **NORTHING:** 5028428.17 **ELEVATION:** 66.11

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

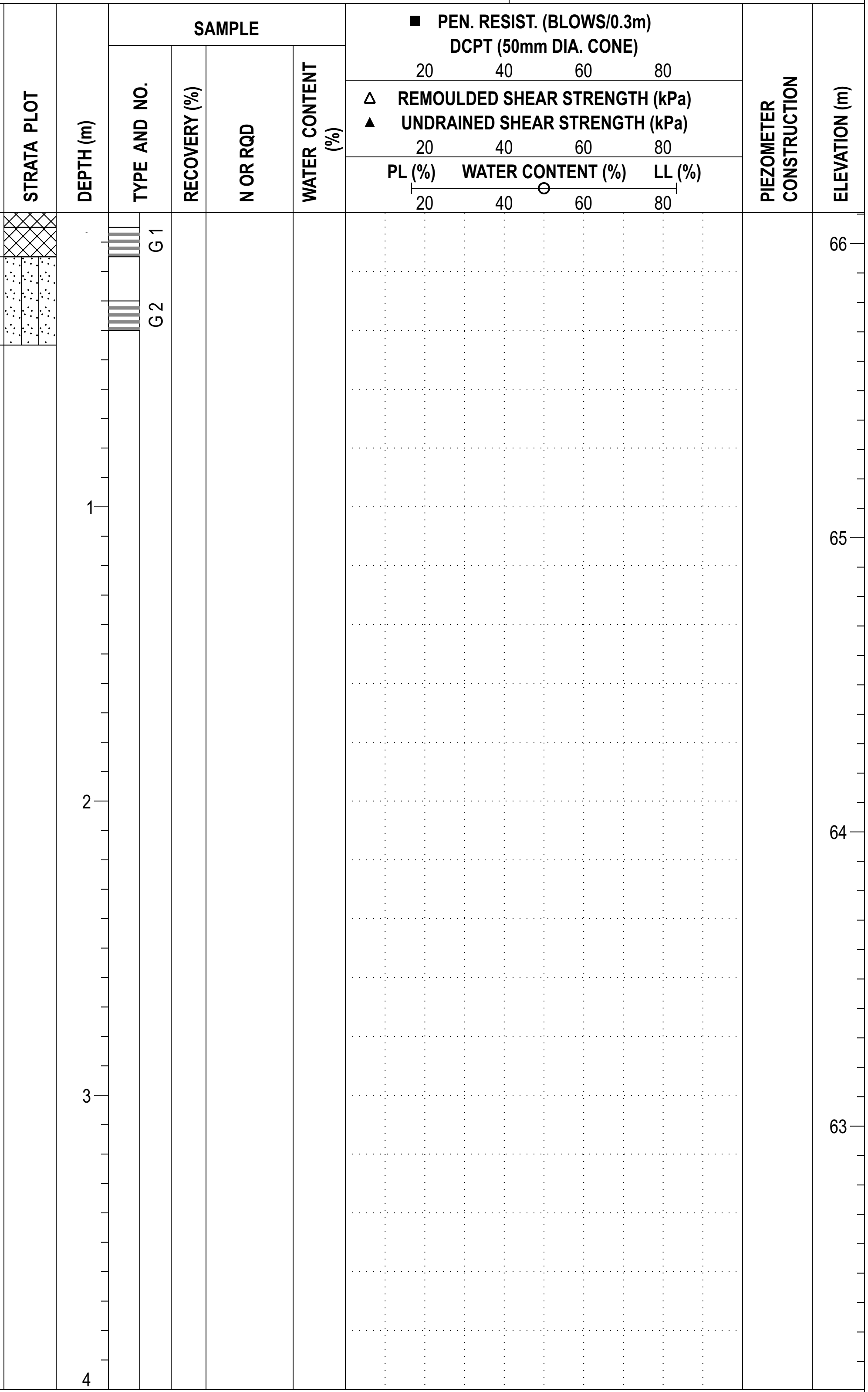
FILE NO. : PG2178

HOLE NO. : TP11-25

SAMPLE DESCRIPTION	Depth [Elevation]
GROUND SURFACE	
FILL: Compact, crushed stone	0.05m [66.06m]
FILL: Compact, dark brown silty clay with topsoil, sand, gravel, crushed stone	0.15m [65.95m]
Compact, light brown **SILTY** fine **SAND** with gravel	0.45m [65.66m]

End of Test Pit

Practical refusal on bedrock at 0.45 m depth



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362875.60 **NORTHING:** 5028398.22 **ELEVATION:** 66.24

