Geotechnical Investigation

Proposed Residential Development

4828 Bank Street
Ottawa, Ontario

Prepared for Bank & Dun Developments Inc.

Report PG7262-1 Revision 2 dated April 11, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Bank & Dun Developments Inc. to conduct a geotechnical investigation for a proposed residential development located at 4828 Bank Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

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

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

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on available information and drawings provided by the client, it is anticipated that the future development will consist of two six-storey buildings (Buildings 1 and 2), each with two underground levels.

Associated access lanes, at-grade parking, and landscaped and hardscaped areas are also anticipated as part of the development. The development is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was conducted between September 12, and 19, 2024, and consisted of 10 boreholes advanced to a maximum depth of 9.0 m below the existing ground surface. Furthermore, at that time, a total of 10 boreholes were completed within the east portion of the subject site a maximum depth of 6.2 m below the existing ground surface along with the current geotechnical program and covered under a different report. In addition, historical test holes were completed by this firm and others in 2005 during previous geotechnical investigations.

The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG7262-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The testing procedure consisted of augering, excavating, and coring to the required depth at the selected location and sampling the overburden and/or bedrock.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sampler. The bedrock was cored to assess the bedrock quality. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores (RC) were placed in cardboard boxes.

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

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus in boreholes.

Diamond drilling was completed at the selected borehole location to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

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

Groundwater

For the current investigation, monitoring wells were installed in BH 8-24, BH 10-24, BH 12-24, and BH 13-24, and the remainder of the boreholes were fitted with a flexible polyethylene standpipe to permit monitoring of the groundwater levels. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

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

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ Up to 1.5 m of slotted 32 or 51 mm diameter PVC screen at base the base of the boreholes.

☐ 32 or 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
☐ No.3 silica sand backfill within annular space around screen.
☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
☐ Clean backfill from top of bentonite plug to the ground surface.

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The location of the boreholes is presented on Drawing PG7262-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples and rock cores were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture content testing was completed on all recovered soil samples from the current investigation. The results of the testing are presented in Section 4.2 and are provided in Appendix 1.

3.4 Analytical Testing

Two (2) soil samples were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were 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.

3.5 Hydraulic Conductivity Testing

Hydraulic conductivity testing was attempted by means of falling and rising head slug tests at four (4) monitoring well locations to determine the hydraulic properties of the bedrock at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the bedrock aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.03 m. While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. The testing results are further discussed in Subsection 4.4 of this report.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped, vacant, and grass and/or gravel-covered with small trees throughout various locations. The site is bordered by vacant and undeveloped land to the north, by Bank Street to the east, by Dun Skipper Drive followed by vacant and undeveloped land to the south, and by Cedar Creek Drive followed by residential properties to the west.

Reference should be made to Figure 1 – Key Plan in Appendix 2 of the current report.

The ground surface across the subject site gradually slopes down toward the north with approximate geodetic elevations ranging from 97.9 to 101.5 m. The subject site is relatively at grade with surrounding roadways and properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil and/or fill underlain by a silty sand deposit and/or glacial till layer further underlain by bedrock.

Fill extending to depths ranging from 0.7 to 1.45 m below the existing ground surface. The fill was generally observed to consist of brown silty sand with variable amounts of gravel, clay, and crushed stone. Further, a deposit of silty sand extending to depths ranging from 0.7 to 1.45 m below the existing ground surface was observed at BH 11-24, BH 12-24, BH 15-24, and BH 19-24. The silty sand layer consists of loose brown silty sand, trace to some gravel and clay.

The fill and silty sand layers were observed to be underlain by a layer of glacial till deposit extended to approximate depths ranging between 1.5 to 6.7 m below the ground surface. The glacial till deposit generally consists of dense to very dense, grey silty sand or sandy silt with variable amounts of gravel, cobbles, and boulders.

Practical refusal to augering was encountered at depths ranging from 1.5 to 2.0 m below the existing ground surface at several borehole locations across the subject site.

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 test hole location.

Bedrock

Bedrock consisting of interbedded grey sandstone and dolomite was encountered at boreholes BH 12-24, BH 19-24, and BH 20-24 at depths of 3.6, 5.8, and 6.7 m below the existing ground surface, respectively. The bedrock was cored at the locations of boreholes BH 12-24, BH 19-24, and BH 20-24 to a depth of 6.1, 7.4, and 9.0 m, respectively. RDQ values indicate that the bedrock consists of fair to excellent quality.

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 test hole location.

Based on available geological mapping, the bedrock consists of interbedded sandstone and dolomite of the March Formation and is expected to be encountered at depths ranging from 3 to 5 m.

4.3 Groundwater

Groundwater levels were recorded at each test hole location and presented in Table 1 below. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 8-24*	100.79	5.38	95.41	September 26, 2024
BH 9A-24	101.43	Dry	N/A	
BH 10-24*	100.62	4.07	96.55	
BH 11-24	100.20	Dry	N/A	
BH 12-24*	98.82	4.17	94.65	
BH 13-24*	99.38	5.28	94.1	
BH 14-24	100.91	1.01	99.9	

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 15-24	97.96	Dry	N/A	
BH 19-24	100.69	4.85	95.84	
BH 20-24	100.93	7.27	93.66	

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

* - A monitoring well has been installed in these boreholes.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately elevations from **95.30 to 96.30 m**. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity tests were attempted at four (4) monitoring well locations throughout the subject site on September 26, 2024. However, given the minimal water column in BH8-24 and BH13-24, slug tests were only conducted at BH10-24 and BH12-24. The testing results are summarized in Table 2 below and included in Appendix 1.

Table 2 – Summary of Hydraulic Conductivity Testing Results.

Test Hole ID	Ground Surface Elevation (m)	Testing Depth Interval (m bgs)	Testing Elevation Interval (m)	K (m/sec)	Test Type	Material
BH 10-24	100.62	4.57-6.07	96.05-94.55	9.54×10^{-7}	Falling Head	Bedrock
				1.23×10^{-6}	Rising Head	
BH12-24	98.82	4.60-6.10	94.22-92.72	1.03×10^{-5}	Rising Head	Bedrock
				2.59×10^{-5}	Rising Head	

Summary of Results

Hydraulic conductivity testing conducted at the monitoring wells screened within the bedrock yielded hydraulic conductivity values ranging from 9.54×10^{-7} to 2.59×10^{-5} m/sec. These values generally fall within the range of published values for bedrock and are consistent with values Paterson has observed at sites with similar subsurface material. It should be noted that the hydraulic conductivity of the bedrock may vary based on the bedrock quality and hydrostatic pressure across the subject site.

5.0 Discussion

5.1 Geotechnical Assessment

Based on the available information, it is understood that the proposed development will consist of two six-storey residential buildings, each with two underground levels. From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed residential buildings be founded on conventional spread footings bearing on an undisturbed, dense to very dense glacial till deposit or clean surface sounded bedrock.

It is anticipated that the removal of bedrock and/or large boulders will be required for building construction and servicing installation. Therefore, the contractor should be prepared for bedrock removal and the presence of large boulders within the subject site.

Due to the absence of the sensitive silty clay deposit within the subject site, the proposed development will not be subjected to permissible grade raise restrictions.

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 or construction debris, should be stripped from under any paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

It is expected that the majority of overburden materials will be excavated to the glacial till deposit surface and/or bedrock surface for the entire buildings footprint to accommodate the one level of underground parking.

The existing fill material, where free of significant amounts of organic material, should be proof rolled by a vibratory roller making several passes under dry and above-freezing conditions, and reviewed and approved by Paterson Group at the time of construction. Provided that minimal flexing is observed the fill layer can be left in place as subgrade for pavement structure.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and 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 maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD.

The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Terraxx.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II, or select subgrade material. This material should be tested and approved by Paterson 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 paved areas should be compacted to at least 100% of its SPMDD.

Bedrock/Boulder Removal

Bedrock and/or boulder removal may be required at the subject site and can be accomplished by hoe ramming where the bedrock and/or boulders are weathered, and/or where only small quantities need to be removed. Sound bedrock and/or boulders may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Excavating boulders and bedrock will often lead to over excavation due to the natural aspect of boulders and lamination in the rock. The contractor should be ready to backfill and compact over excavated areas with engineered fill or lean concrete. Paterson should review field conditions as they arise on site.

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 the 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 or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. 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 of the overburden. The 1 m horizontal ledge setback can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face is expected to be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried out for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction.

Overbreak in Bedrock

Sedimentary bedrock formations, such as sandstone, limestone, dolomite, and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

It is highly recommended that the contractor/owner engage Paterson early into the excavation activities to identify anomalies within the bedrock. Vibration monitoring is also very essential at this stage which can be completed by Paterson as well. Also, it is important to note that completing an as-built of bedrock removal, including overbreaks, is required to be completed by the contractor and reviewed by Paterson to avoid confusion on the reasons for overbreaking the rock and to ensure that all parties are aware of the site conditions. Paterson should receive all as-built surveys post removal of the bedrock.

Protection of Subgrade

It is recommended that a minimum of 75 mm thick lean concrete mud slab be placed on undisturbed, in-situ compact to very dense glacial till where the excavation extends below the groundwater level. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade as a result of construction and moving equipment.

5.3 Foundation Design

Bearing Resistance Value (Conventional Shallow Foundation)

Footings, up to 6 m wide, founded on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on a clean, surface sounded sandstone and dolomite bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Footings placed on concrete in-filled, zero entry, vertical tranches extended to the bedrock surface can be designed to a similar bearing resistance values as the bedrock surface. It should be noted that the vertical trenches should extend horizontally a minimum of 150 mm beyond the footing faces in all directions. A minimum of 25 MPa concrete (28-day strength) should be used below the proposed footings.

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.

Settlement

Footings bearing on an undisturbed soil or an acceptable weathered bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to potential post- construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Bedrock to Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on the soil bearing medium to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

The width of the sub-excavation should be a minimum of 500 mm greater than the width of the footing. Steel reinforcement, extending a minimum of 3 m on both sides of the 2 m long transition, should be placed in the top portions of the footing and foundation walls.

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 an undisturbed glacial till or a weathered bedrock bearing surface when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or weathered bedrock or a material of the same or higher capacity as the in situ soil or weathered bedrock.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site classification for the proposed buildings in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012.

The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array was located within the proposed buildings footprint, as presented in Drawing PG7262-1 - 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 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 10.0, 3.0, and 2.0 m away from the first geophone and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected, and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the proposed buildings foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases. Based on our testing results, the average overburden shear wave velocity is **638 m/s**, while the bedrock shear wave velocity is **2,794 m/s**. Further, the testing results indicate the average overburden thickness to be approximately 7 m.

For conventional footings placed on the overburden, with an assumed underside footing depth of 6 m below the existing ground surface, the Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2024.

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

$$V_{s30=} \frac{30\ m}{\left(\frac{1\ m}{638\ m/s} + \frac{29\ m}{2{,}794\ m/s}\right)}$$

$$V_{s30=}\ 2{,}511\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity Vs_{30}, for the proposed buildings with footings within 3 m of the bedrock surface is **2,511 m/s**. Therefore, a **Site Class A** is applicable for the design of the proposed buildings according to the 2024 Ontario Building Code (OBC).

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Floor Slab / Slab-on-Grade Construction

It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are anticipated where a concrete floor slab will be used, it is recommended that the upper 300 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

It is understood that the proposed residential buildings consist of two underground levels. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3).

A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. A hydrostatic pressure should be added for the portion below the groundwater level.

Where the soil is to be retained, there are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. 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 dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

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/m^3)
H = height of the wall (m)

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

Actual earth 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 $0.375 \cdot ac \cdot \gamma \cdot H^2/g$ 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 0.351 g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using:

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

Pavement Structure Over Overburden

The following pavement structures may be considered for rigid pavement, car-only parking, and heavy traffic areas. The proposed pavement structures are shown in Tables 3, 4, and 5.

Table 3 - Recommended Rigid Pavement Structure - Lower Level

Thickness (mm)	Material Description
125	**Rigid Concrete Pavement -** 32 MPa concrete with air entrainment
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, OPSS Granular B Type II material placed over in situ soil, fill or rock	

Table 4 - Recommended Pavement Structure - Car-Only Parking Areas and Fire-Truck Routes

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 soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Table 5 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas

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

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 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Over Podium Deck Area and Raft Foundations

It is anticipated that the podium deck structure may be provided for landscaping or to accommodate car-only parking areas, access lanes, fire truck lanes, and loading areas. Based on the concrete slab subgrade for this area and/or over basements located over a raft slab, the pavement structure indicated in the following tables may be considered for design purposes:

Table 6 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck and Raft Slab)

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

SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab
*If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab.
**Thickness is dependent on grade of insulation as noted in paragraphs below.

Table 7 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck and Raft Slab)

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

SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab
* If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab.
**Thickness is dependent on grade of insulation as noted in paragraphs below.

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V frost taper is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m horizontally from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified by others to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60), or High Load (HI-40) extruded polystyrene. The pavement structures' base layer thickness in Table 6 and Table 7 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that Styrofoam rigid insulation is not considered suitable for this application.

6.0 Design and Construction Precautions

6.1 Groundwater Control for Construction

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the building's foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future buildings (and prior to tender) to review and provide supplemental information for the buildings' foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Foundation Drainage and Waterproofing

It is recommended that a perimeter foundation drainage system be provided for the below-grade areas and the proposed structures. The system, where required, should consist of a 150 mm diameter, geotextile-wrapped, perforated, and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The clear crushed stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the sump pump pit or storm sewer or ditch.

A composite drainage board and membrane should extend on the vertical foundation wall and extend to the bottom of the footings draining water towards the perimeter drainage system. The composite drainage system should consist of Delta Terraxx or equivalent. The foundation wall concrete should be properly prepared to receive a waterproofing membrane such as Colphene BSW or equivalent.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation when water infiltration can be better assessed and once the foundation layout and sump system location has been finalized.

Foundation Backfill

Where applicable, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In (If Applicable)

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall as depicted in Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-in Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed buildings to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double-sided pours (if applicable), etc.

6.2 Protection 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 alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

Other 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. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation and Service Trenches

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the buildings footprint are recommended to be provided 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 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 side-slopes.

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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Excavation below the bedrock should be benched a minimum of 450 mm from the overburden if opened over an extended period and can be completed near vertical.

Temporary Shoring

For preliminary design purposes, the temporary system may consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

These systems can be cantilevered, anchored or braced. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 8.

Table 8 – Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ’), kN/m^3	13

Soldier Pile and Lagging System

The earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65·K·γ·H for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of K·γ·H for a cantilever shoring system. H is the height of the excavation.

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

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the undrained unit weights are used for earth pressure calculations, should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used for the full height, with no hydrostatic groundwater pressure component.

A minimum factor of safety of 1.5 should be used.

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 the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of silty clay or bedrock. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.
It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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 match 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 material’s SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface. The contractor should be prepared to direct water away from all 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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Given the range of hydraulic conductivity values from site specific slug testing, a PTTW is recommended as there is the possibility that > 400,000 L/day of water may be encountered during the construction phase.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 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 (GU – General Use cement) would be appropriate for this site. The chloride content and pH of the samples indicate that they are not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to slightly 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, landscaping, and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to groundwater suppression systems, underfloor drainage systems, and waterproofing details for elevator shafts.

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 systems.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Observation of driving and re-striking of all pile foundations.
- ☐ 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 Bank & Dun Developments Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 11, 2025
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Yashar Ziaeimehr, M.A.Sc., EIT

Joey R. Villeneuve, P.Eng., ing., M.A.Sc.

Report Distribution:

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❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

HISTORICAL SOIL PROFILE AND TEST DATA SHEETS

ROCK CORE PHOTOGRAPHS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376245.10 **NORTHING:** 5019329.13 **ELEVATION:** 99.64

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 12, 2024

FILE NO. : PG7262

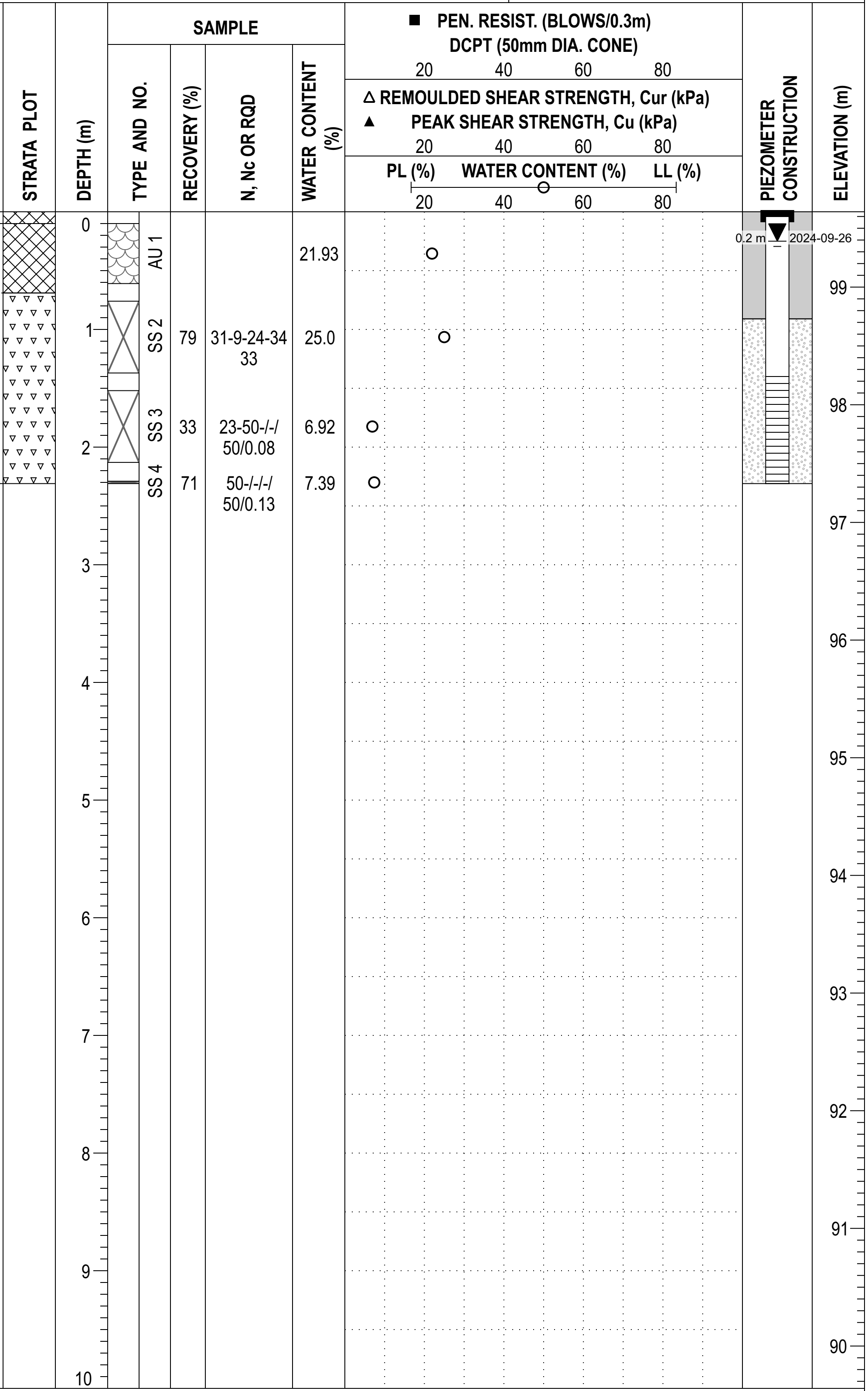
HOLE NO. : BH 1-24

SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
FILL: Crushed stone 0.10m [99.54m]	AU 1			21.93
FILL: Loose, silty sand, trace clay and gravel 0.69m [98.95m]				
GLACIAL TILL: Dense to very dense silty sand, with gravel, cobbles and boulders, trace clay - Clay content decreasing with depth 2.31m [97.33m]	SS 2	79	31-9-24-34 33	25.0
	SS 3	33	23-50-/-/ 50/0.08	6.92
	SS 4	71	50-/-/-/ 50/0.13	7.39

End of Borehole

Practical refusal to augering at 2.31 m depth

(GWL at 0.25 m - September 26, 2024)



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376203.98 **NORTHING:** 5019340.94 **ELEVATION:** 99.56

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 12, 2024

FILE NO. : PG7262

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Gravel and crushed stone 0.10m [99.46m]

FILL: Loose, brown silty sand, some gravel 0.69m [98.87m]

GLACIAL TILL: Compact to very dense, brown silty sand, with gravel, cobbles and boulders, trace clay

