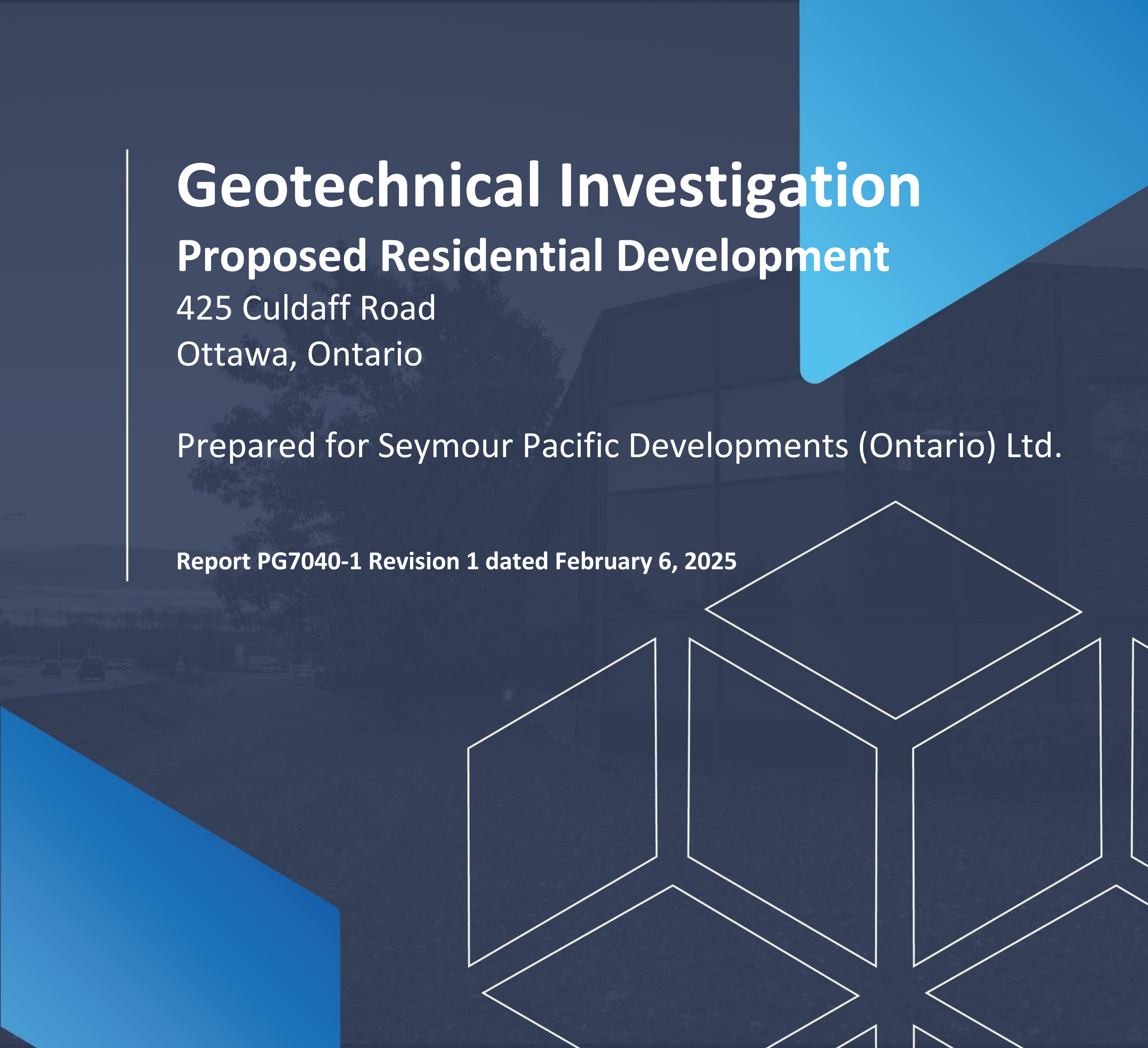

Geotechnical Investigation

Proposed Residential Development

425 Culdaff Road
Ottawa, Ontario

Prepared for Seymour Pacific Developments (Ontario) Ltd.

Report PG7040-1 Revision 1 dated February 6, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Seymour Pacific Developments (Ontario) Ltd. to conduct a geotechnical investigation for the proposed development to be located at 425 Culdaff Road in the City of Ottawa (reference should be made 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 test holes.