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP12-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE 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									
0.05m [66.19m]									
FILL: Compact, dark brown silty clay with topsoil, sand, gravel, crushed stone	☒		G 1						66
0.15m [66.09m]									
Compact, light brown **SILTY** fine **SAND** with gravel	▨		G 2						
0.45m [65.79m]									
End of Test Pit									
Practical refusal on bedrock at 0.45 m depth		1							65
		2							64
		3							63
		4							

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362953.45 **NORTHING:** 5028436.33 **ELEVATION:** 66.42

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : PG2178

HOLE NO. : TP13-25

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									
FILL: Compact, crushed stone 0.10m [66.32m]									
FILL: Compact, dark brown silty fine sand with some blast rock, gravel, trace topsoil and clay 0.75m [65.67m]			G 1 G 2						66
Compact, light brown **SILTY** fine **SAND** with gravel, cobbles, trace clay 1.05m [65.37m]		1	G 3						
End of Test Pit Practical refusal on bedrock at 1.05 m depth		2 3 4							65 64 63

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-10, 09:38 Paterson_Template KS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

335 Roosevelt Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362976.95 **NORTHING:** 5028462.62 **ELEVATION:** 66.41

PROJECT:

ADVANCED BY: Back Hoe

REMARKS:

DATE: April 4, 2025

FILE NO. : **PG2178**

HOLE NO. : **TP14-25**

■ 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

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE								
FILL: Loose to compact, crushed stone 0.20m [66.21m]	☒							
FILL: Compact, dark brown silt, medium sand, gravel, cobbles and crushed stone, trace clay 0.45m [65.96m]	☒		G 1					66
Compact, light brown **SILTY** fine **SAND** with gravel, trace clay 1.00m [65.41m]	⁘	1	G 2					
End of Test Pit Practical refusal on bedrock at 1.05 m depth								65
		2						64
		3						63
		4						

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.

P:/AutoCAD Drawings/Test Hole Data Files/PG21xx/PG2178/data.sqlite 2025-04-10, 09:38 Paterson_Template KS

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 1

SOIL DESCRIPTION

GROUND SURFACE

Asphaltic concrete 0.05

FILL: Dark brown silty sand, some gravel 0.46

0.74

Grey-brown **SILTY CLAY,** some sand

BEDROCK: Grey limestone

- black shaley limestone from 1.5 to 2.5m depth

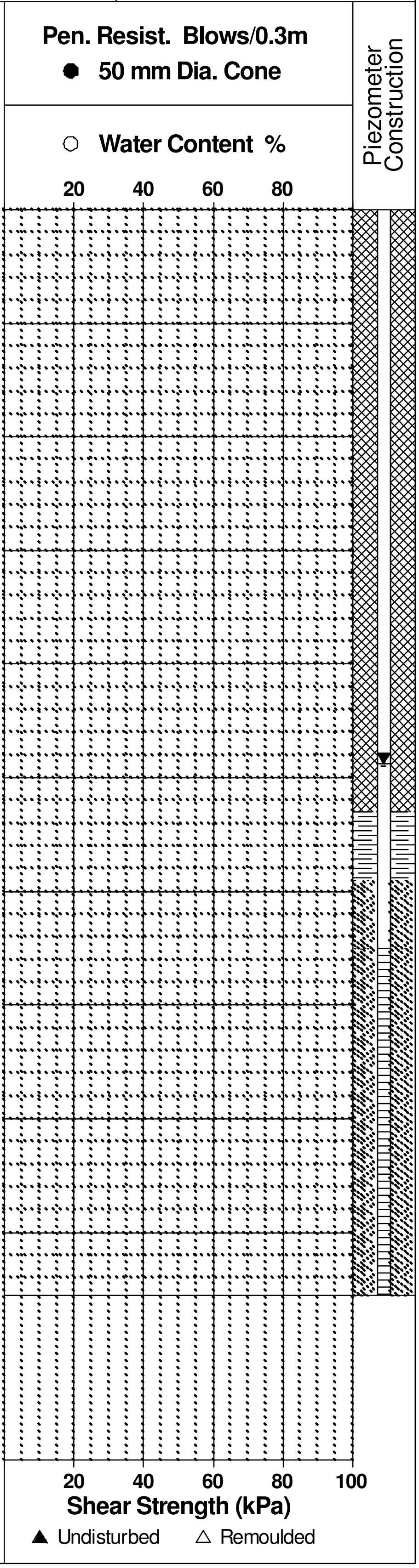
9.47

End of Borehole

(GWL @ 4.88m-Nov. 16/10)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2		50+
RC	1	94	22
RC	2	100	51
RC	3	100	78
RC	4	100	84
RC	5	100	35
RC	6	100	96
RC	7	100	100