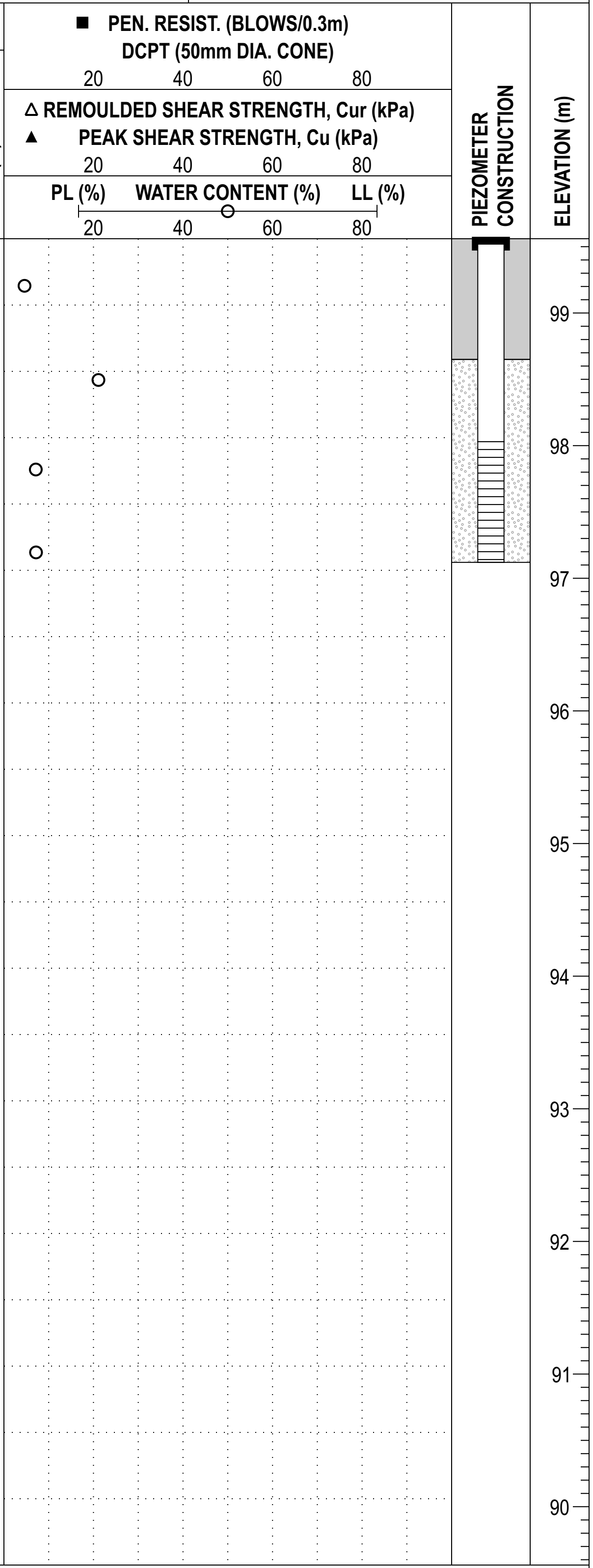
- Clay content decreasing with depth 2.44m [97.12m]

End of Borehole

Practical refusal to augering at 2.44 m depth

(Borehole dry - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			4.6
SS 2	79	3-8-18-41 26	21.11
SS 3	94	12-40-50-/ 90/0.13	7.12
SS 4	91	26-50-/-/ 50/0.13	7.17



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376265.39 **NORTHING:** 5019379.03 **ELEVATION:** 97.64

PROJECT: Proposed Residential and Commercial Developments

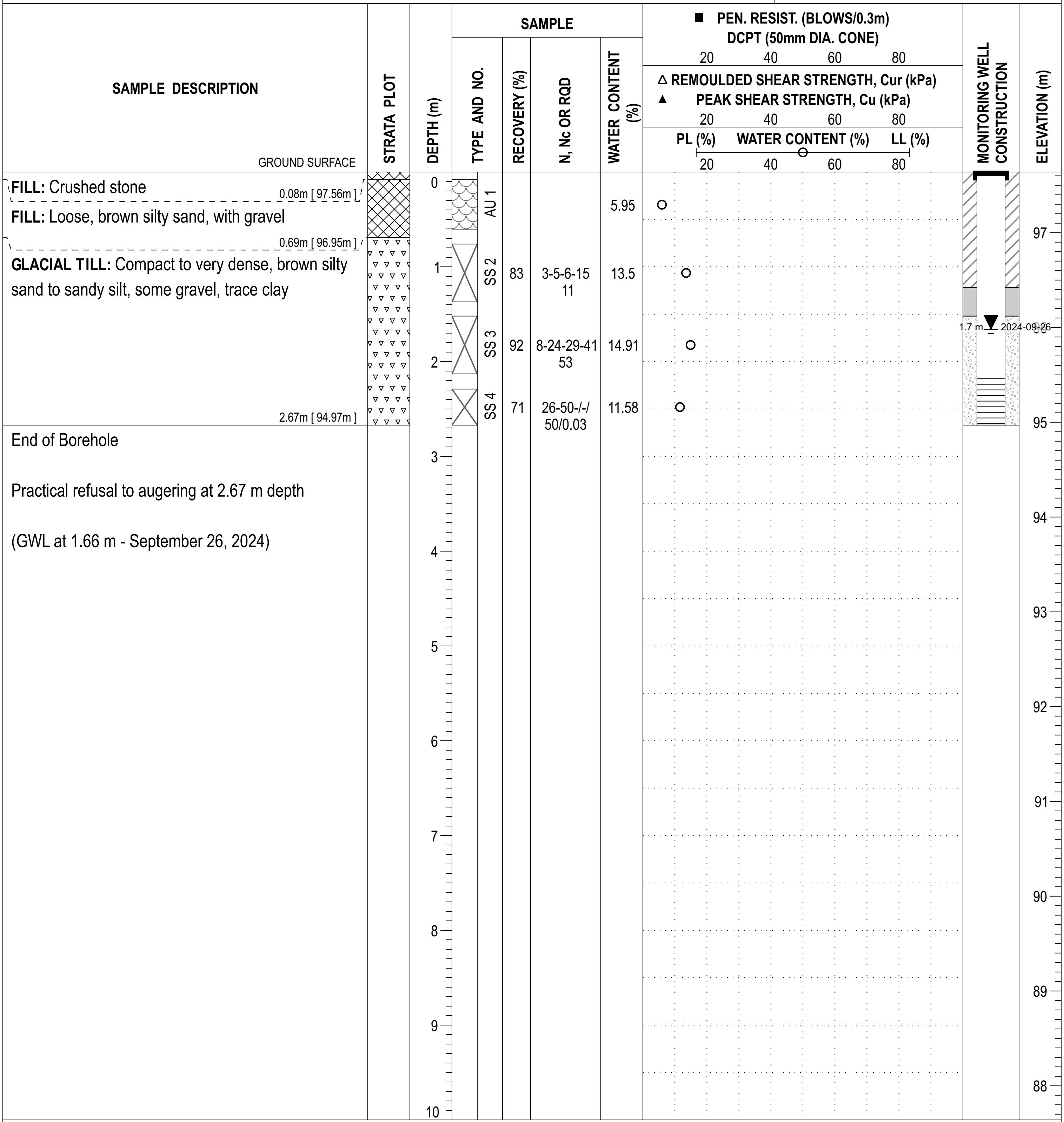
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 12, 2024

FILE NO. : PG7262

HOLE NO. : BH 3-24



SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Crushed stone — 0.08m [97.56m]

FILL: Loose, brown silty sand, with gravel — 0.69m [96.95m]

GLACIAL TILL: Compact to very dense, brown silty sand to sandy silt, some gravel, trace clay — 2.67m [94.97m]

End of Borehole

Practical refusal to augering at 2.67 m depth

(GWL at 1.66 m - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			5.95
SS 2	83	3-5-6-15 11	13.5
SS 3	92	8-24-29-41 53	14.91
SS 4	71	26-50-/-/ 50/0.03	11.58

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376280.40 **NORTHING:** 5019343.40 **ELEVATION:** 98.80

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

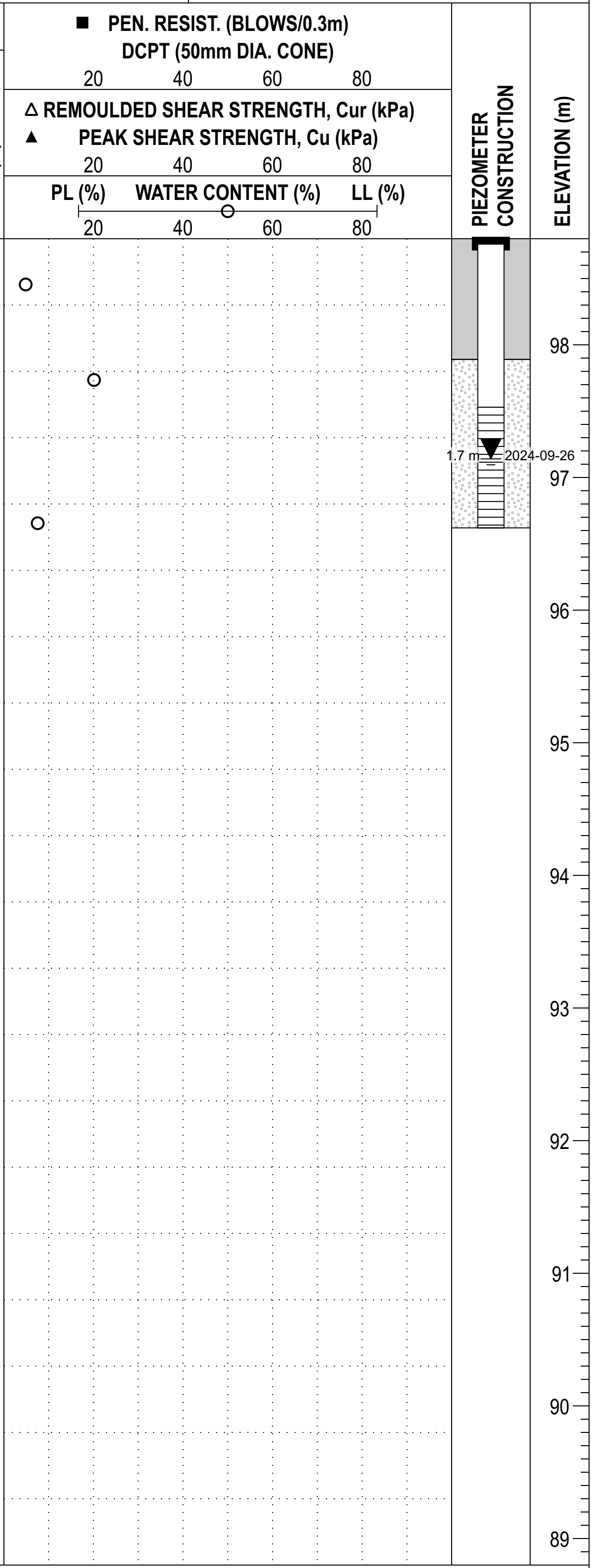
REMARKS:

DATE: September 12, 2024

FILE NO. : PG7262

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE		0					
FILL: Crushed stone 0.08m [98.72m]							
FILL: Loose brown silty sand, with gravel 0.69m [98.11m]			AU 1			4.86	98
GLACIAL TILL: Compact, brown silty sand to sandy silt, with gravel, cobbles and boulders, trace clay		1	SS 2	63	5-5-7-13 12	20.13	
			SS 3	0	50-/-/-/ 50/0.05		97
2.18m [96.62m]		2	SS 4	100	50-/-/-/ 50/0.08	7.54	
End of Borehole							
Practical refusal to augering at 2.18 m depth		3					96
(GWL at 1.66 m - September 26, 2024)		4					95
		5					94
		6					93
		7					92
		8					91
		9					90
		10					89



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376222.68 **NORTHING:** 5019391.16 **ELEVATION:** 97.60

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 12, 2024

FILE NO. : PG7262

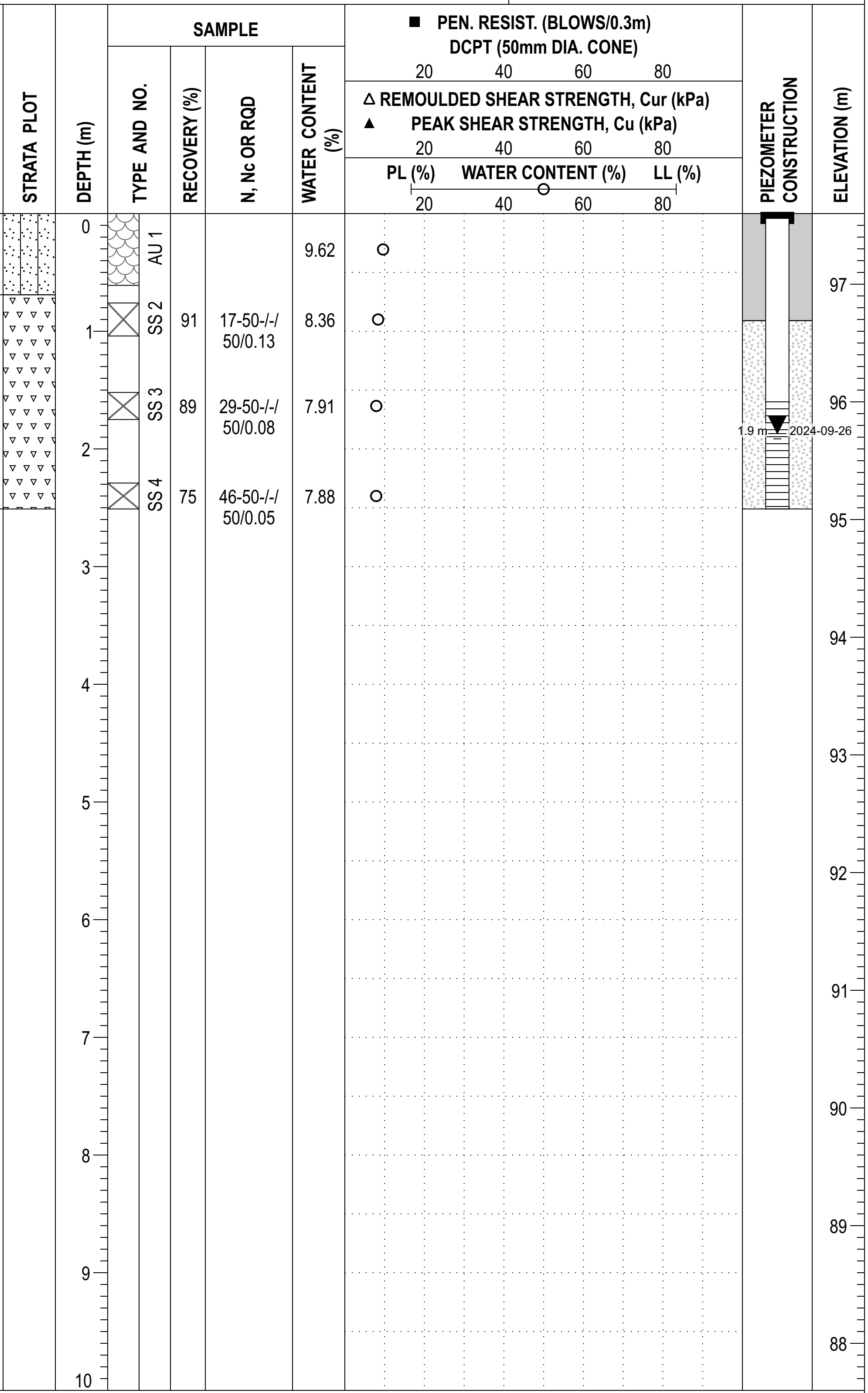
HOLE NO. : BH 5-24

SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
Loose, brown **SILTY SAND**, trace gravel and organics	AU 1			9.62
0.69m [96.91m]				
GLACIAL TILL: Very dense, brown silty sand, with cobbles and boulders, trace gravel	SS 2	91	17-50-/-/ 50/0.13	8.36
	SS 3	89	29-50-/-/ 50/0.08	7.91
	SS 4	75	46-50-/-/ 50/0.05	7.88
2.51m [95.09m]				

End of Borehole

Practical refusal to augering at 2.51 m depth

(GWL at 1.87 m - September 26, 2024)



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376201.46 **NORTHING:** 5019413.96 **ELEVATION:** 97.13

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

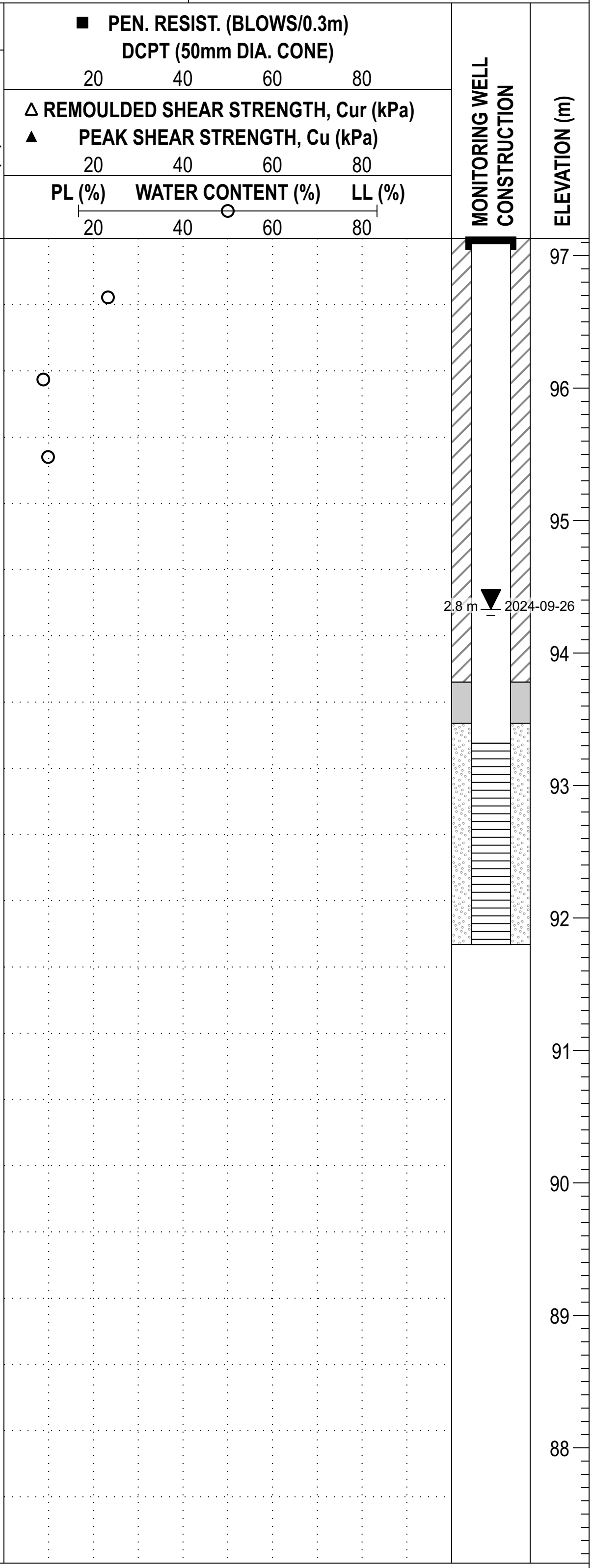
REMARKS:

DATE: September 13, 2024

FILE NO. : PG7262

HOLE NO. : BH 6-24

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
TOPSOIL and organics 0.28m [96.85m]					
Loose, brown **SILTY SAND**, some clay 0.69m [96.44m]		AU 1			23.24
GLACIAL TILL: Very dense, brown silty sand, occasional gravel, cobbles and boulders	1	SS 2	92	24-30-45-50 75	8.8
		SS 3	80	23-50-/-/ 50/0.1	9.86
- Boulders between 1.8 m to 3.0 m depth	2	RC 1	19		
- Coring was completed through out boulders 3.96m [93.17m]	3				
BEDROCK: good to excellent quality, interbedded grey sandstone and dolomite	4	RC 2	100	RQD 100	
		RC 3	100	RQD 89	
	5	RC 4	100	RQD 100	
6.15m [90.98m]	6				
End of Borehole (GWL at 2.80 m - September 26, 2024)					



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376274.95 **NORTHING:** 5019311.77 **ELEVATION:** 99.57