☐ 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 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

It is understood that the proposed development is anticipated to consist of a six-storey residential structure which will be provided with one basement level of underground parking matching the footprint of the overlying structure. Associated access lanes, walkways and landscaped areas are also anticipated as part of the proposed development. It is expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on March 25 to March 26 and May 2, 2024, and consisted of advancing a total of seven (7) boreholes, four (4) probeholes, and nine (9) test pits to a maximum depth of 7.7 m below existing ground surface. A previous investigation was undertaken by Paterson in March of 2018. At that time, one (1) test pit was advanced within the subject site to maximum depth of 1.5 m.

The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG7040-1 - Test Hole Location Plan included in Appendix 2.

Boreholes and probeholes were advanced using a low-clearance rubber-track mounted drill rig operated by a two-person crew. The drilling procedure consisted of augering and/or coring to the required depths at the selected locations and sampling the overburden soils and bedrock. The fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Test pits were advanced using a hydraulic shovel and backfilled with the excavated soil upon completion. The test pit procedure consisted of excavating to the required depth at the selected locations and sampling the overburden. The fieldwork was conducted under the full-time supervision of our personnel.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, using a 50 mm diameter split-spoon sampler, or core recovery barrels during the drilling program. Grab samples were collected from the test pits at selected intervals. The split-spoon, auger samples, and grab samples were classified on site and placed in sealed plastic bags.

Rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination. The depths at which the grab samples, split-spoon, auger flights, and rock core samples were recovered from the test holes are shown as G, SS, AU, and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Diamond drilling was completed at boreholes BH 1-24 and BH 2-24 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 thickness of the overburden was also evaluated by the use of probeholes at several test hole locations. This technique consisted of advancing augers to the depth of practical auger refusal. Sampling of the overburden was not undertaken at the probehole locations.

Slug testing (falling head testing) was completed at groundwater monitoring well locations installed during the field program to establish the estimated hydraulic conductivity of the underlying soil deposit and bedrock formation.

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

Groundwater

Groundwater monitoring wells were installed at BH 1-24 and BH 4B-24, however the remaining boreholes and probeholes were fitted with a flexible polyethylene standpipe with the exception of PH 2-24 to BH 4-24. Monitoring wells and standpipes were installed to allow groundwater level monitoring subsequent to the completion of the field program.

Typical monitoring well construction details are described below:

- Slotted 32- or 51-mm diameter PVC screen at the base of the aforementioned 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.

The groundwater level readings were obtained after a suitable stabilization period following the completion of the field investigation. Groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Groundwater Level Monitoring

Submersible dataloggers (TD-Diver, VanEssen Instruments) were installed in both monitoring wells on April 2, 2024, to record groundwater levels, primarily over the spring months and early summer. The datalogger can measure the equivalent hydrostatic pressure of the water above the sensor diaphragm to calculate the total water depth. The groundwater monitoring program extended from April 2 to August 8, 2024.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum. Reference should be made to Drawing PG7040-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. One sample of recovered bedrock core was submitted for uniaxial compressive strength testing.

3.4 Analytical Testing

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

3.5 In-Situ Infiltration Testing

In-situ infiltration testing was conducted using a Pask (Constant Head Well) Permeameter to estimate infiltration rates of the unsaturated surficial soils at the subject site. The tests were conducted at three (3) test pit locations. The test pits were excavated in approximately 0.5 m increments to allow for safe entry into the pits, as well as infiltration testing to be conducted at different elevations.

At each location, two (2) to three (3) infiltration tests were conducted. At approximately 0.3 to 0.5 m above each testing elevation, an 83 mm auger hole was excavated to the desired testing elevation using a Riverside/Bucket. Soils from the auger flights were visually inspected and initially classified on-site. The tests were conducted by filling the permeameter reservoir with water and inverting it into the hole, ensuring it was relatively vertical and rested at the bottom of the hole.

The water level of the reservoir was monitored at 0.5-to-5-minute intervals until the rate of fall out of the permeameter reached equilibrium, known as quasi “steady state” flow rate. Quasi steady state flow can be considered to have been obtained after measuring 3 to 5 consecutive rate of fall readings with identical values. The values for the steady state rate of fall were recorded for each location.

The steady state rate of fall was converted to a field saturated hydraulic conductivity value (K_{fs}) using the Engineering Technology Canada Ltd. conversion tables. Unfactored infiltration rates were estimated based on the methodology outlined in the Ontario Ministry of Municipal Affairs and Housing – Supplementary Guidelines to the Ontario Building Code, 1997 – SG-6 – Percolation Time and Soil Descriptions. The testing results are further discussed in Subsection 4.4.