DEPTH (m)	ELEV. (m)
0	66.39
1	65.39
2	64.39
3	63.39
4	62.39
5	61.39
6	60.39
7	59.39
8	58.39
9	57.39



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 2

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Dark brown silty sand, some gravel	0.46
Grey-brown **SILTY CLAY**	0.60
BEDROCK: Grey limestone - black shaley limestone from 1.6m to 2.5m depth	9.42
End of Borehole (GWL @ 6.53m-Nov. 16/10)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	0	50+
RC	1	84	58
RC	2	100	57
RC	3	98	83
RC	4	100	92
RC	5	98	98
RC	6	100	93
SS	7	100	100

DEPTH (m)	ELEV. (m)
0	66.37
1	65.37
2	64.37
3	63.37
4	62.37
5	61.37
6	60.37
7	59.37
8	58.37
9	57.37

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

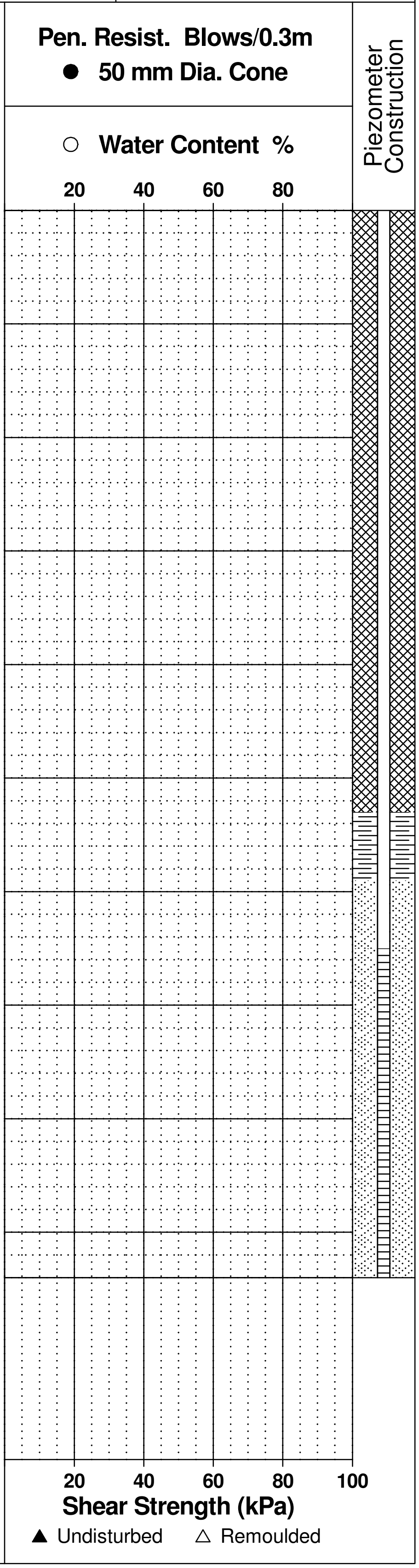
BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.43
Asphaltic concrete	0.05						
FILL; Brown silty sand with gravel	0.60	AU	1				
FILL: Light brown silty sand, some gravel, trace clay	0.97	SS	2	0	50+	1	65.43
BEDROCK: Grey limestone - black shaley limestone from 1.5m to 1.7m depth		RC	1	88	63		
		RC	2	97	35	2	64.43
						3	63.43
		RC	3	100	75		
						4	62.43
		RC	4	98	87	5	61.43
						6	60.43
		RC	5	98	85		
						7	59.43
		RC	6	100	89	8	58.43
		RC	7	96	96	9	57.43
	9.40						
End of Borehole (BH dry - Nov. 16/10)							