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 13, 2024

FILE NO. : PG7262

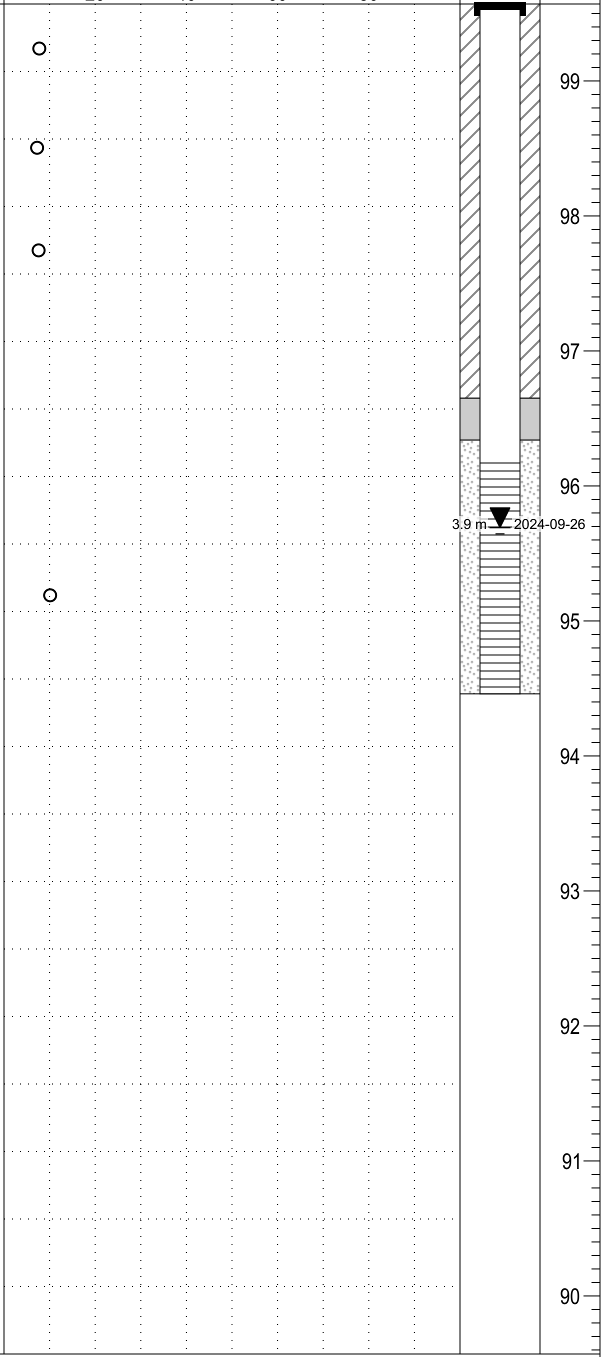
HOLE NO. : BH 7-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE			
GROUND SURFACE			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
FILL: Crushed stone		0	AU 1			7.74
0.61m [98.96m]						
FILL: Loose, brown silty sand, with gravel and crushed stone, trace clay						
0.69m [98.88m]		1	SS 2	92	7-18-20-21 38	7.26
GLACIAL TILL: Dense, brown silty sand, with gravel, cobbles and boulders						
- Broken rock fragments at 1.93 m depth		2	SS 3	42	16-50-/-/ 50/0.13	7.59
			SS 4	0	50-/-/-/ 50/0.1	
- Boulders and dense silty sand below 2.69 m depth			RC 1	61		
- Coring was completed through out boulders		3	SS 5	50	50-/-/-/ 50/0.05	
		4	RC 2	40		
			SS 6	57	24-50-/-/ 50/0.03	10.12
			RC 3	42		
		5	RC 4	31		
5.89m [93.68m]						
End of Borehole		6				
(GWL at 3.88 m - September 26, 2024)		7				
		8				
		9				
		10				

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

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

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



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376195.45 **NORTHING:** 5019280.99 **ELEVATION:** 100.79

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 16, 2024

FILE NO. : PG7262

HOLE NO. : BH 8-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Crushed stone and gravel 0.10m [100.69m]

FILL: Loose to very loose silty sand, trace gravel 0.69m [100.10m]

GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders

- Trace clay between 2.3 m to 2.6 m depth

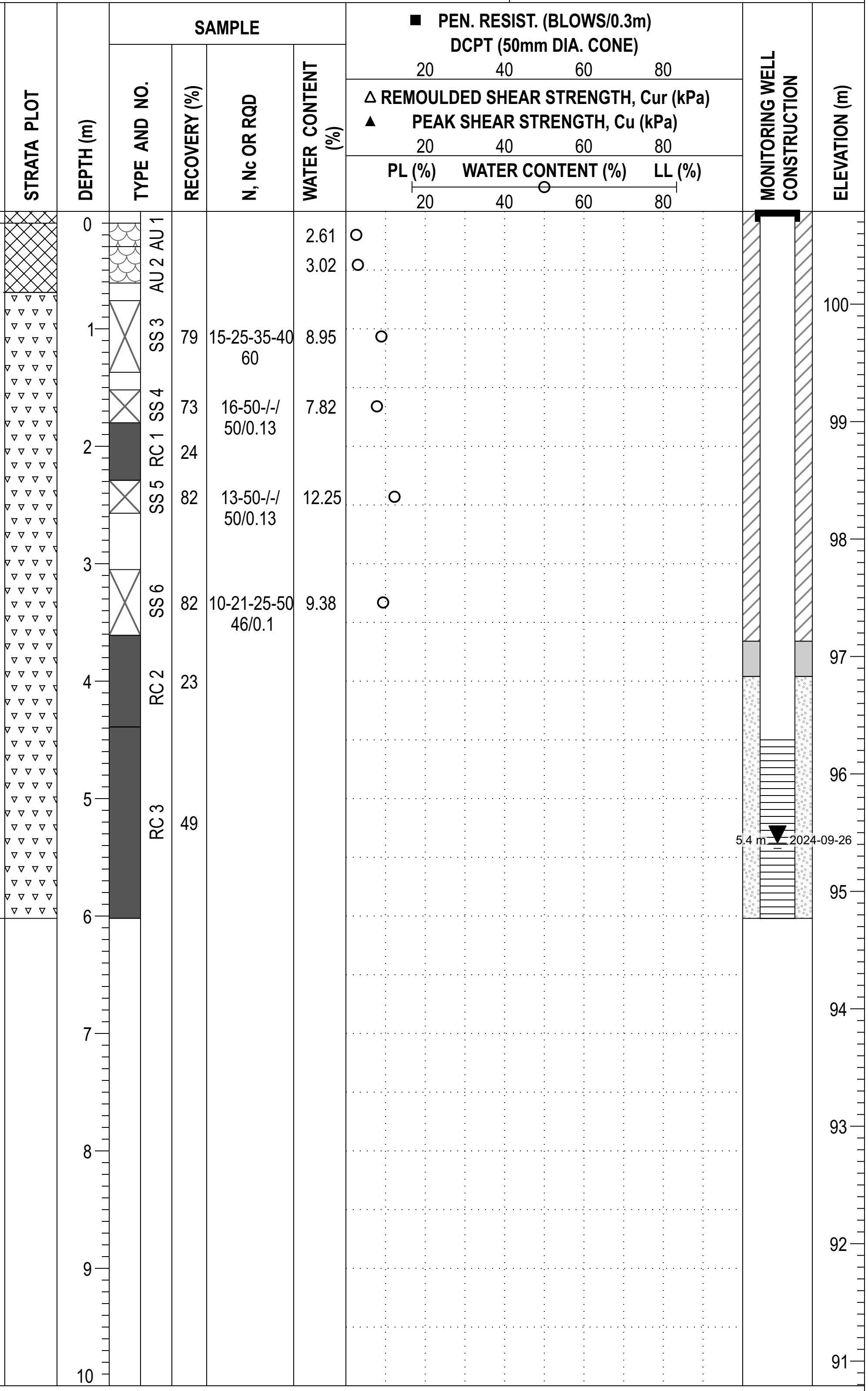
- Coring was completed through out boulders

6.02m [94.77m]

End of Borehole

(GWL at 5.38 m - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			2.61
AU 2			3.02
SS 3	79	15-25-35-40 60	8.95
SS 4	73	16-50-/-/ 50/0.13	7.82
RC 1	24		
SS 5	82	13-50-/-/ 50/0.13	12.25
SS 6	82	10-21-25-50 46/0.1	9.38
RC 2	23		
RC 3	49		



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376167.81 **NORTHING:** 5019263.12 **ELEVATION:** 101.43

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

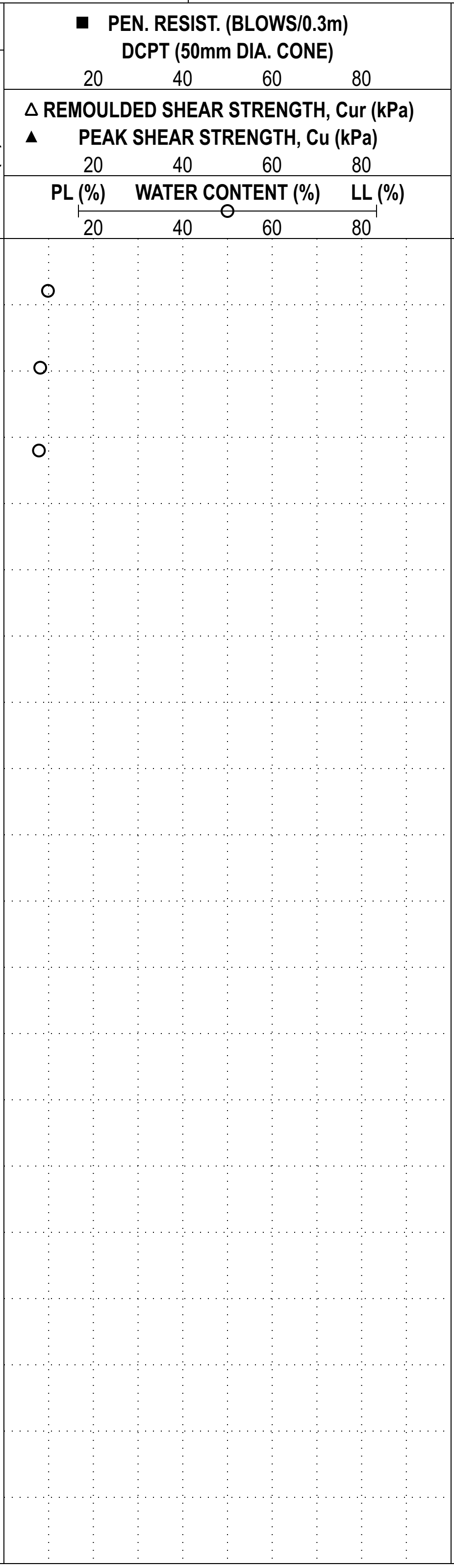
DATE: September 16, 2024

FILE NO. : PG7262

HOLE NO. : BH 9-24

SAMPLE DESCRIPTION	DEPTH (m)	ELEVATION (m)
GROUND SURFACE	0	101.43
FILL: Crushed stone and gravel	0.10m	[101.33m]
FILL: Loose, brown silty sand, trace gravel	0.69m	[100.74m]
GLACIAL TILL: Very dense, brown silty sand, trace gravel	1.68m	[99.75m]
End of Borehole Practical Refusal to auger at 1.68 m depth		

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			9.85
SS 2	100	12-31-50-/ 81/0.13	8.12
SS 3	71	29-50-/-/ 50/0.03	7.83



PIEZOMETER CONSTRUCTION

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376167.81 **NORTHING:** 5019263.12 **ELEVATION:** 101.43

PROJECT: Proposed Residential and Commercial Developments

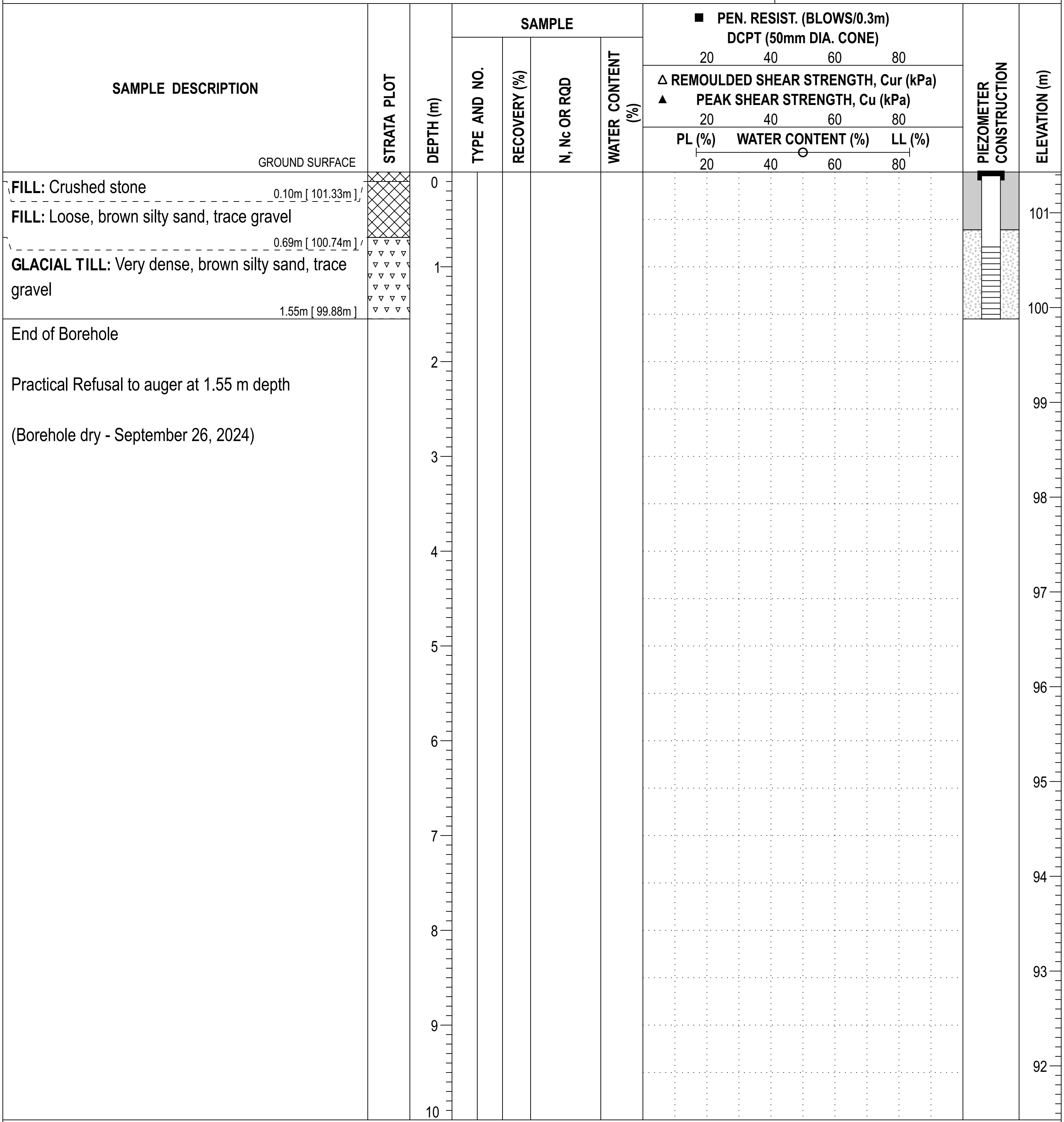
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 16, 2024

FILE NO. : PG7262

HOLE NO. : BH 9A-24



FILL: Crushed stone 0.10m [101.33m]

FILL: Loose, brown silty sand, trace gravel 0.69m [100.74m]

GLACIAL TILL: Very dense, brown silty sand, trace gravel 1.55m [99.88m]

End of Borehole

Practical Refusal to auger at 1.55 m depth

(Borehole dry - September 26, 2024)

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376135.93 **NORTHING:** 5019308.57 **ELEVATION:** 100.62

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 16, 2024

FILE NO. : PG7262

HOLE NO. : BH10-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Crushed stone and gravel 0.13m [100.49m]

FILL: Loose, brown silty sand, trace gravel 0.69m [99.93m]

GLACIAL TILL: Very dense, brown silty sand, with cobbles and boulders, trace gravel

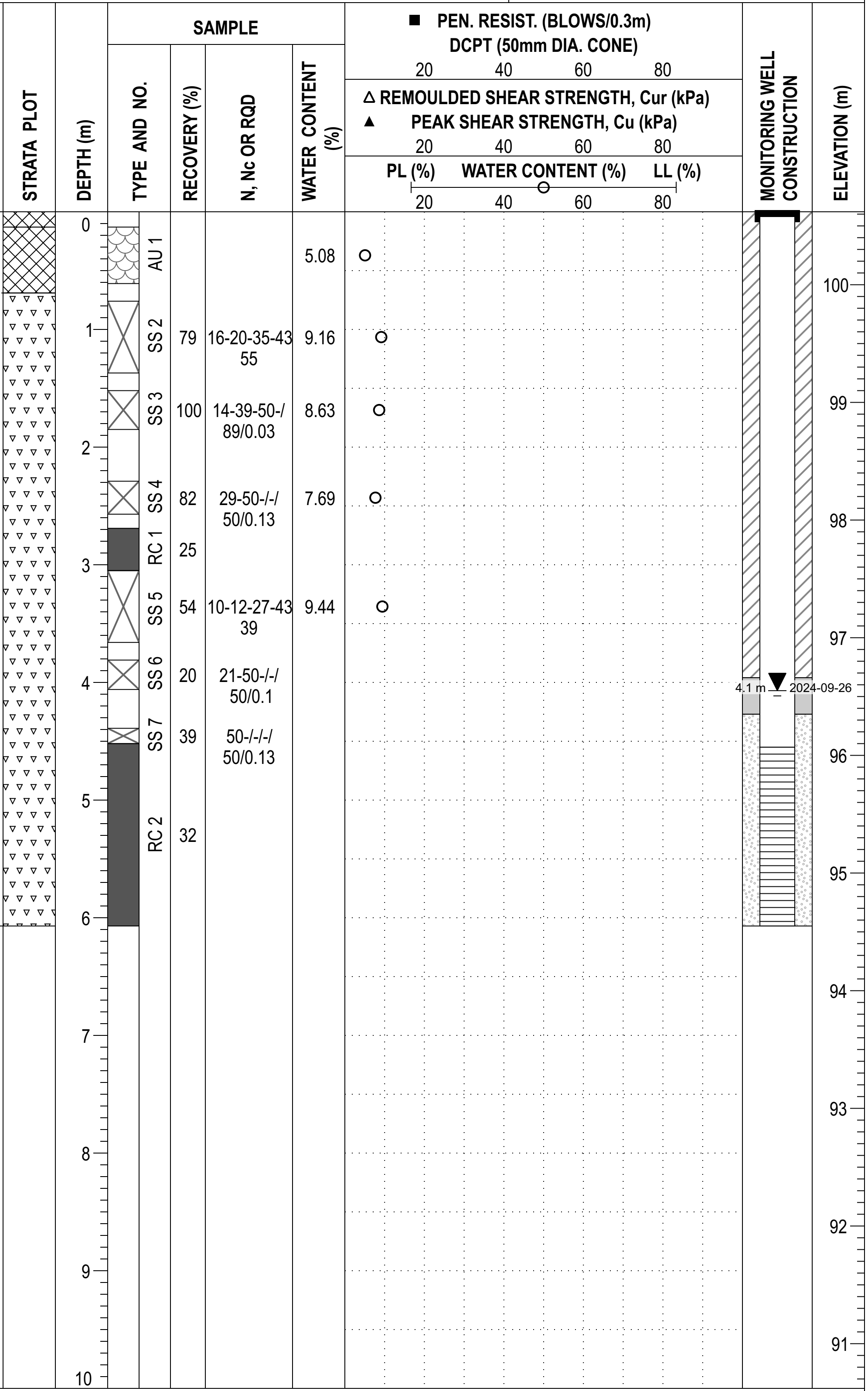
- Coring was completed through out boulders

- Rock fragments below 4.4 m depth

6.07m [94.55m]

End of Borehole

(GWL at 4.07 m - September 26, 2024)



TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			5.08
SS 2	79	16-20-35-43 55	9.16
SS 3	100	14-39-50-/ 89/0.03	8.63
SS 4	82	29-50-/-/ 50/0.13	7.69
RC 1	25		
SS 5	54	10-12-27-43 39	9.44
SS 6	20	21-50-/-/ 50/0.1	
SS 7	39	50-/-/-/ 50/0.13	
RC 2	32		

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376179.45 **NORTHING:** 5019314.49 **ELEVATION:** 100.20

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 17, 2024

FILE NO. : PG7262

HOLE NO. : BH11-24

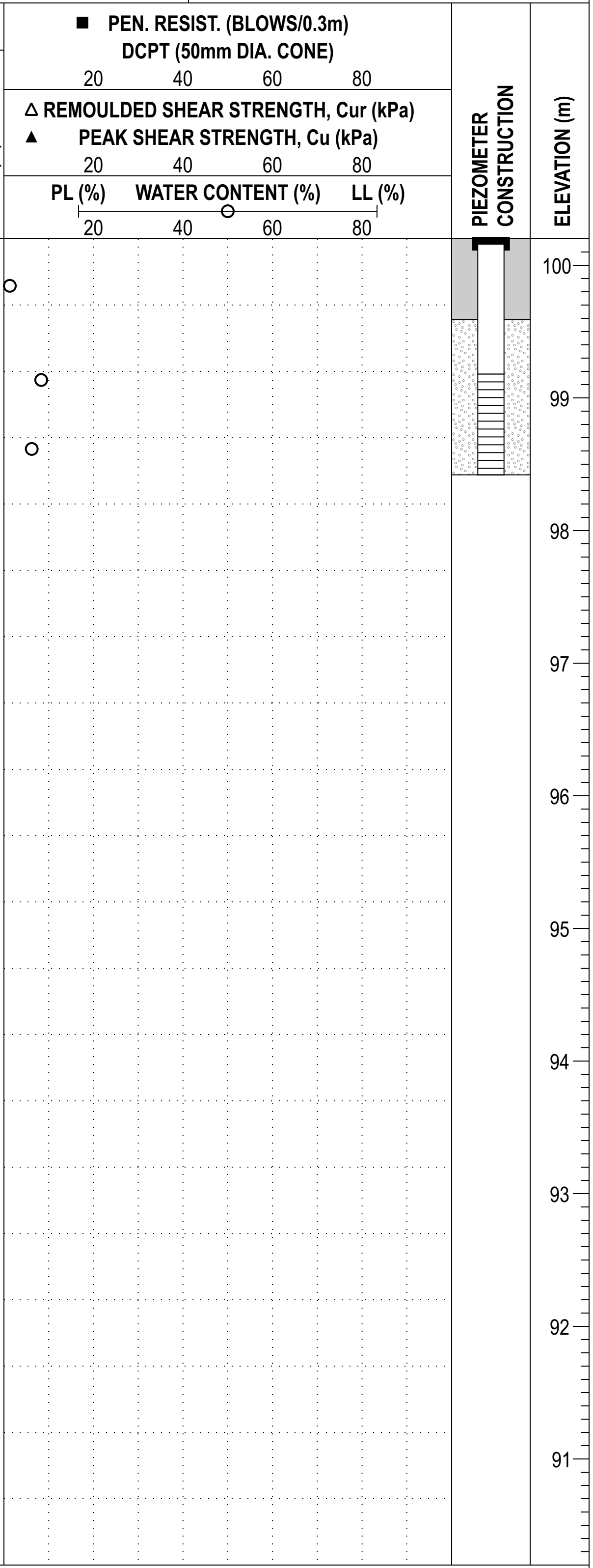
SAMPLE DESCRIPTION	DEPTH (m)
GROUND SURFACE	0
FILL: Crushed stone and gravel	0.10m [100.10m]
SILTY SAND, trace gravel	0.69m [99.51m]
GLACIAL TILL: Dense, brown silty sand, with gravel, cobbles and boulders	1.78m [98.42m]

End of Borehole

Practical Refusal to auger at 1.78 m depth

(Borehole dry - September 26, 2024)

SAMPLE			
TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			1.32
SS 2	83	6-21-21-50 42	8.33
SS 3	40	50-/-/-/ 50/0.13	6.21



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376163.47 **NORTHING:** 5019346.65 **ELEVATION:** 98.82

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

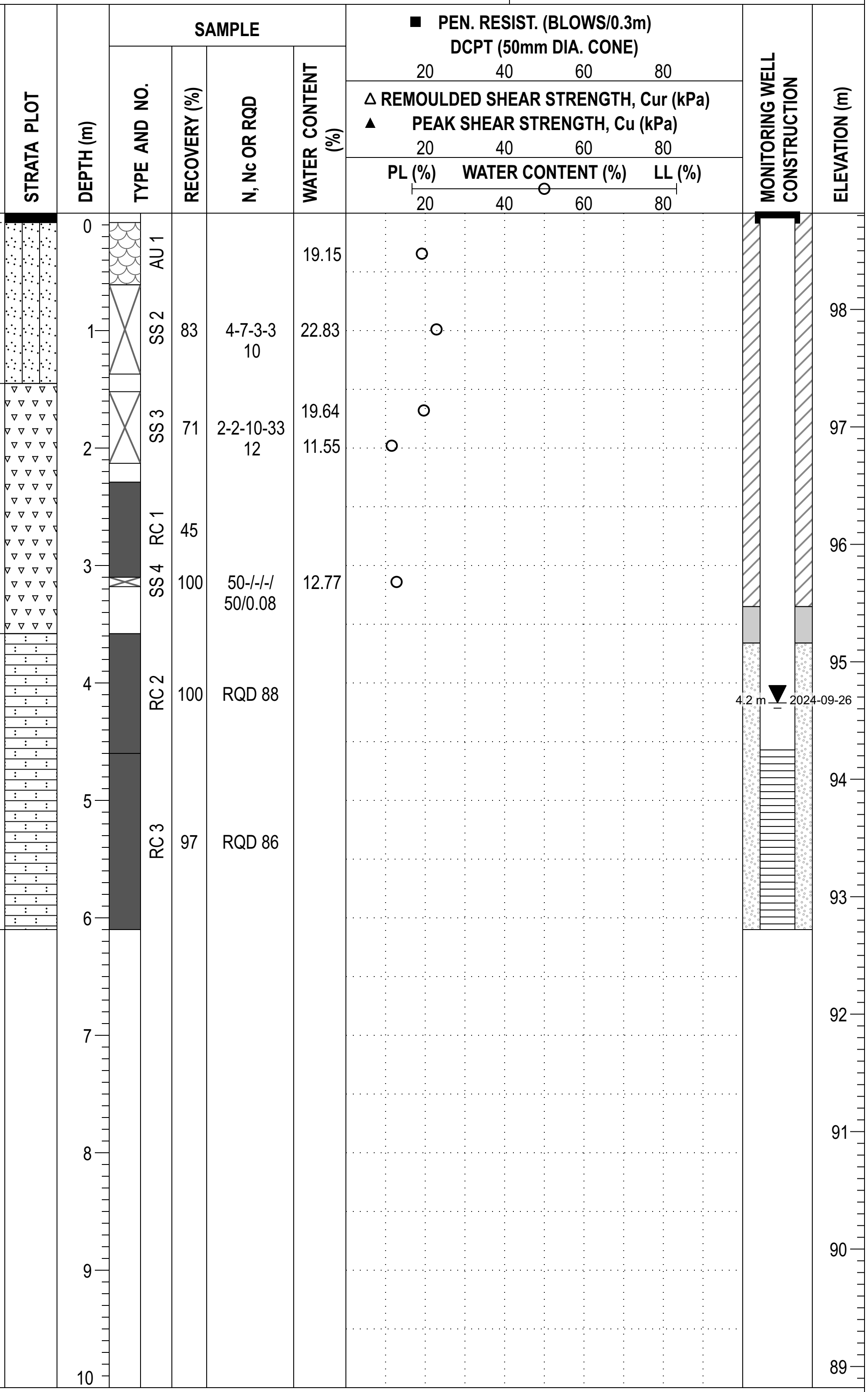
REMARKS:

DATE: September 17, 2024

FILE NO. : **PG7262**

HOLE NO. : **BH12-24**

SAMPLE DESCRIPTION	
GROUND SURFACE	
TOPSOIL and organics	0.08m [98.74m]
Loose, brown **SILTY SAND**, trace clay	1.45m [97.37m]
GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, cobbles and boulders - Coring was completed through out boulders	3.58m [95.24m]
BEDROCK: good quality, interbedded grey sandstone and dolostone	6.10m [92.72m]
End of Borehole (GWL at 4.17 m - September 26, 2024)	



TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			19.15
SS 2	83	4-7-3-3 10	22.83
SS 3	71	2-2-10-33 12	19.64 11.55
RC 1	45		
SS 4	100	50-/-/-/ 50/0.08	12.77
RC 2	100	RQD 88	
RC 3	97	RQD 86	

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376107.94 **NORTHING:** 5019374.85 **ELEVATION:** 99.38

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

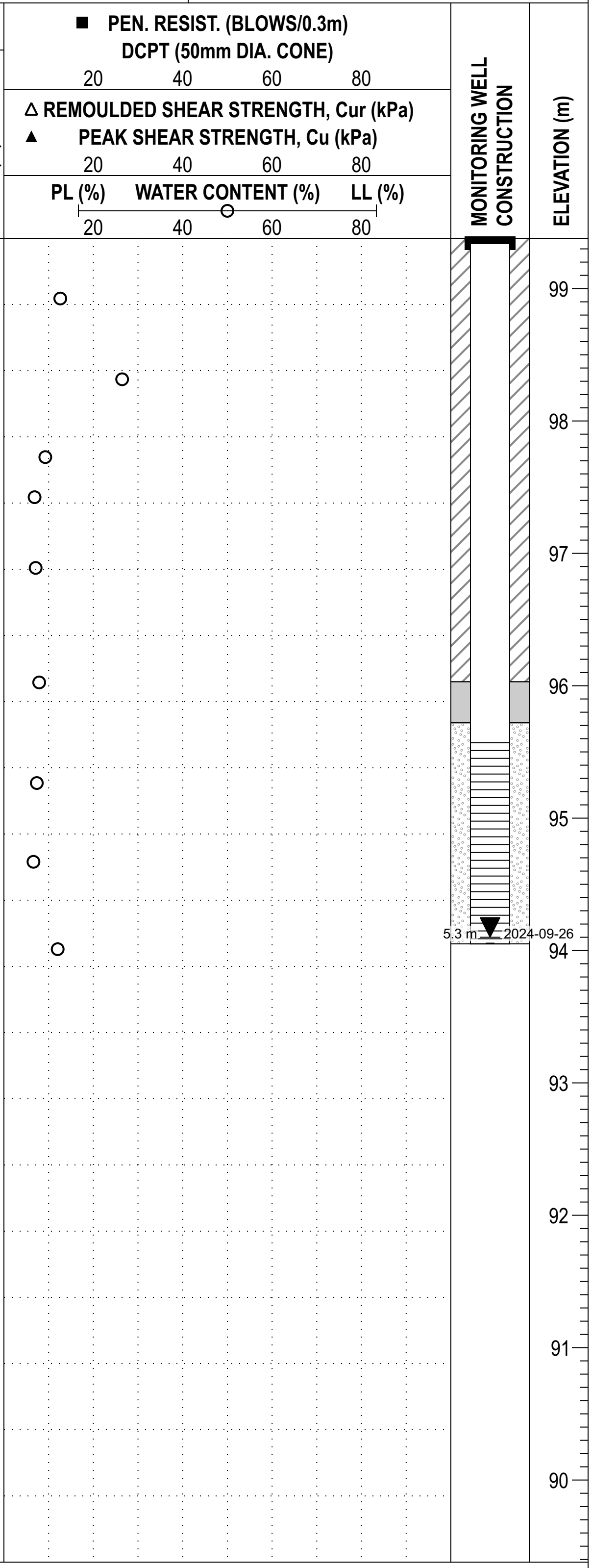
REMARKS:

DATE: September 17, 2024

FILE NO. : PG7262

HOLE NO. : BH13-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE		0				
FILL: Loose, brown silty sand, trace gravel and cobbles			AU 1			12.61
		1	SS 2	71	3-4-4-12 8	26.45
1.45m [97.93m]						
GLACIAL TILL: Very dense, brown silty sand, trace gravel and cobbles			SS 3	91	15-30-37-50 67	9.26 6.86
		2				
			SS 4	100	47-42-50-/ 92/0.1	7.1
		3	SS 5	100	30-38-39-50 77	7.89
		4	SS 6	71	18-30-50-/ 80/0.13	7.36
			SS 7	91	16-50-/-/ 50/0.13	6.61
- Brown sand seam between 5.3 m to 5.4 m depth - Grey below 5.4 m depth		5				
5.41m [93.97m]			SS 8	100	50-/-/-/ 50/0.08	12.03
End of Borehole (GWL at 5.28 m - September 26, 2024)		6				



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376121.38 **NORTHING:** 5019328.50 **ELEVATION:** 100.91

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

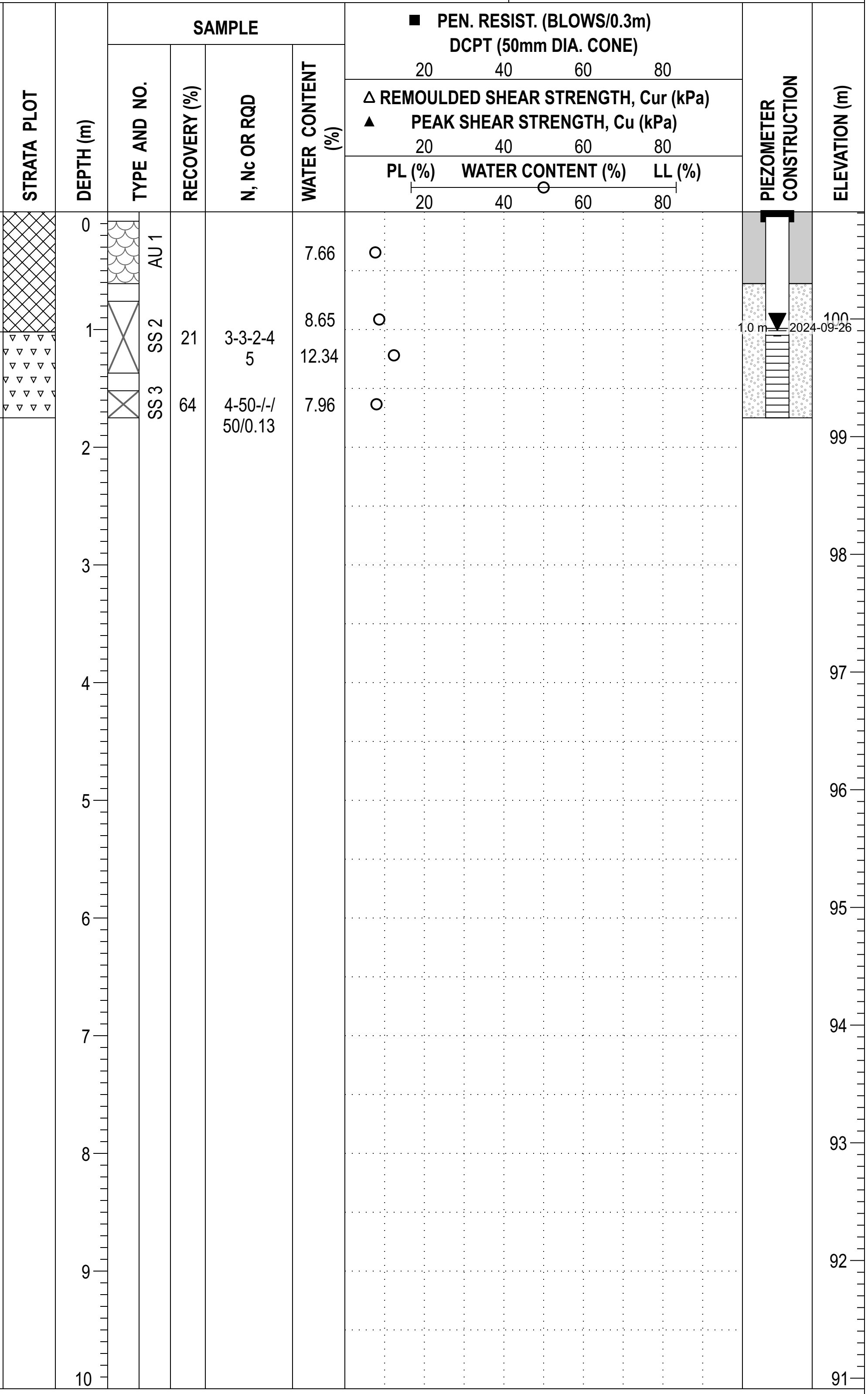
FILE NO. : PG7262

HOLE NO. : BH14-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
FILL: Loose, brown silty sand, with crushed stone, trace gravel		AU 1			7.66
1.02m [99.89m]	1				
GLACIAL TILL: Compact to very dense, brown silty sand, some gravel and cobbles, trace clay - Clay content decreases with depth		SS 2	21	3-3-2-4 5	8.65 12.34
		SS 3	64	4-50-/-/ 50/0.13	7.96
1.75m [99.16m]					
End of Borehole	2				

Practical Refusal to auger at 1.75 m depth

(GWL at 1.01 m - September 26, 2024)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376143.40 **NORTHING:** 5019388.87 **ELEVATION:** 97.96

PROJECT: Proposed Residential and Commercial Developments

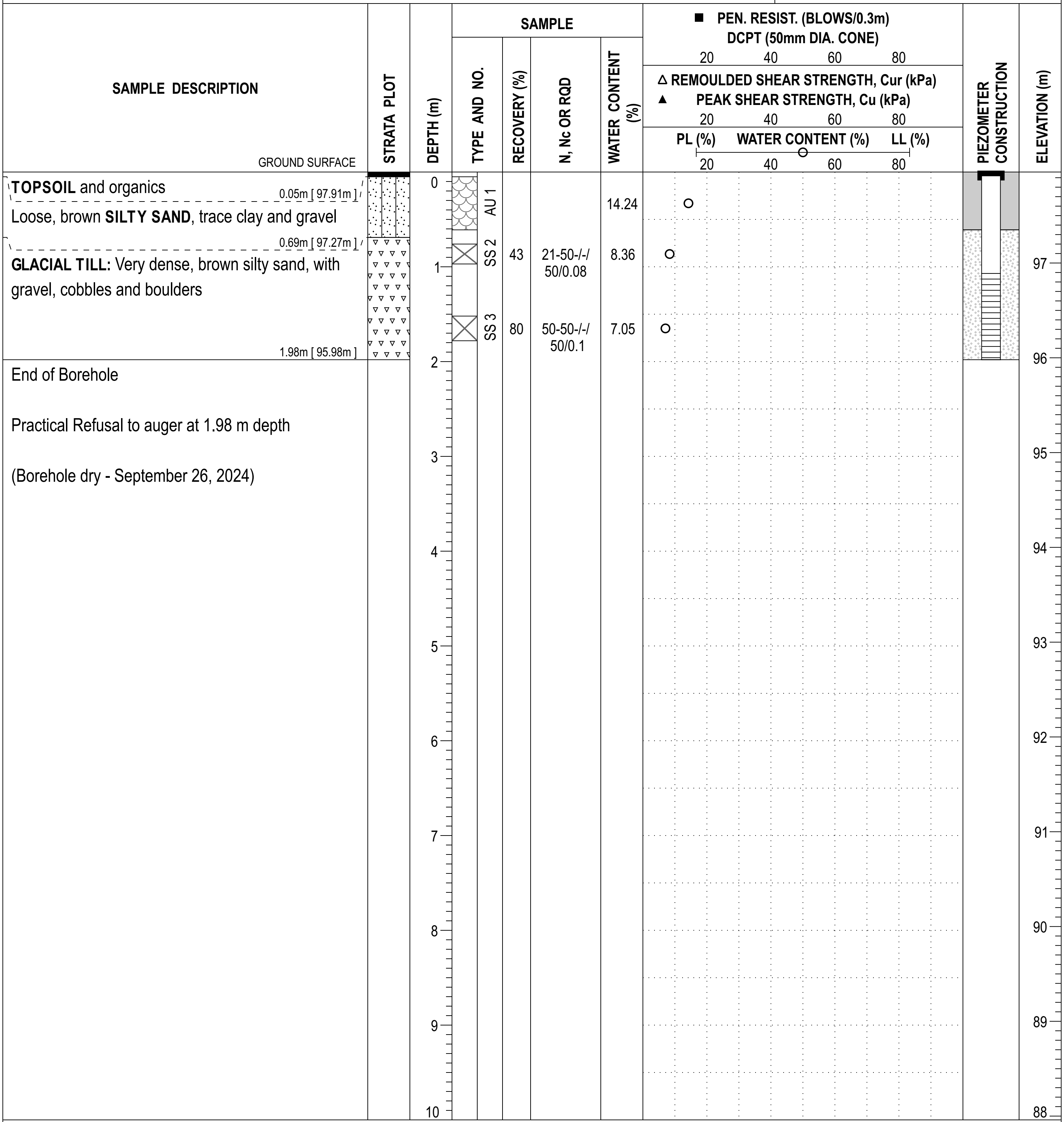
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

FILE NO. : PG7262

HOLE NO. : BH15-25



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
TOPSOIL and organics — 0.05m [97.91m]				
Loose, brown **SILTY SAND**, trace clay and gravel — 0.69m [97.27m]	AU 1			14.24
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders — 1.98m [95.98m]	SS 2	43	21-50-/-/ 50/0.08	8.36
	SS 3	80	50-50-/-/ 50/0.1	7.05

End of Borehole

Practical Refusal to auger at 1.98 m depth

(Borehole dry - September 26, 2024)

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376170.56 **NORTHING:** 5019394.51 **ELEVATION:** 97.69