patersongroup

Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

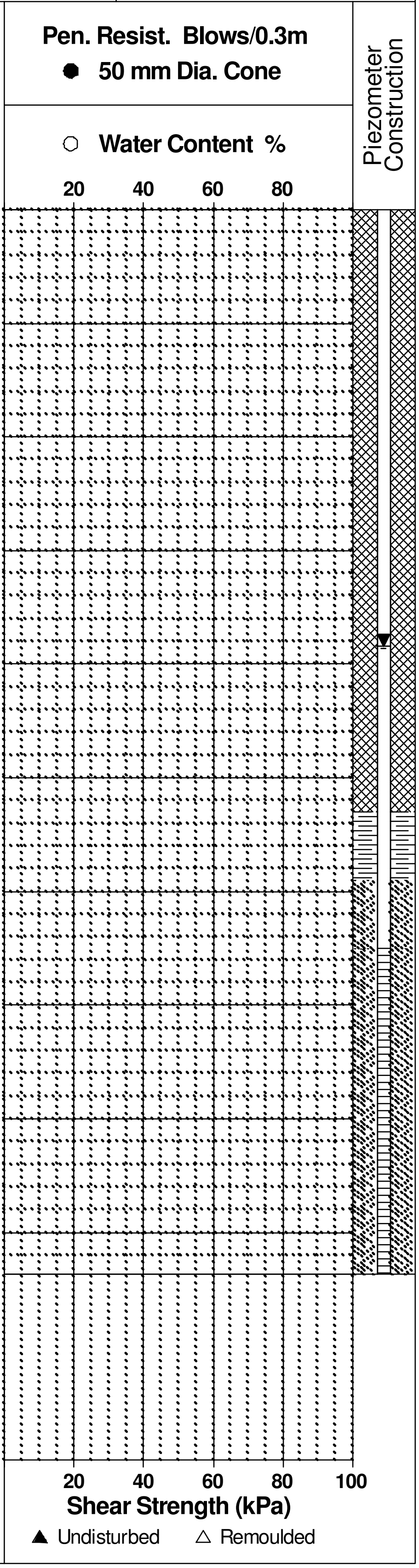
BORINGS BY CME 55 Power Auger

DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.64
Asphaltic concrete 0.04						
Crushed stone 0.15	AU	1				
FILL: Brown silty sand, some gravel 0.91						
Grey-brown **SILT** 1.17	SS	2	90	10	1	65.64
BEDROCK: Shaley limestone	RC	1	92	36	2	64.64
3.00					3	63.64
	RC	2	100	55		
					4	62.64
BEDROCK: Grey limestone	RC	3	100	60	5	61.64
					6	60.64
	RC	4	100	93		
					7	59.64
	RC	5	100	92	8	58.64
					9	57.64
	RC	6	100	100		
9.37						
End of Borehole						
(GWL @ 3.84m-Nov. 16/10)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

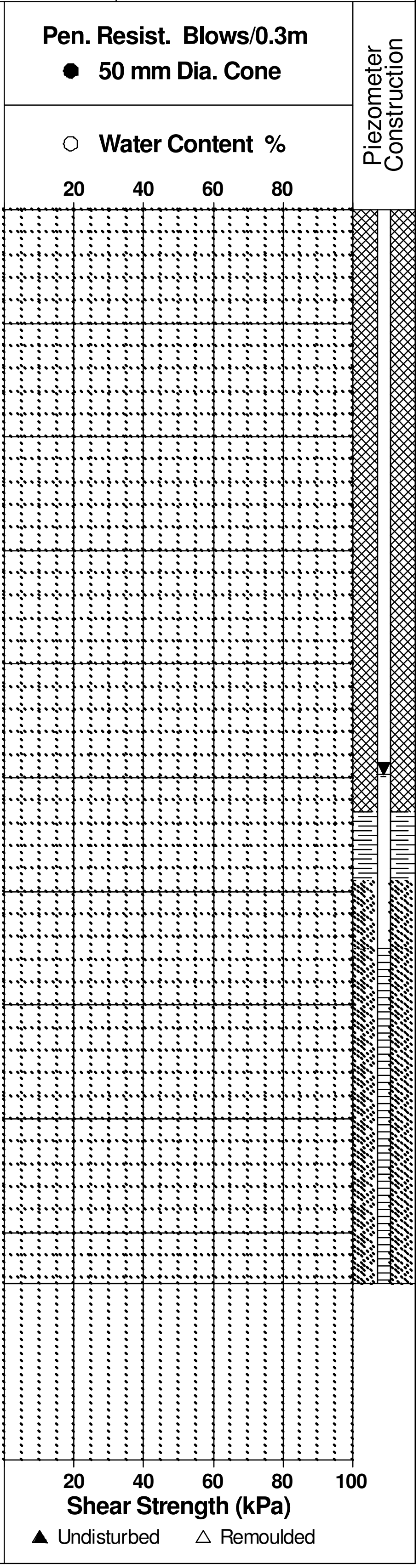
DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Dark brown silty sand with gravel, trace metals, brick and asphalt — 0.76	SS	1	50	8
FILL: Black silty sand — 0.94	SS	2	78	50+
Light brown **SILT** — 0.99	RC	1	86	0
BEDROCK: Grey limestone - shaley limestone from 1.9m to 2.0m depth	RC	2	97	45
	RC	3	98	85
	RC	4	100	77
	RC	5	100	95
	RC	6	100	82
	RC	7	100	83
9.40 End of Borehole (GWL @ 4.97m-Nov. 16/10)				

DEPTH (m)	ELEV. (m)
0	66.50
1	65.50
2	64.50
3	63.50
4	62.50
5	61.50
6	60.50
7	59.50
8	58.50
9	57.50



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 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