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

FILE NO. : PG7262

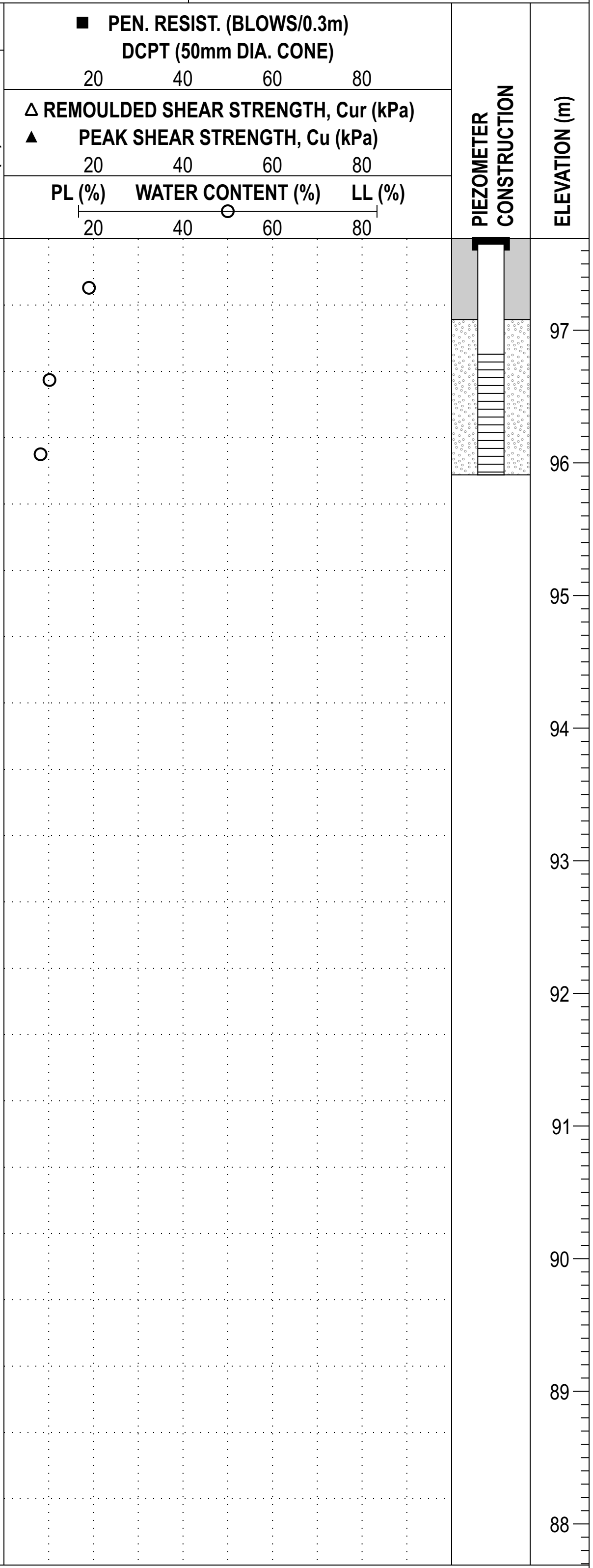
HOLE NO. : BH16-24

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
TOPSOIL and organics — 0.13m [97.56m]					
Loose, brown **SILTY SAND**, trace clay — 0.69m [97.00m]		AU 1			19.0
GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, cobbles and boulders — 1.78m [95.91m]	1	SS 2	83	6-16-21-36 37	10.17
		SS 3	100	30-50-/-/ 50/0.05	8.18

End of Borehole

Practical Refusal to auger at 1.78 m depth

(Borehole dry - September 26, 2024)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376243.01 **NORTHING:** 5019438.17 **ELEVATION:** 96.36

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

FILE NO. : **PG7262**

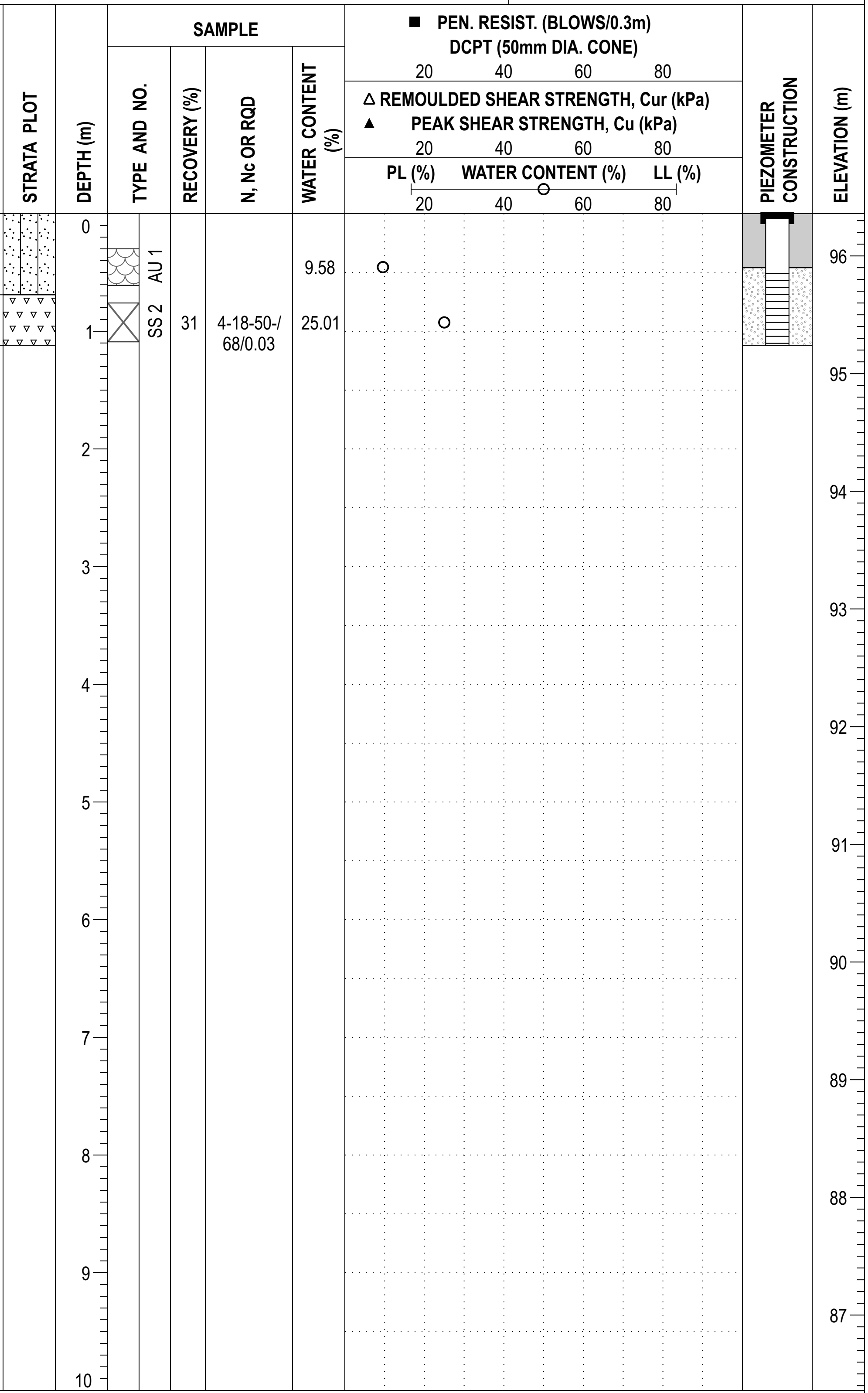
HOLE NO. : **BH17-27**

SAMPLE DESCRIPTION GROUND SURFACE	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
Loose, brown **SILTY SAND**, trace clay 0.69m [95.67m]	0	AU 1			9.58
GLACIAL TILL: Very dense, brown silty clay, with gravel, cobbles and boulders 1.12m [95.24m]	1	SS 2	31	4-18-50-/ 68/0.03	25.01

End of Borehole

Practical Refusal to auger at 1.12 m depth

(Borehole dry - September 26, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376244.36 **NORTHING:** 5019291.25 **ELEVATION:** 100.10

PROJECT: Proposed Residential and Commercial Developments

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

FILE NO. : PG7262

HOLE NO. : BH18-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
FILL: Brown silty sand, with cobbles and boulders 0.51m [99.59m]	▨	0							100
End of Borehole Practical Refusal to auger at 0.51 m depth		1							99
		2							98
		3							97
		4							96
		5							95
		6							94
		7							93
		8							92
		9							91
		10							

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376256.13 **NORTHING:** 5019297.30 **ELEVATION:** 100.00

PROJECT: Proposed Residential and Commercial Developments

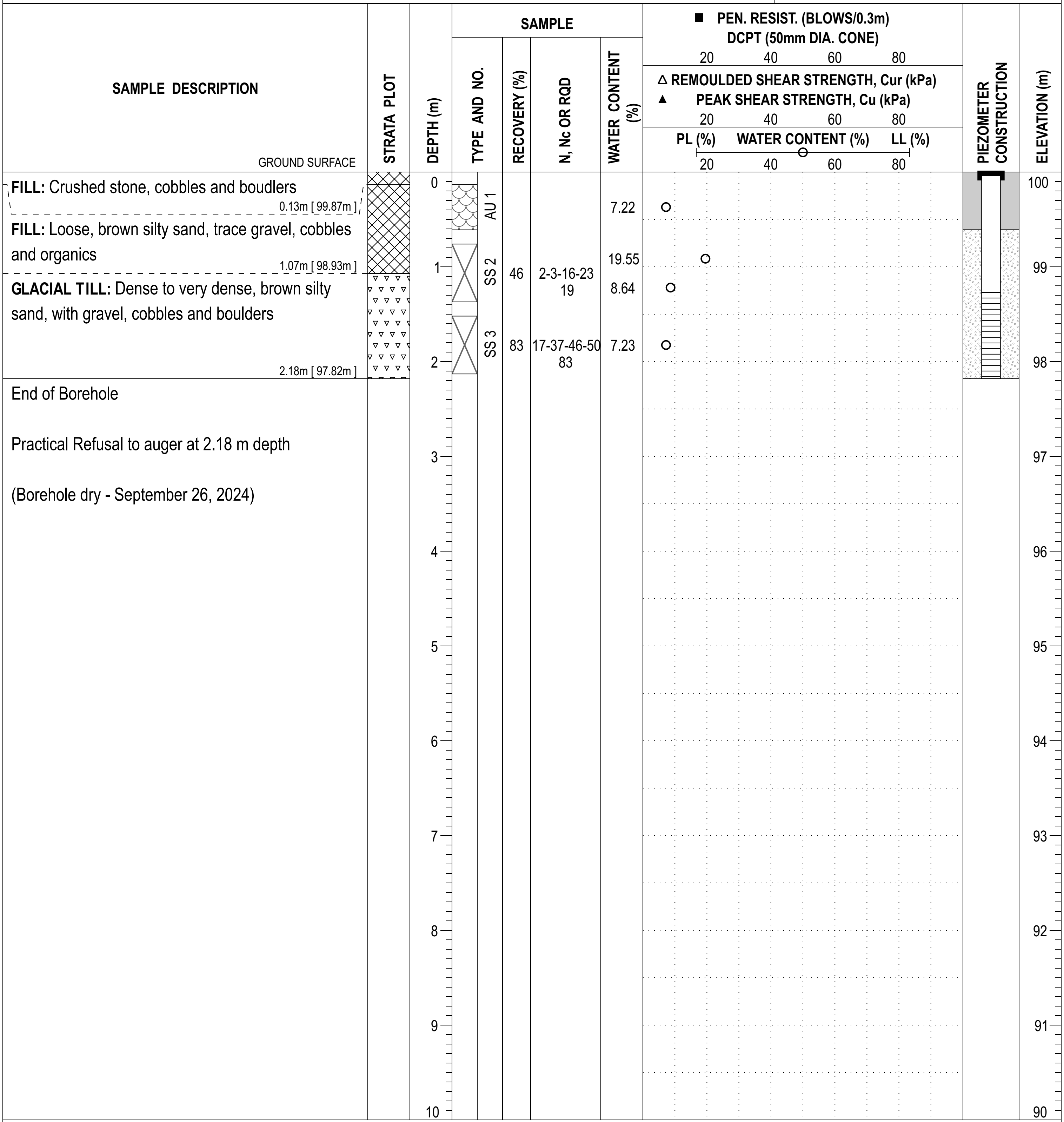
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 18, 2024

FILE NO. : PG7262

HOLE NO. : BH18A-24



FILL: Crushed stone, cobbles and boudlers

0.13m [99.87m]

FILL: Loose, brown silty sand, trace gravel, cobbles and organics

1.07m [98.93m]

GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, cobbles and boulders

2.18m [97.82m]

End of Borehole

Practical Refusal to auger at 2.18 m depth

(Borehole dry - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			7.22
SS 2	46	2-3-16-23 19	19.55 8.64
SS 3	83	17-37-46-50 83	7.23

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376167.66 **NORTHING:** 5019279.22 **ELEVATION:** 100.69

PROJECT: Proposed Residential and Commercial Developments

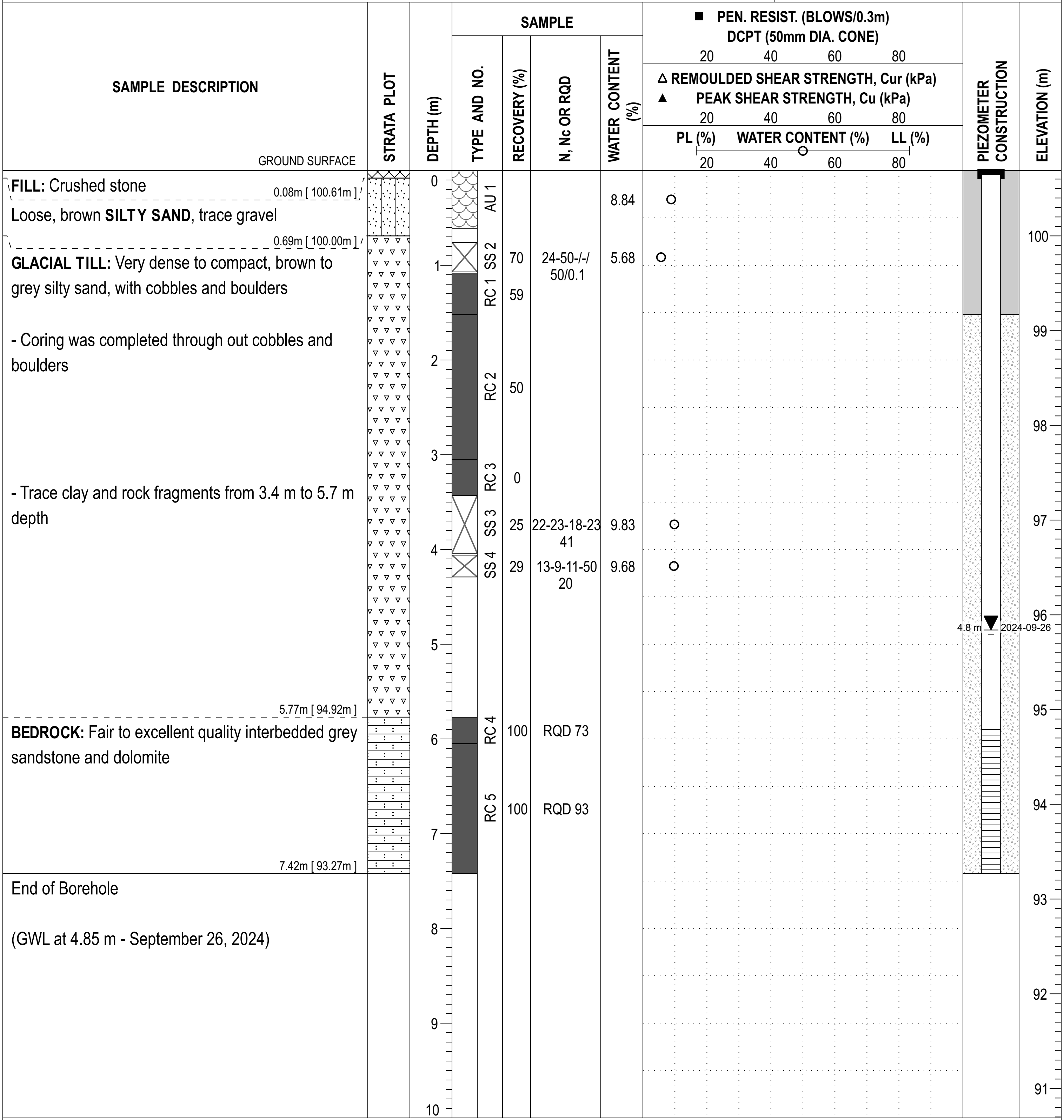
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 19, 2024

FILE NO. : PG7262

HOLE NO. : BH19-24



FILL: Crushed stone — 0.08m [100.61m]

Loose, brown **SILTY SAND**, trace gravel — 0.69m [100.00m]

GLACIAL TILL: Very dense to compact, brown to grey silty sand, with cobbles and boulders

- Coring was completed through out cobbles and boulders

- Trace clay and rock fragments from 3.4 m to 5.7 m depth

5.77m [94.92m]

BEDROCK: Fair to excellent quality interbedded grey sandstone and dolomite

7.42m [93.27m]

End of Borehole

(GWL at 4.85 m - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			8.84
SS 2	70	24-50-/-/ 50/0.1	5.68
RC 1	59		
RC 2	50		
RC 3	0		
SS 3	25	22-23-18-23 41	9.83
SS 4	29	13-9-11-50 20	9.68
RC 4	100	RQD 73	
RC 5	100	RQD 93	

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4828 Bank Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 376115.17 **NORTHING:** 5019351.94 **ELEVATION:** 100.93

PROJECT: Proposed Residential and Commercial Developments

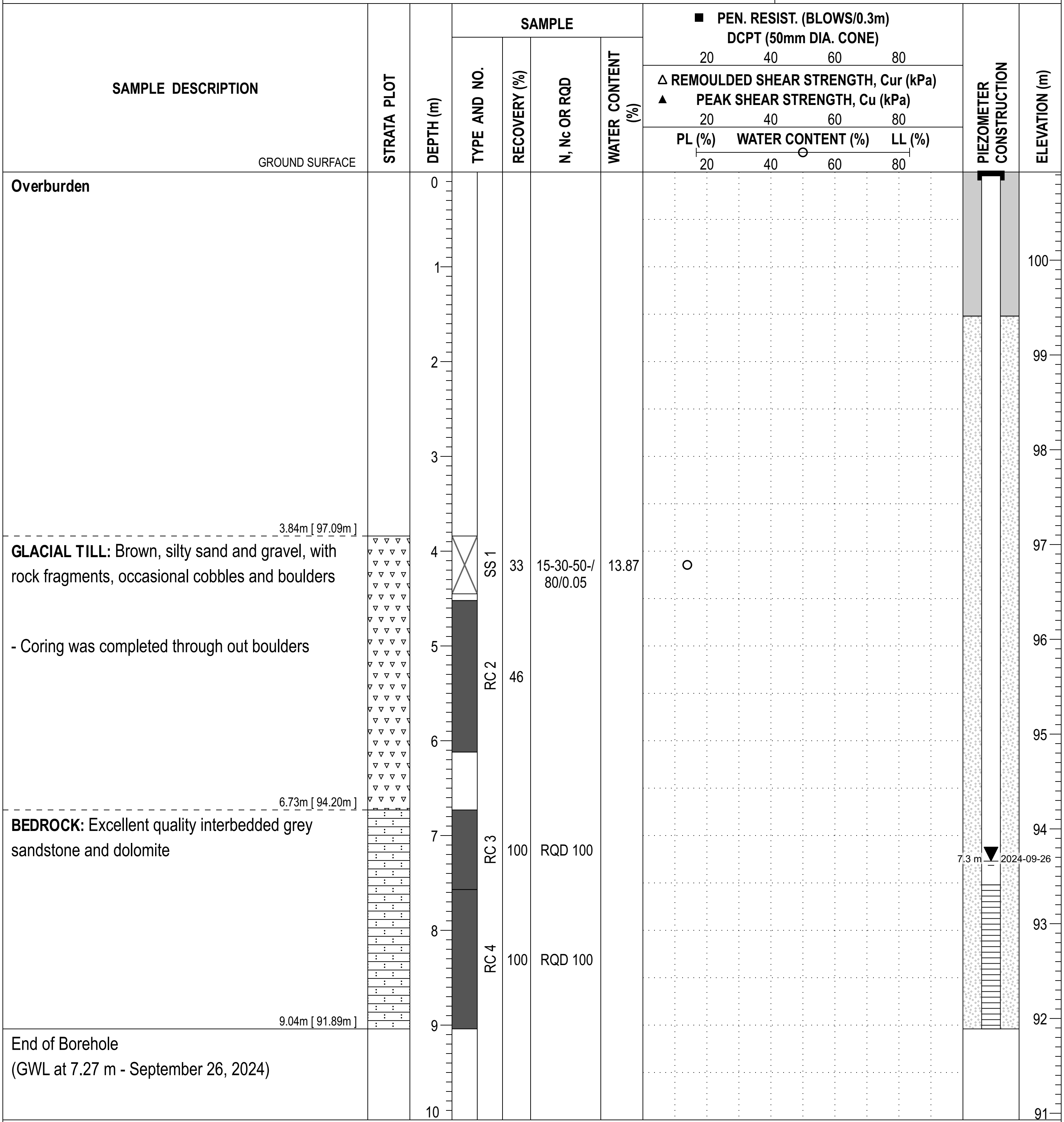
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 19, 2024