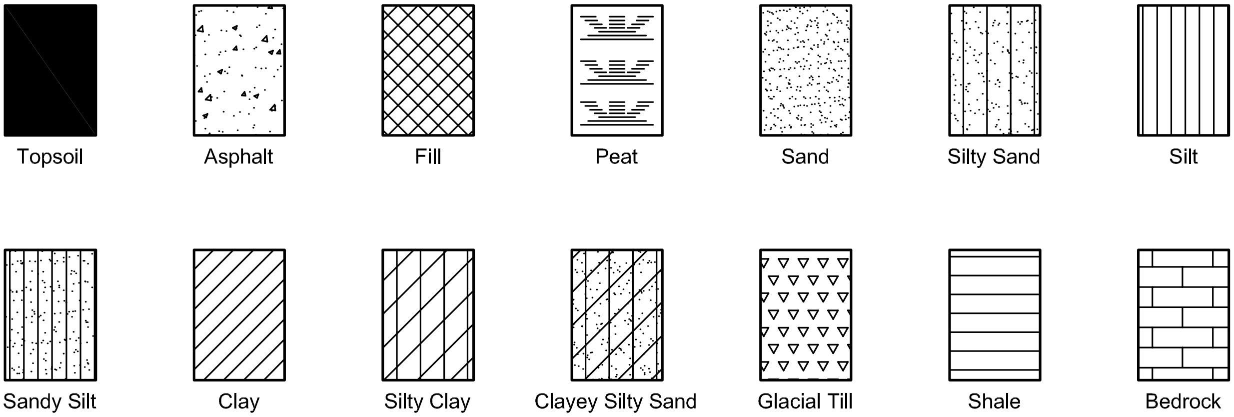
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

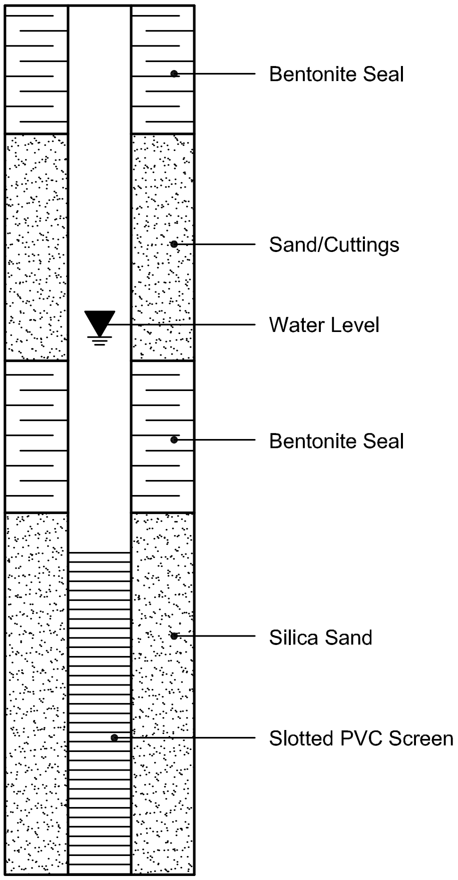
SYMBOLS AND TERMS (continued)