FILE NO. : PG7262

HOLE NO. : BH20-24



Overburden

3.84m [97.09m]

GLACIAL TILL: Brown, silty sand and gravel, with rock fragments, occasional cobbles and boulders

- Coring was completed through out boulders

6.73m [94.20m]

BEDROCK: Excellent quality interbedded grey sandstone and dolomite

9.04m [91.89m]

End of Borehole

(GWL at 7.27 m - September 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	33	15-30-50-/ 80/0.05	13.87
RC 2	46		
RC 3	100	RQD 100	
RC 4	100	RQD 100	

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

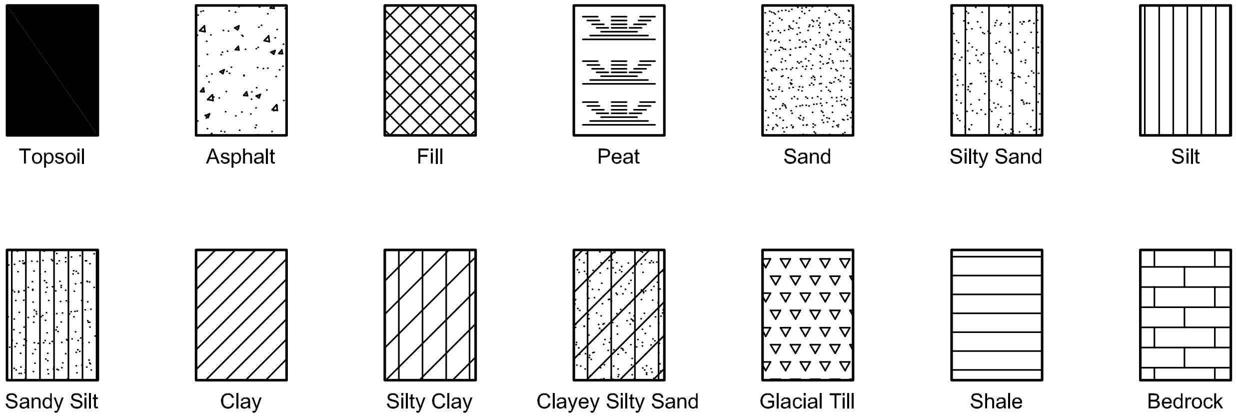
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

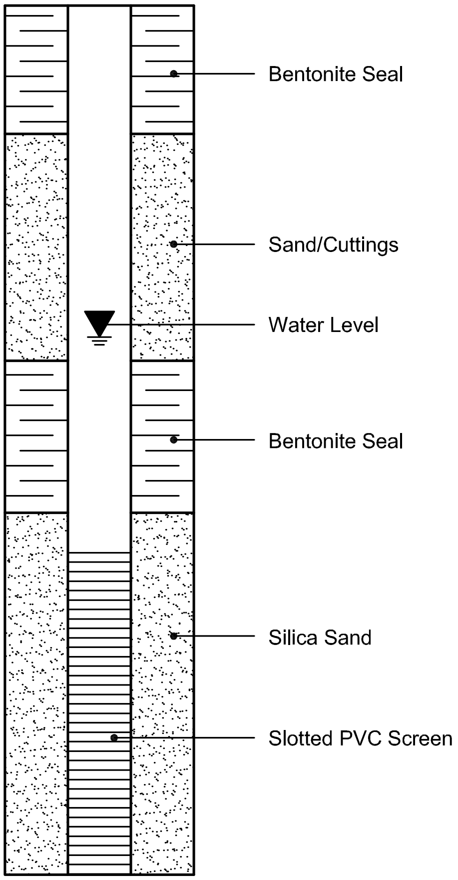
SYMBOLS AND TERMS (continued)

STRATA PLOT

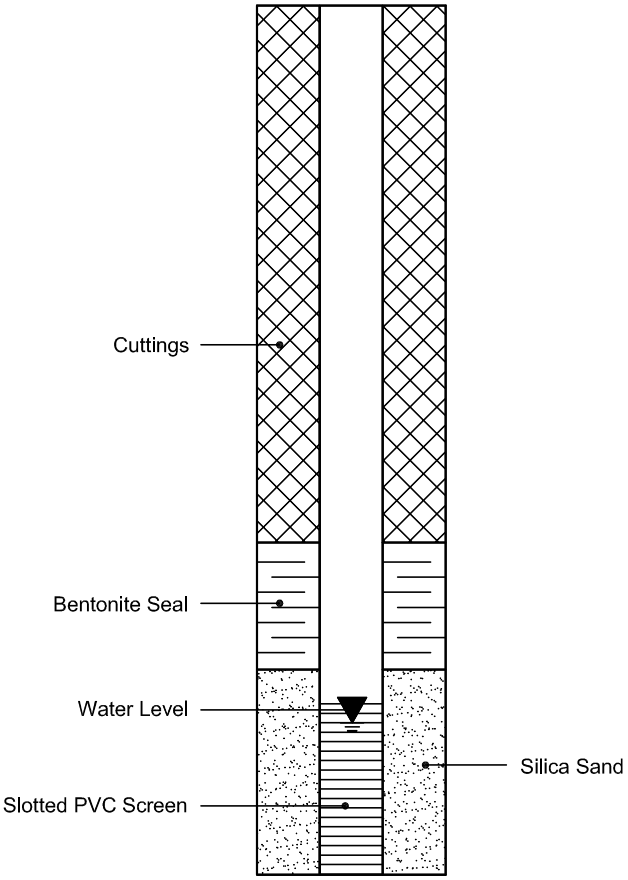


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



patersongroup Consulting Engineers

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

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Development, Bank Street at Blais Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis O'Sullivan Vollebekk Surveying.

REMARKS

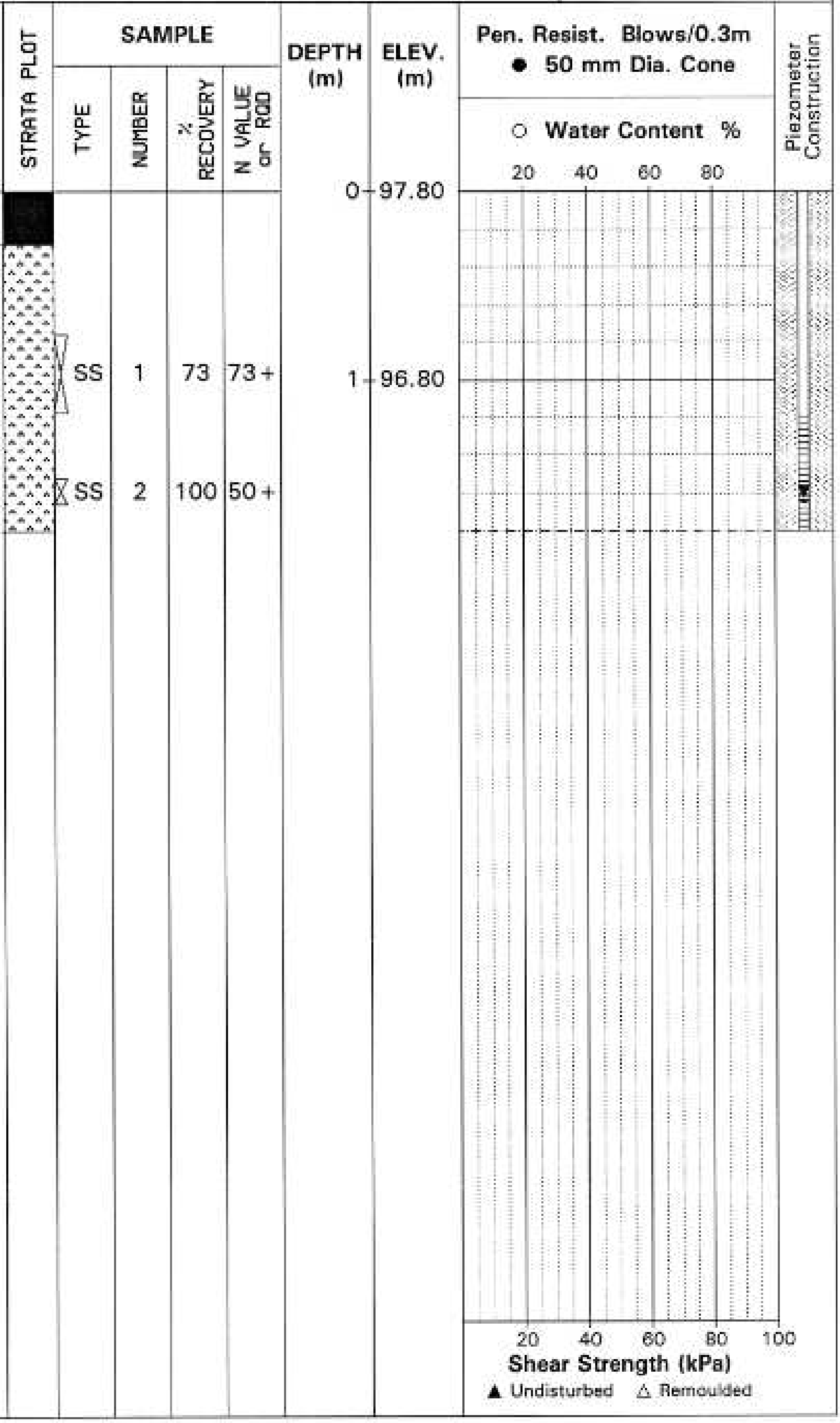
BORINGS BY CME 55 Power Auger

DATE 21 JUL 05

FILE NO. PG0627

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.80
TOPSOIL 0.28							
GLACIAL TILL: Very dense, brown sandy silt with gravel, cobbles and boulders		SS	1	73	73+	1	96.80
		SS	2	100	50+		
1.80							
End of Borehole Practical refusal to augering @ 1.80m depth (GWL @ 1.62m-Sep. 6/05)							



patersongroup Consulting Engineers

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

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Development, Bank Street at Blais Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis O'Sullivan Vollebekk Surveying.

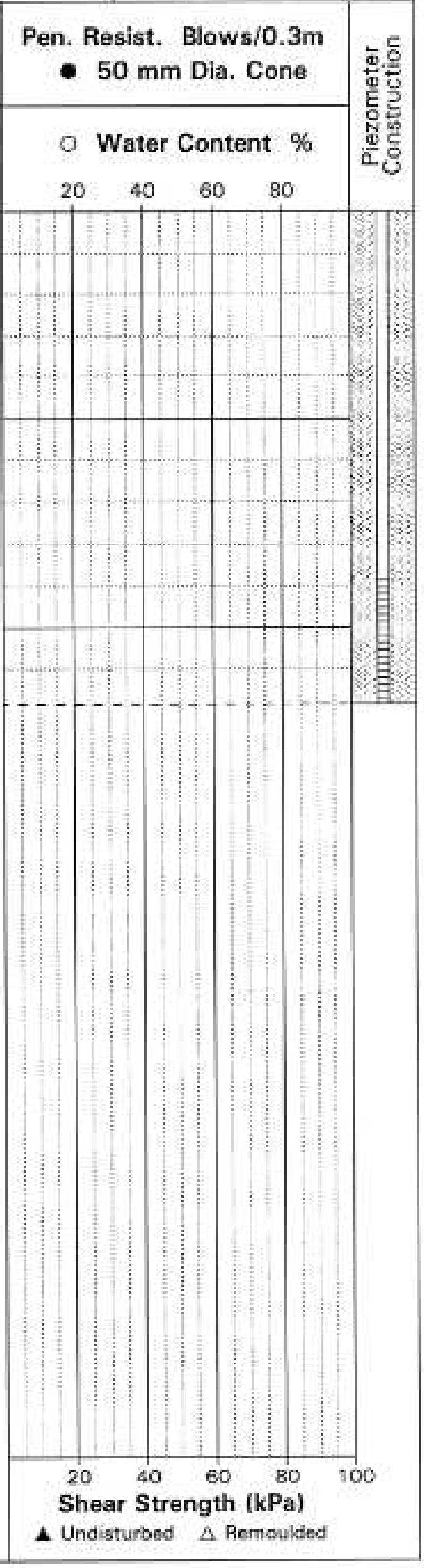
REMARKS

FILE NO. PG0627

HOLE NO. PH 4

BORINGS BY CME 55 Power Auger **DATE** 21 JUL 05

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.64
TOPSOIL 0.28	■						
GLACIAL TILL: Very dense, brown silty fine sand with gravel, cobbles and boulders	▨					1	97.64
	▨					2	96.64
2.37	▨	SS	1		50+		
End of Borehole Practical refusal to augering @ 2.37m depth (BH dry-Sep. 6/05)							



JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

90-6

CLIENT Ship & Krakow Architects

LOCATION Leitrim, Ontario

DATES: BORING 90-06-27

WATER LEVEL

PROJECT No. 30067

BOREHOLE No. 90-6

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
	99.70	Ground Surface						
0	99.4	Dark brown, TOPSOIL and ROOTMAT						
		Brown and grey, fine to medium, SAND, trace silt						
1								
	98.1	Very dense, brown, silty sand, some gravel and rock fragments at top of bedrock, TILL			SS	1	250	*
2	97.4	End of Borehole (Bedrock)						
3		* Split spoon refusal						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_p W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 9

△ Pocket Penetrometer Test

☐ Field Vane Test

Photograph of Rock Cores – BH6-24 – RC1 to RC3

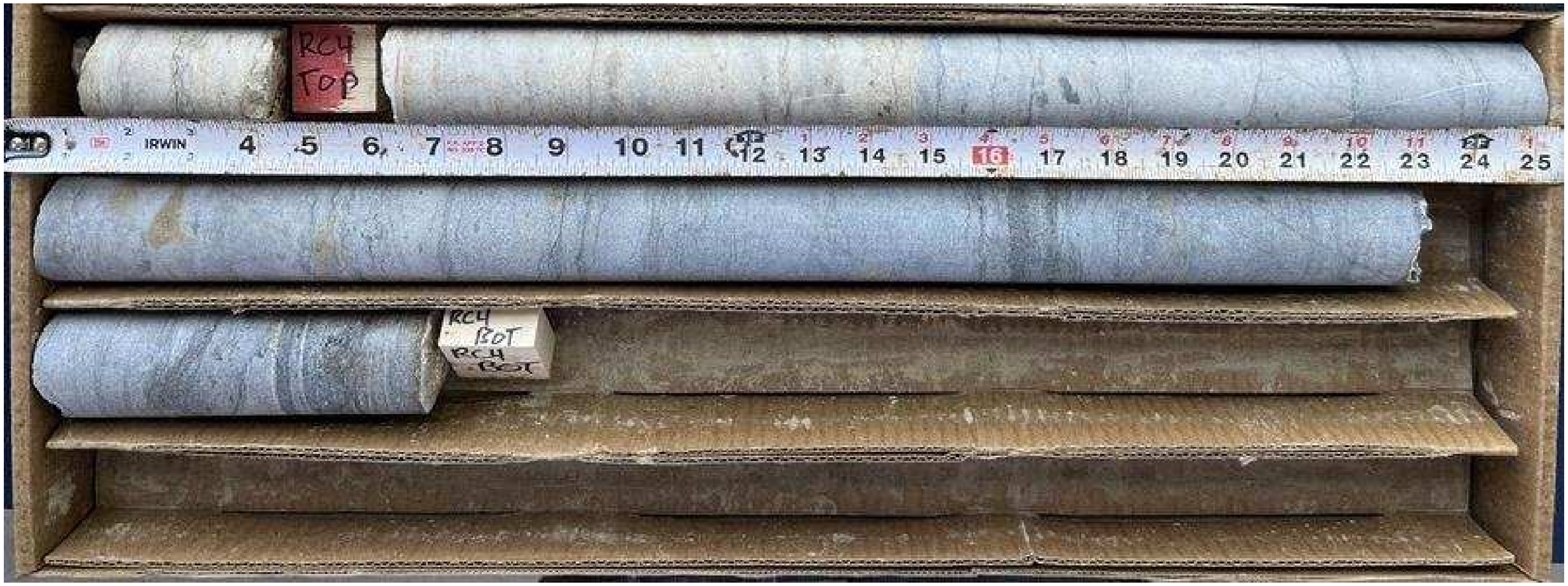
Photograph of Rock Core obtained from BH 6-24 from intervals RC1, RC2, and RC3

Rock Core RC1 interval ranged between 6’ to 9’8”; Rock Core RC2 interval ranged between 9’8” to 13’2”; Rock Core RC3 interval ranged between 13’2” to 15’3”

Recovery RC1 (%) = 19; Recovery RC2 (%) = 100; Recovery RC3 (%) = 100

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = 100; Rock Quality Designation RC3 (RQD - %) = 89

Photograph of Rock Cores – BH6-24 – RC4



Photograph of Rock Core obtained from BH 6-24 from interval RC4

Rock Core interval ranged between 15’3” to 20’2”

Recovery (%) = 100

Rock Quality Designation (RQD - %) = 100

Photograph of Rock Cores – BH7-24 – RC1 to RC4

Photograph of Rock Core obtained from BH 7-24 from intervals RC1, RC2, RC3, and RC4

Rock Core RC1 interval ranged between 8’10” to 11’; Rock Core RC2 interval ranged between 11’ to 14’1”; Rock Core RC3 interval ranged between 14’1” to 16’1”; Rock Core RC4 interval ranged between 16’1” to 19’4”

Recovery RC1 (%) = 61; Recovery RC2 (%) = 40; Recovery RC3 (%) = 42; Recovery RC4 (%) = 31

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = N/A - Boulder; Rock Quality Designation RC3 (RQD - %) = N/A - Boulder; Rock Quality Designation RC4 (RQD - %) = N/A - Boulder

Photograph of Rock Cores – BH8-24 – RC1 to RC3

Photograph of Rock Core obtained from BH 8-24 from intervals RC1, RC2, and RC3

Rock Core RC1 interval ranged between 5’4” to 9’5”; Rock Core RC2 interval ranged between 9’5” to 14’5”; Rock Core RC3 interval ranged between 14’5” to 19’9”

Recovery RC1 (%) = 24; Recovery RC2 (%) = 23; Recovery RC3 (%) = 49

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = N/A - Boulder; Rock Quality Designation RC3 (RQD - %) = N/A - Boulder

Photograph of Rock Cores – BH10-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 10-24 from intervals RC1, and RC2

Rock Core RC1 interval ranged between 8’10” to 10’2”; Rock Core RC2 interval ranged between 14’ to 19’11”

Recovery RC1 (%) = 25; Recovery RC2 (%) = 32

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = N/A - Boulder

Photograph of Rock Cores – BH12-24 – RC1 and RC2

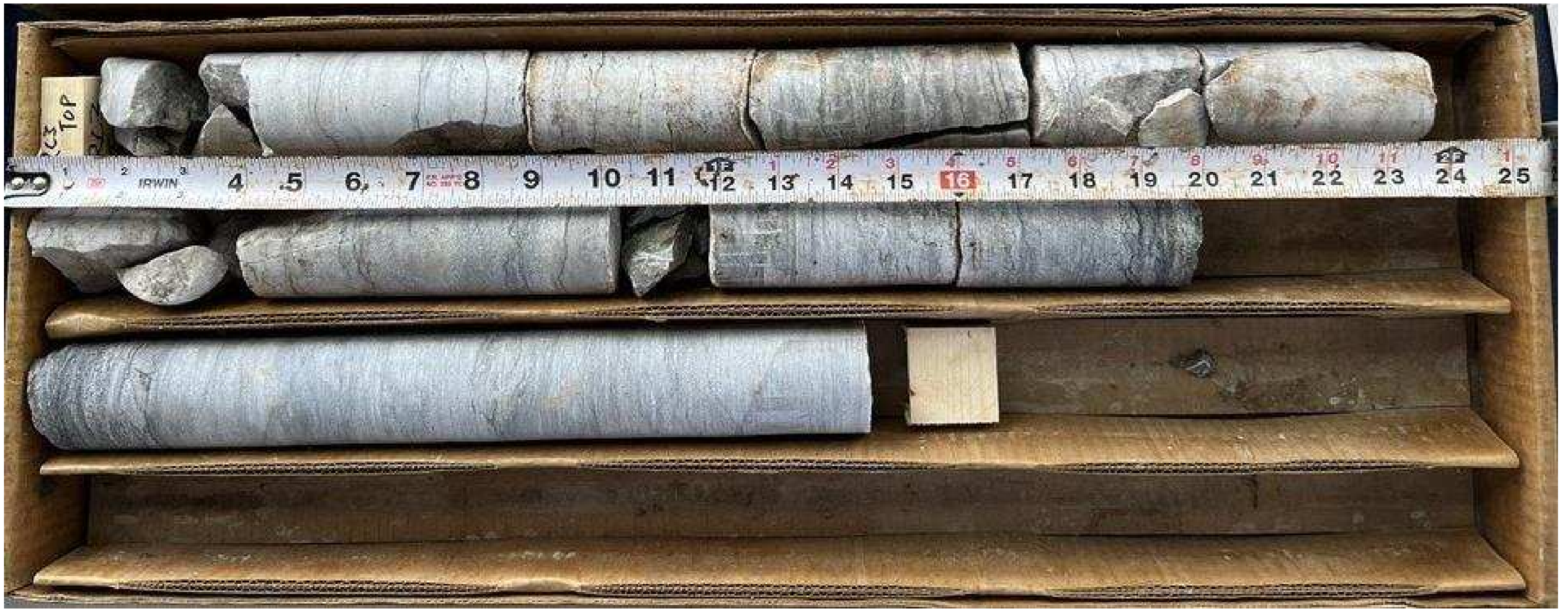
Photograph of Rock Core obtained from BH 12-24 from intervals RC1, and RC2