STRATA PLOT

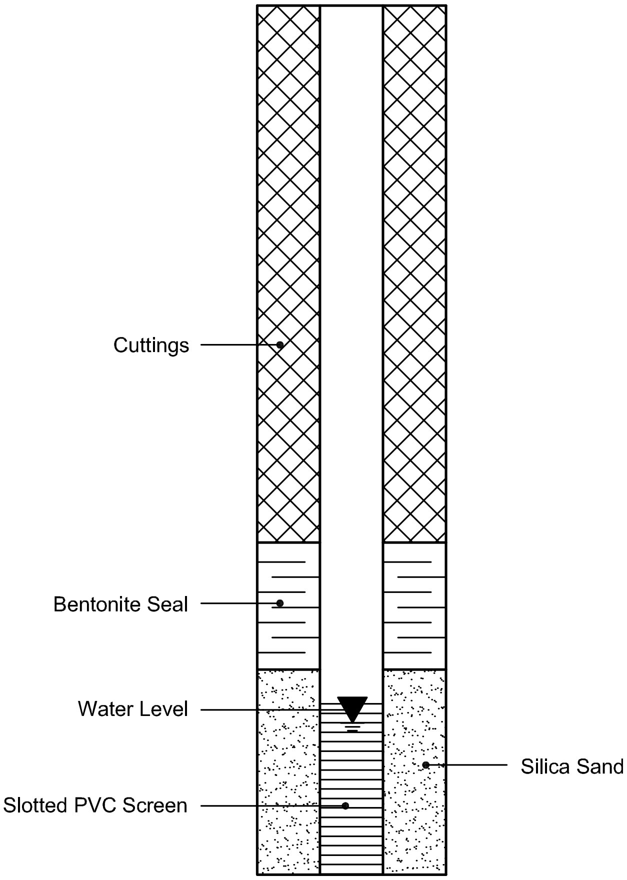


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 15-Apr-2025

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

Order Date: 9-Apr-2025

Client PO: 62814

Project Description: PG2178

	Client ID:	TP13-25 G3	-	-	-	-	-
	Sample Date:	04-Apr-25 09:00	-	-	-		
	Sample ID:	2515330-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	83.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.58	-	-	-	-	-
Resistivity	0.1 Ohm.m	51.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	10	-	-	-	-	-
Sulphate	10 ug/g	28	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG2178-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP

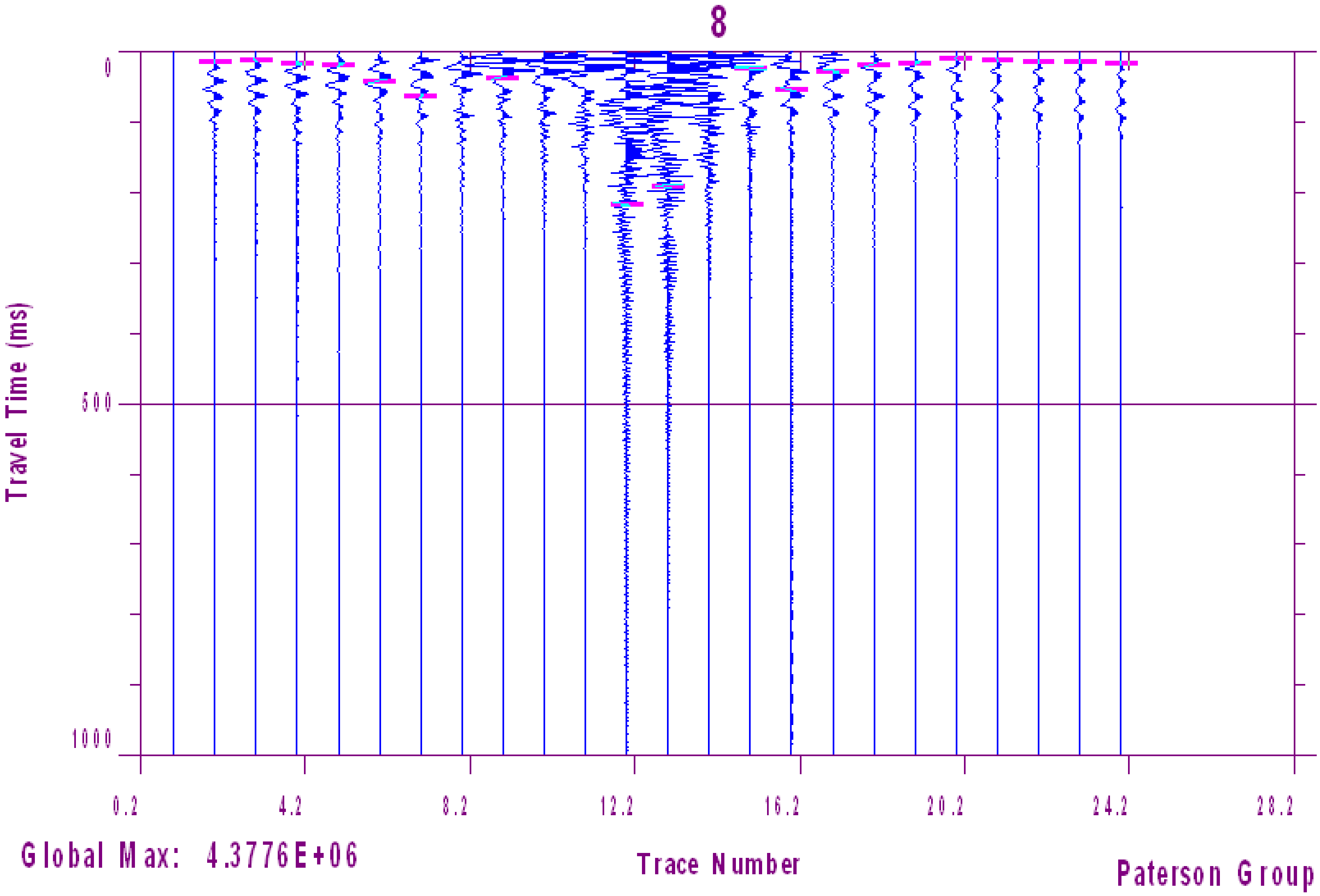


Figure 2 – Shear Wave Velocity Profile at Shot Location 11.5 m

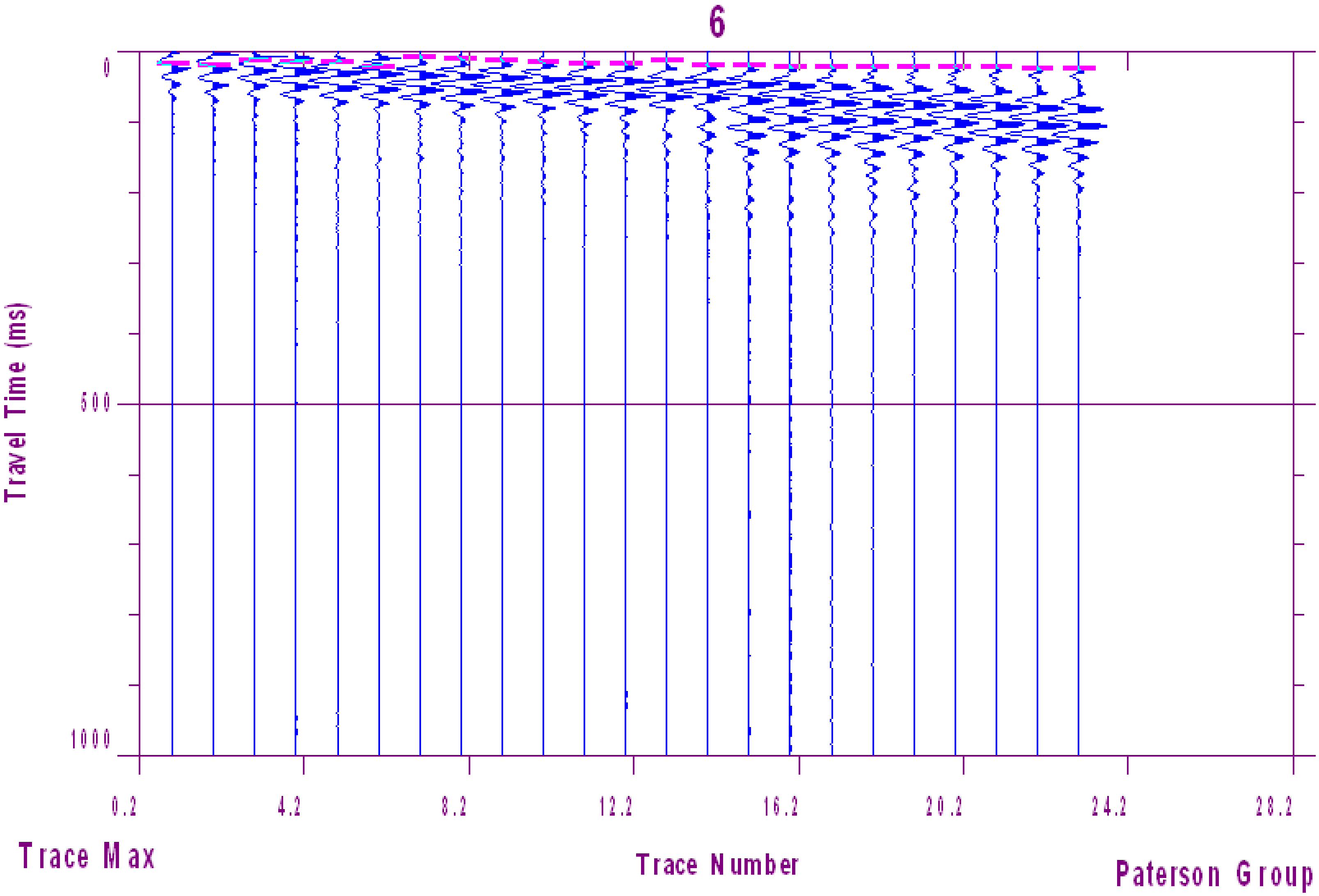
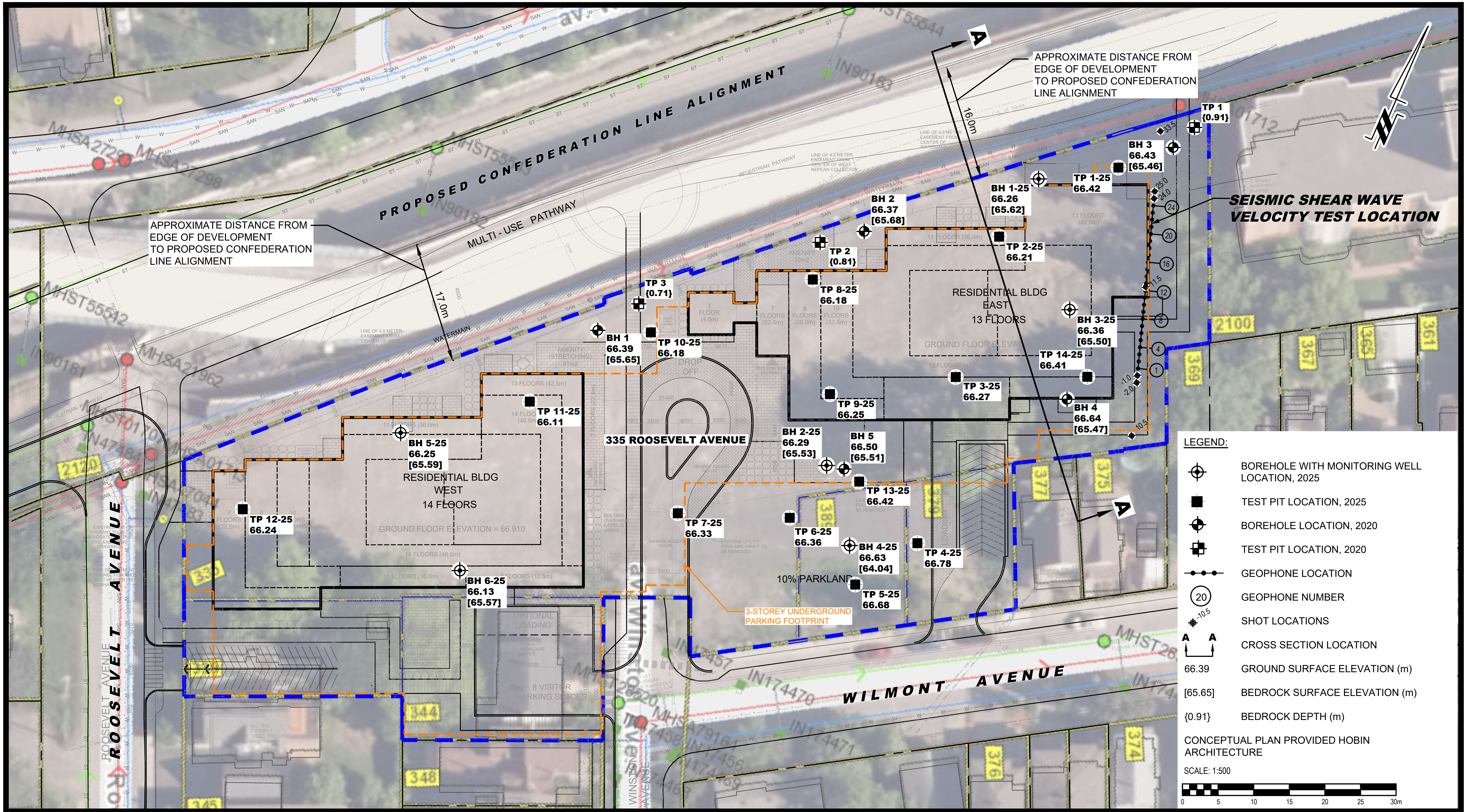