Rock Core RC1 interval ranged between 7’6” to 10’2”; Rock Core RC2 interval ranged between 10’2” to 15’1”

Recovery RC1 (%) = 45; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = 88

Photograph of Rock Cores – BH12-24 – RC3



Photograph of Rock Core obtained from BH 12-24 from interval RC3

Rock Core interval ranged between 15’1” to 20’

Recovery (%) = 97

Rock Quality Designation (RQD) = 86

Photograph of Rock Cores – BH19-24 – RC1 to RC4

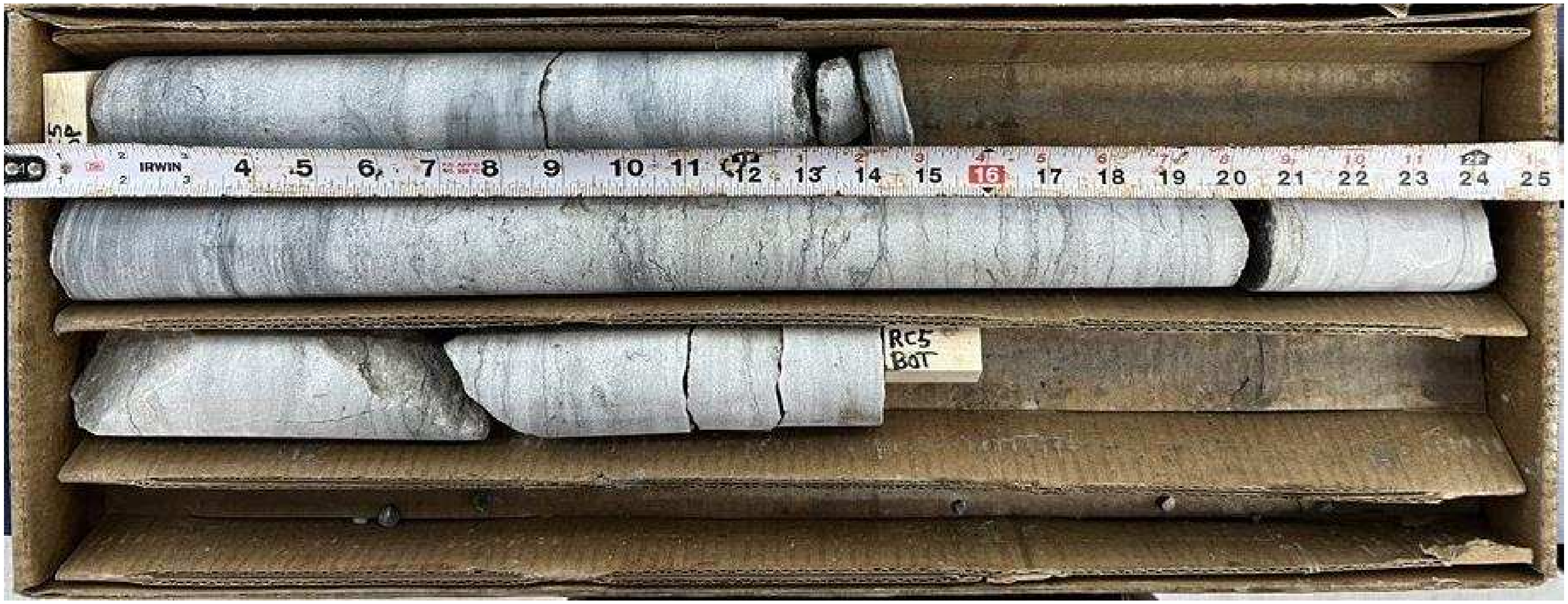
Photograph of Rock Core obtained from BH 19-24 from intervals RC1, RC2, RC3, and RC4

Rock Core RC1 interval ranged between 3’7” to 5’; Rock Core RC2 interval ranged between 5’ to 10’; Rock Core RC3 interval ranged between 10’ to 11’6”; Rock Core RC4 interval ranged between 14’11” to 19’10”

Recovery RC1 (%) = 59; Recovery RC2 (%) = 50; Recovery RC3 (%) = 0; Recovery RC4 (%) = 100

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = N/A - Boulder; Rock Quality Designation RC3 (RQD - %) = N/A - Boulder; Rock Quality Designation RC4 (RQD - %) = 73

Photograph of Rock Cores – BH19-24 – RC5



Photograph of Rock Core obtained from BH 19-24 from interval RC5

Rock Core interval ranged between 19’10” to 24’4”

Recovery (%) = 94

Rock Quality Designation (RQD) = 93

Photograph of Rock Cores – BH20-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 20-24 from intervals RC1, and RC2

Rock Core RC1 interval ranged between 14’10” to 20’1”; Rock Core RC2 interval ranged between 19’6” to 24’10”

Recovery RC1 (%) = 46; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD - %) = N/A - Boulder; Rock Quality Designation RC2 (RQD - %) = 100

Photograph of Rock Cores – BH20-24 – RC3

Photograph of Rock Core obtained from BH 20-24 from interval RC3

Rock Core RC3 interval ranged between 24’10” to 29’8”

Recovery RC3 (%) = 100

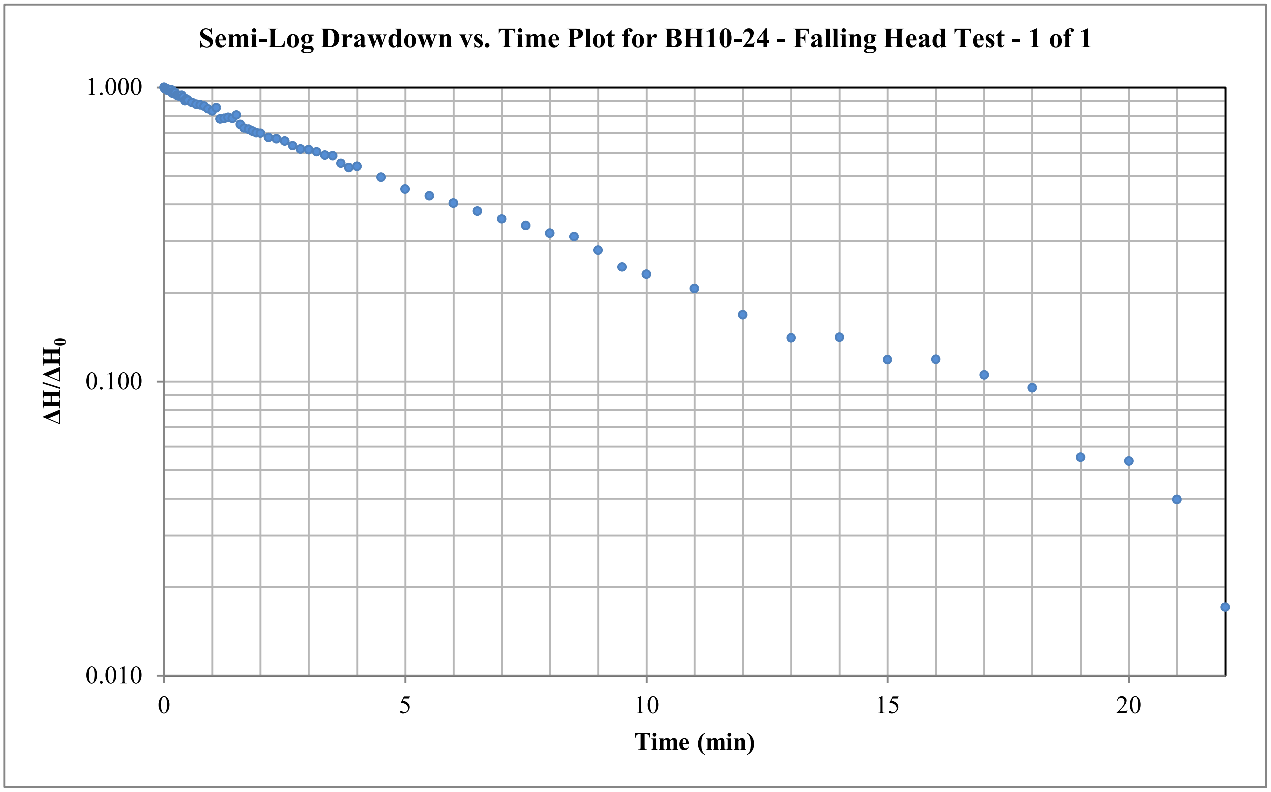
Rock Quality Designation RC3 (RQD - %) = 100

Client ID:		BH8 - 24 SS4	-	-	-	-	-
Sample Date:		16-Sep-24 09:00	-	-	-		
Sample ID:		2438279-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	93.0	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.37	-	-	-	-	-
Resistivity	0.1 Ohm.m	53.4	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	71	-	-	-	-	-

	Client ID:	BH13-24 SS4	-	-	-		
	Sample Date:	17-Sep-24 09:00	-	-	-	-	-
	Sample ID:	2438297-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	92.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.59	-	-	-	-	-
Resistivity	0.1 Ohm.m	57.3	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	38	-	-	-	-	-

Hvorslev Hydraulic Conductivity Analysis

Project: Bank & Dun Developments Inc. - 4828 Bank Street
Test Location: BH10-24
Test: Falling Head - 1 of 1
Date: September 26, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

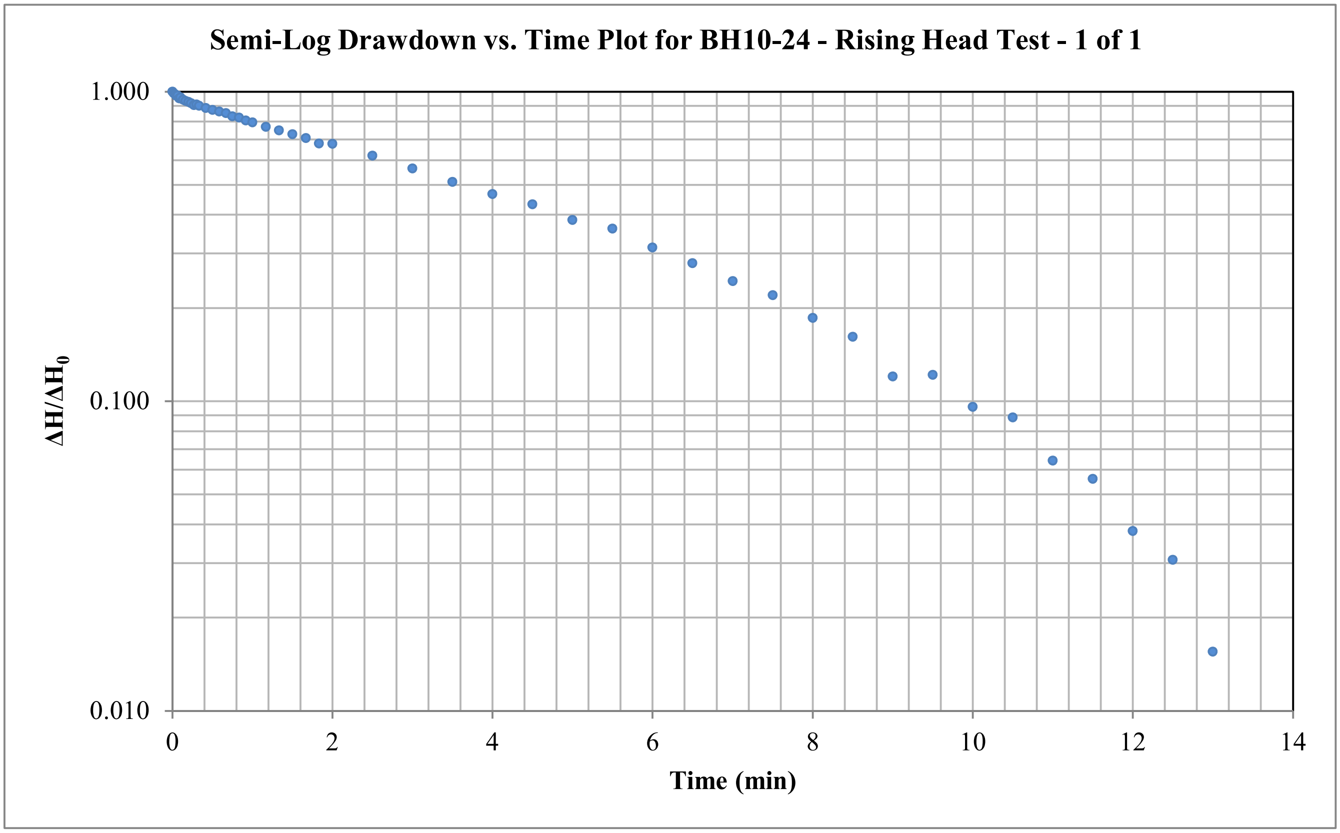
Data Points (from plot):

t^*: 6.640 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 9.54E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Bank & Dun Developments Inc. - 4828 Bank Street
Test Location: BH10-24
Test: Rising Head - 1 of 1
Date: September 26, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.0721

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

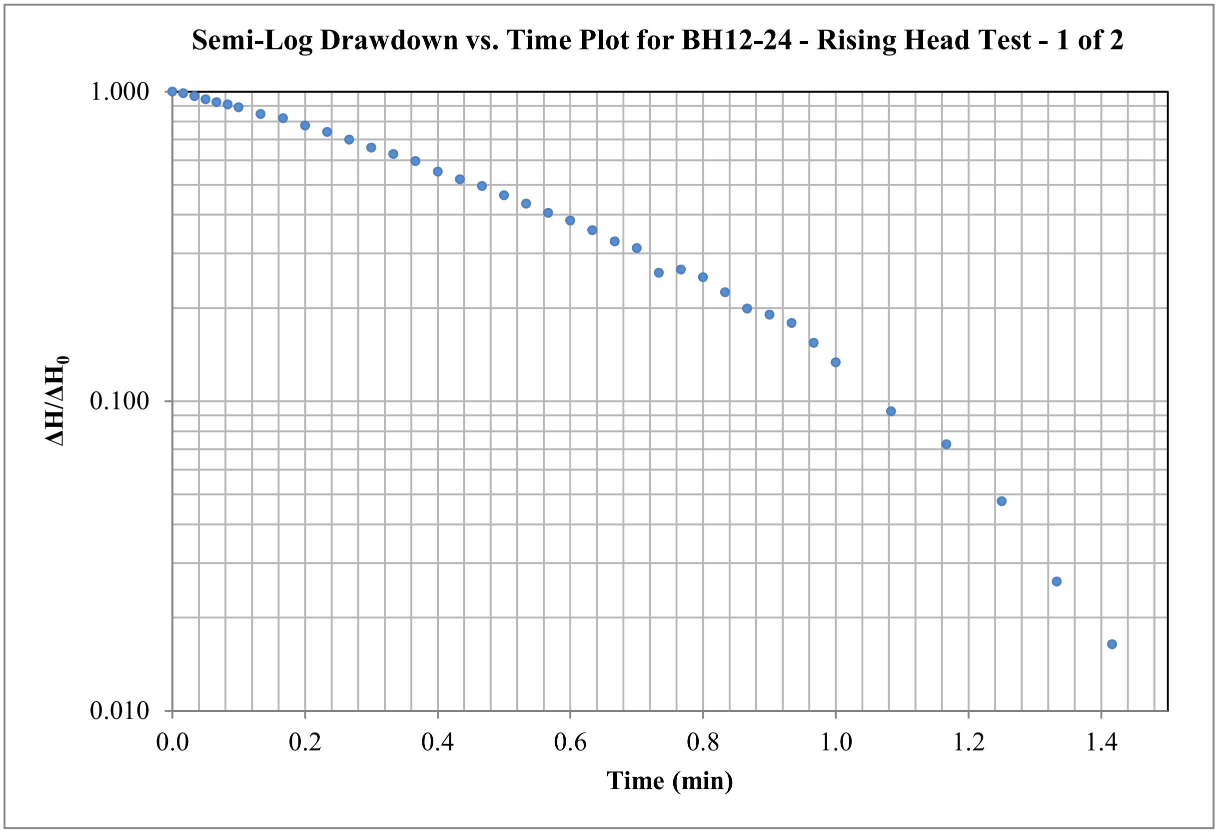
Data Points (from plot):

t*: 5.148 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.23E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Bank & Dun Developments Inc. - 4828 Bank Street
Test Location: BH12-24
Test: Rising Head - 1 of 2
Date: September 26, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^{\,2}}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

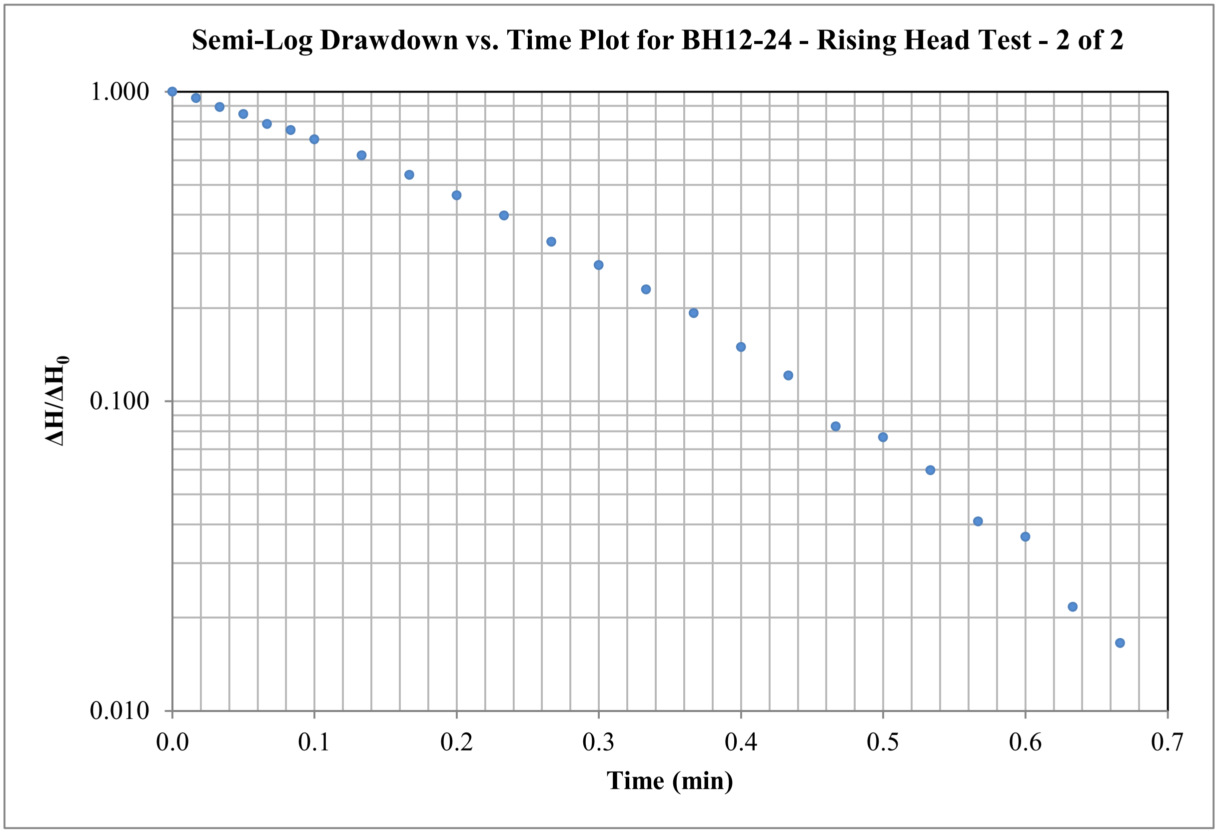
Data Points (from plot):

t*: 0.613 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.03E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Bank & Dun Developments Inc. - 4828 Bank Street
Test Location: BH12-24
Test: Rising Head - 2 of 2
Date: September 26, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t*: 0.244 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.59E-05 m/sec