Figure 3 – Shear Wave Velocity Profile at Shot Location -1 m



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION, 2025
■	TEST PIT LOCATION, 2025
⊕	BOREHOLE LOCATION, 2020
⊞	TEST PIT LOCATION, 2020
•-•-•	GEOPHONE LOCATION
(20)	GEOPHONE NUMBER
◆-10.5	SHOT LOCATIONS
A A	CROSS SECTION LOCATION
66.39	GROUND SURFACE ELEVATION (m)
[65.65]	BEDROCK SURFACE ELEVATION (m)
{0.91}	BEDROCK DEPTH (m)

CONCEPTUAL PLAN PROVIDED HOBIN ARCHITECTURE

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DD/MM/YYYY	INITIAL
3	ADDED 2025 BOREHOLES AND TEST PIT LOCATION	10/04/2025	DR
2	UPDATED TO NEW CONCEPTUAL PLAN	26/02/2025	PB
1	UPDATED TO NEW CONCEPTUAL PLAN	07/09/2020	DP

UNIFORM URBAN DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
335 ROOSEVELT AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500
Date: 06/2020
Drawn by: YA
Report No.: PG2178-1
Checked by: PB
Dwg. No.: PG2178-1
Approved by: SD
Revision No.: 3

p:\autocad drawings\geotechnical\pg21xx\pg2178\pg2178-1-test hole location plan (rev.03).dwg

11x17