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG7262-1 – TEST HOLE LOCATION PLAN

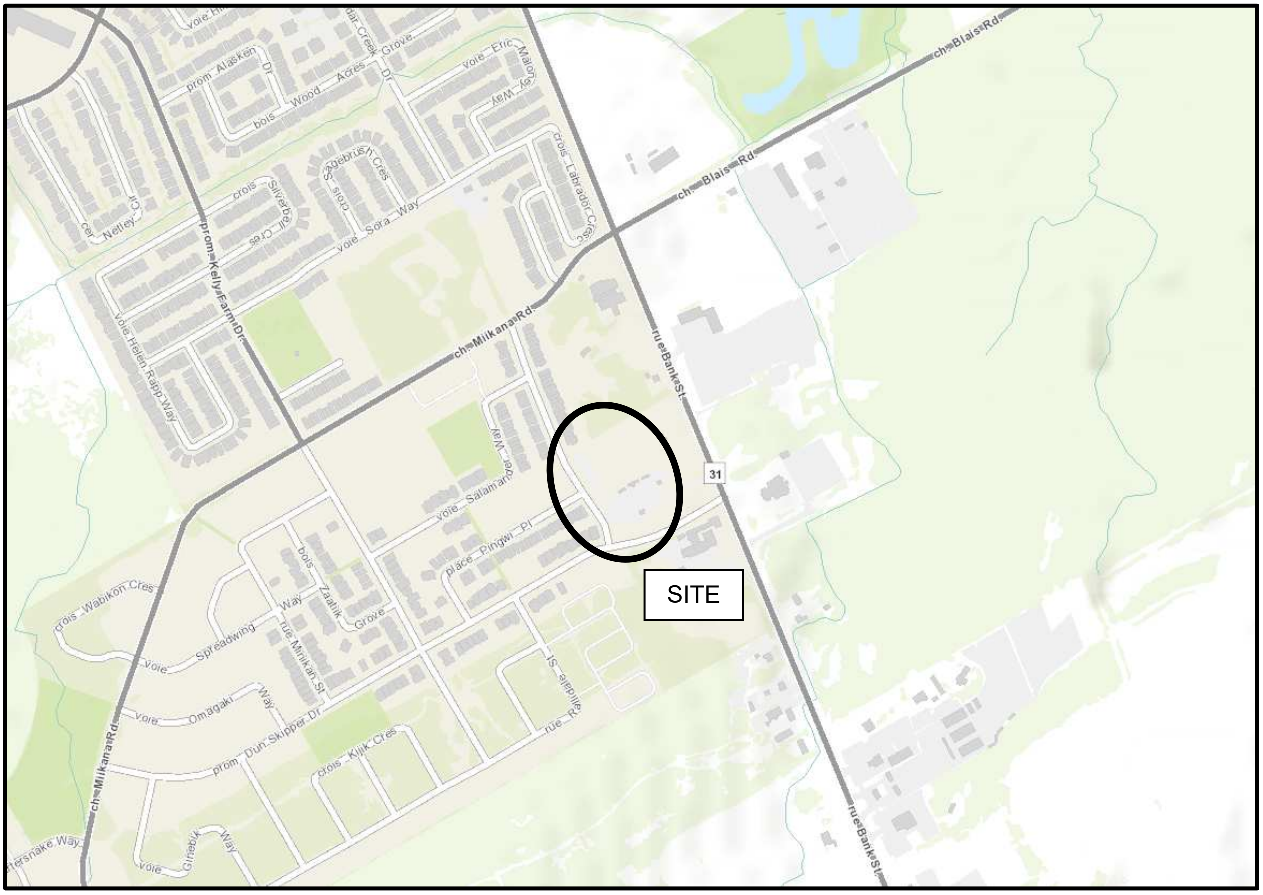


FIGURE 1

KEY PLAN

PATERSON
GROUP

Figure 2 – Shear Wave Velocity Profile at Shot Location -3.5 m

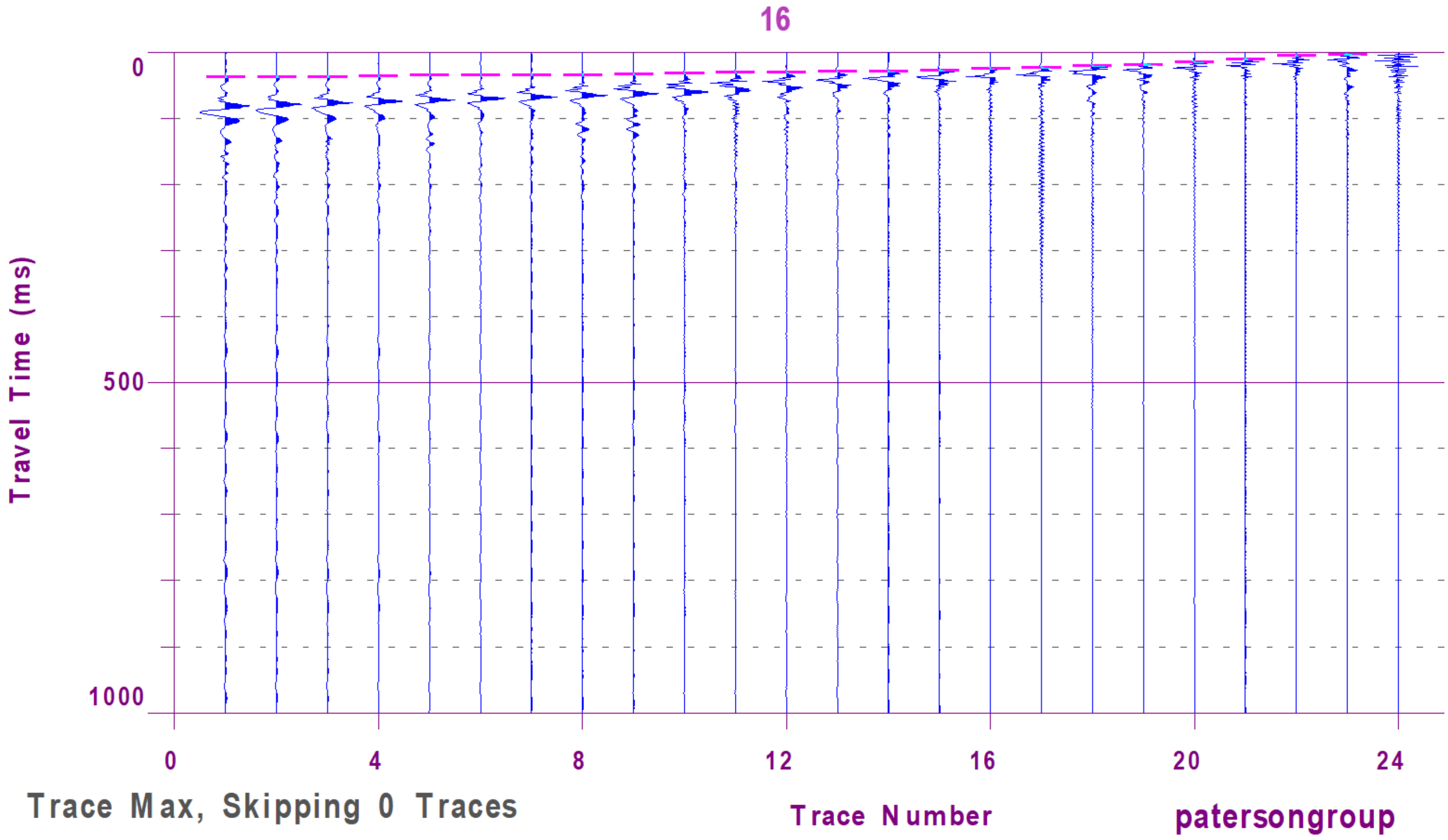
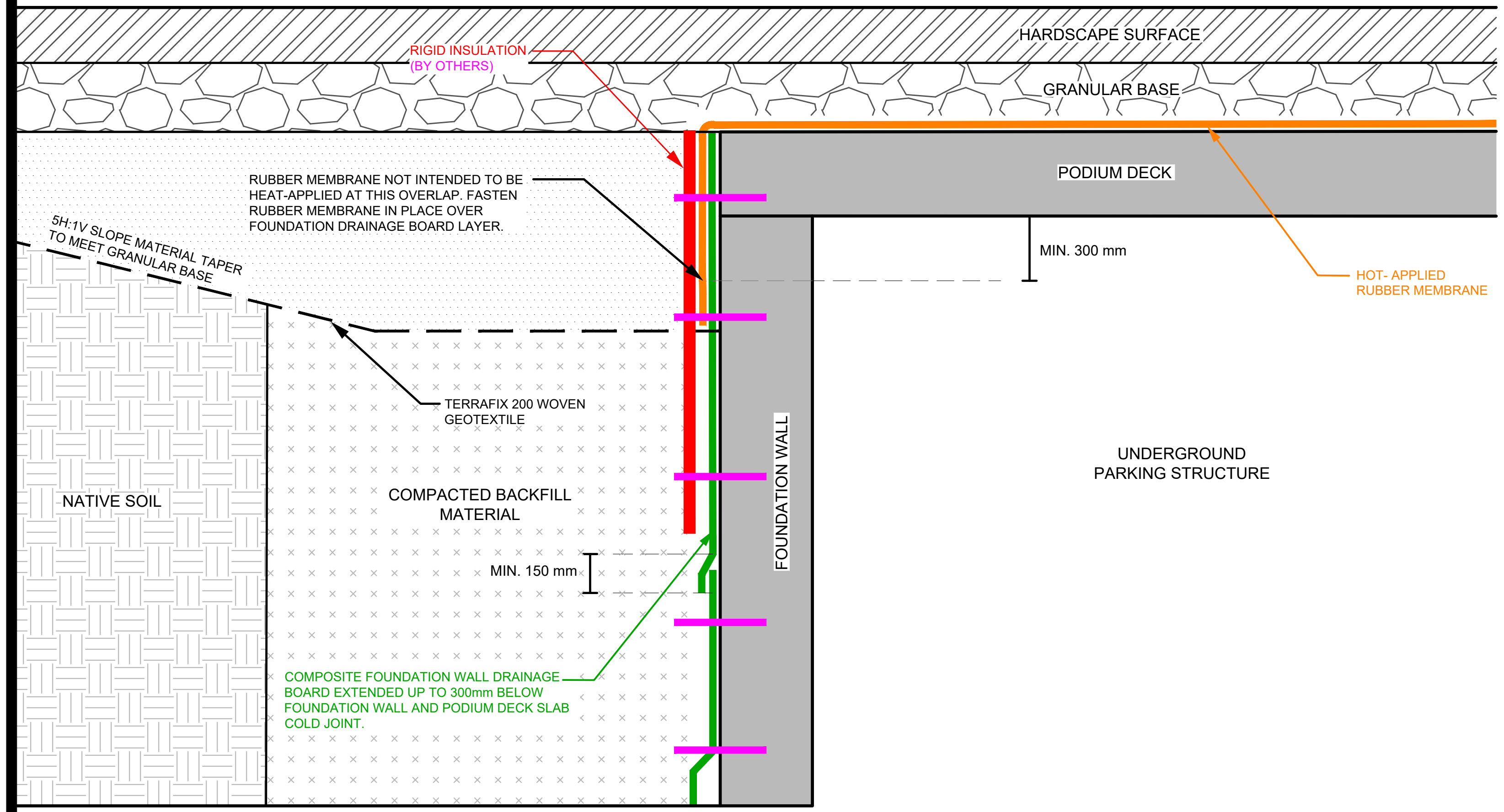
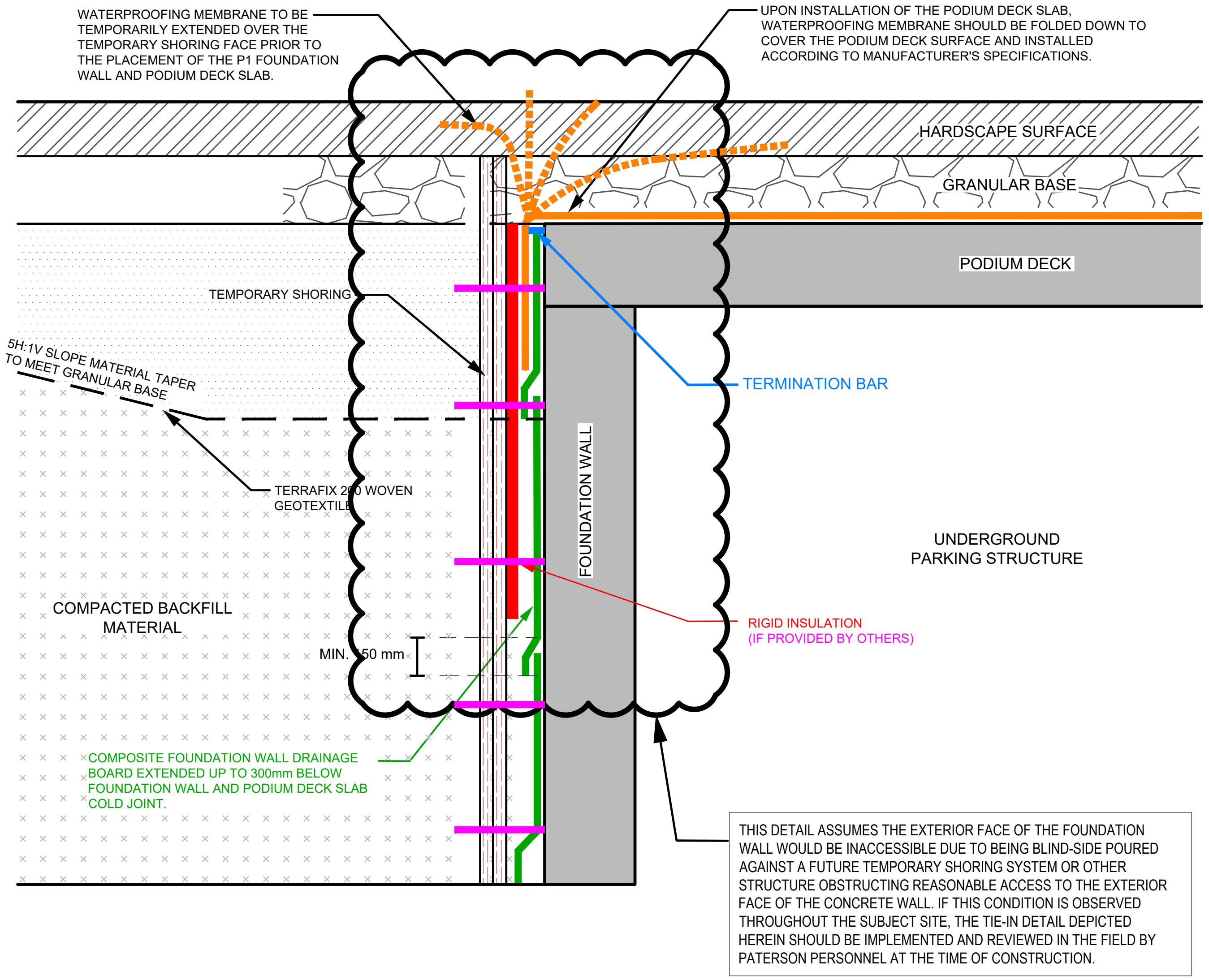


Figure 3 – Shear Wave Velocity Profile at Shot Location 48 m

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL


WATERPROOFING MEMBRANE TO BE TEMPORARILY EXTENDED OVER THE TEMPORARY SHORING FACE PRIOR TO THE PLACEMENT OF THE P1 FOUNDATION WALL AND PODIUM DECK SLAB.

UPON INSTALLATION OF THE PODIUM DECK SLAB, WATERPROOFING MEMBRANE SHOULD BE FOLDED DOWN TO COVER THE PODIUM DECK SURFACE AND INSTALLED ACCORDING TO MANUFACTURER'S SPECIFICATIONS.

THIS DETAIL ASSUMES THE EXTERIOR FACE OF THE FOUNDATION WALL WOULD BE INACCESSIBLE DUE TO BEING BLIND-SIDE POURED AGAINST A FUTURE TEMPORARY SHORING SYSTEM OR OTHER STRUCTURE OBSTRUCTING REASONABLE ACCESS TO THE EXTERIOR FACE OF THE CONCRETE WALL. IF THIS CONDITION IS OBSERVED THROUGHOUT THE SUBJECT SITE, THE TIE-IN DETAIL DEPICTED HEREIN SHOULD BE IMPLEMENTED AND REVIEWED IN THE FIELD BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION.

NOTES:

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

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

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

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

NO.	REVISIONS	DATE	INITIAL

BANK & DUN DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4828 BANK STREET

OTTAWA, **ONTARIO**

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

Scale: N.T.S	**Date:** 10/2024
Drawn by: ZS	**Report No.:** PG7262
Checked by: YZ	**Dwg. No.:** FIGURE 4
Approved by: JV	**Revision No.:**

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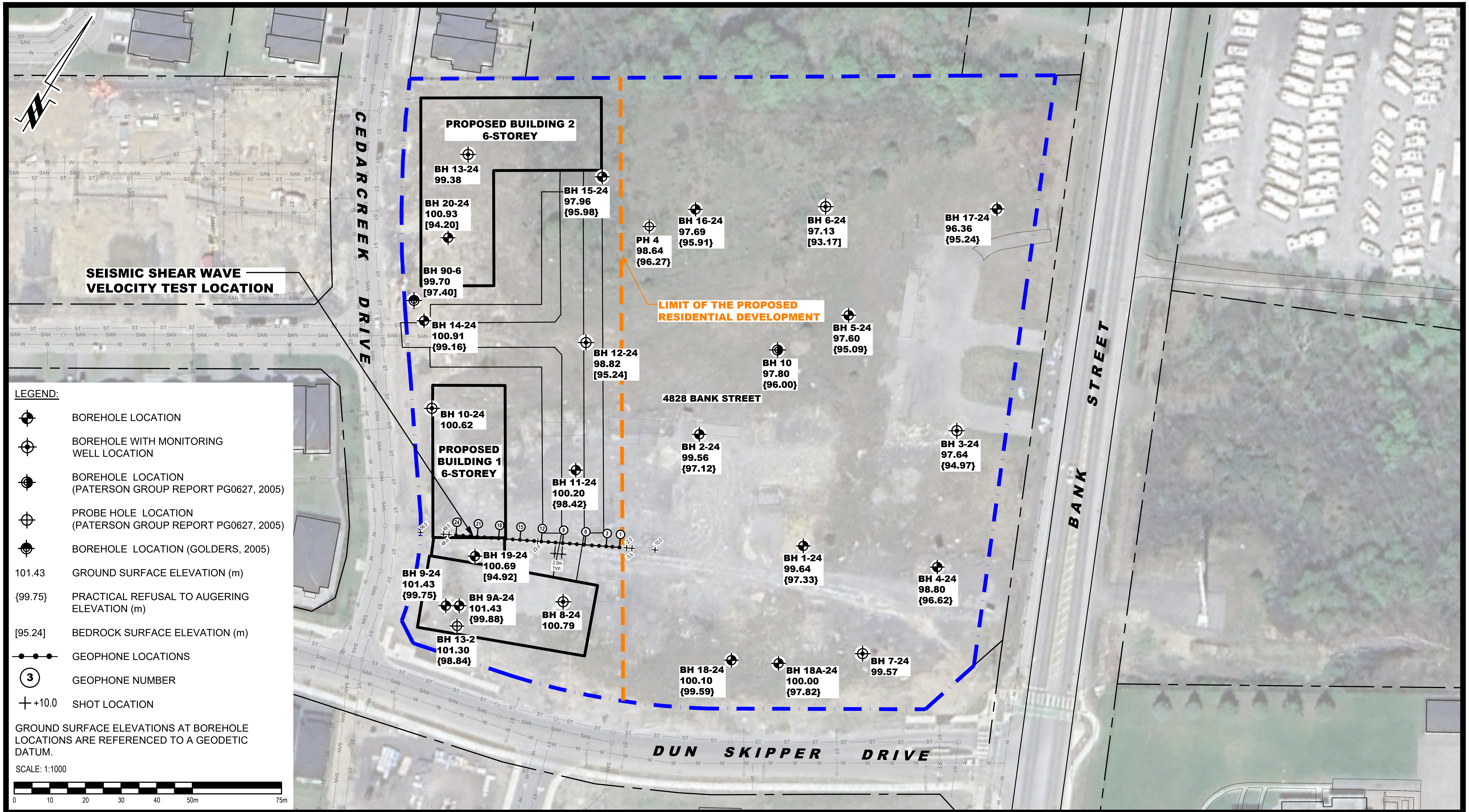

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BANK & DUN DEVELOPMENTS INC.
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OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 10/2024
Drawn by: GK
Report No.: PG7262-1
Checked by: YZ
Dwg. No.: PG7262-1
Approved by: JV
Revision No.:

11x17

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