3.6 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at each monitoring well location to provide insight on the hydraulic properties of the overburden material and bedrock at the subject site. The testing results will be used to estimate potential groundwater infiltration volumes during construction. 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 overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a saturated screen length of 1.3 to 1.5 m and a diameter of 0.03 to 0.05 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 Horslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. The semi-log drawdown vs. time plots for rising and falling head tests at each borehole location are presented in Appendix 1.

The results of testing and hydrogeological recommendations are further discussed in Subsections 4.5.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of undeveloped vacant land with several relatively small piles of soil fill located throughout the parcel. The site was observed to be relatively flat and approximately at grade with adjacent roadways and neighboring properties. The site is bordered by vacant land to the north and west, Derreen Avenue to the east, and Culdaff Road to the south.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of fill underlain by compact to very dense glacial till and further by the bedrock formation. The fill was generally observed to consist of brown silty sand or clay, crushed stone, cobbles, boulders, and organics.

The glacial till layer was encountered at every test hole, with the exception of TP 4-24, TP 5-24 and BH 6-24, and extended to between 1.1 to 5.2 m below ground surface. The glacial till was observed to consist of brown silty sand, gravel, cobbles and boulders with traces of clay and in compact to very dense state of compactness.

Practical refusal to augering and excavation was encountered at each test hole, with the exception of BH 4-24, at depths ranging from 2.2 to 5.3 m below ground surface, respectively.

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

Limestone bedrock was cored in BH 1-24 and BH 2-24 to a depth of 7.7 and 6.1 m below ground surface, respectfully. The recorded average RQD value ranged from 45 to 91, while the recovery values were consistently 100 %. Based on these results the quality of the bedrock ranges from fair to excellent quality.

Based on available geological mapping, the bedrock in the subject area consists of interbedded Limestone and Dolomite of the Gull River Formation, with an overburden thickness of 0 to 10 m.

Reference can be made to Drawing PG7040-2 - Bedrock Contour Plan for the test hole locations and depth which bedrock had been encountered and/or where refusal to augering and/or excavation had been encountered (which was used to infer the bedrock surface). It should be understood that the bedrock contours depicted on the aforementioned contour plan are based on linear interpolation between test holes where bedrock had been confirmed by Paterson and is limited to that information. Actual site conditions and bedrock depths/elevation may vary beyond the test hole locations and as inferred by the depicted contour lines. Based on this, it is possible that the bedrock surface may vary within plus or minus 500 mm to 1 m at contour line locations and where the bedrock surface has not been discretely confirmed by a test hole.

Unconfined Compressive Strength Testing of Bedrock Core Samples

One (1) select bedrock core sampled obtained by Paterson as part of the current investigation was tested for unconfined compressive strength. The results of the test are summarized in Table 1 below and presented on Unconfined Compressive Strength Testing Results on Appendix 1.

Table 1 – Summary of Unconfined Bedrock Compressive Strength Testing Results

Borehole	Sample	Test Core Depth (m)	Test Core Elevation (m)	Unconfined Compressive Strength (MPa)
BH 2-24	RC2	3.66	104.12	44.7

4.3 Groundwater

Groundwater levels were manually measured in the installed monitoring wells and piezometers during the current investigation and are summarized in Table 2 below. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. All test pits were dry upon completion. In addition, daily groundwater level data is currently being collected from the dataloggers that were installed on April 1, 2024. The geotechnical report will be revised at a later date with the monitoring results upon completion of the monitoring program.

In addition to manual water level measurements, a groundwater monitoring program was carried out at the subject site. The groundwater monitoring program provides an overview of the variations of the monitoring well water levels based on seasonal fluctuations. The monitoring wells were equipped with submersible dataloggers (TD-Diver, VanEssen Instruments) to accurately monitor fluctuations in the water levels. Dataloggers were programmed to continuously measure and record water levels at a fixed rate of one (1) reading every 24 hours for approximately 4 months.

Table 2 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-24	107.79	4.27	103.52	April 1, 2024
BH 2-24	107.78	3.80	103.98	
BH 3-24	107.75	1.26	106.49	
BH 4B-24	107.85	4.20	103.65	
BH 5-24	107.59	2.32	105.27	
BH 6-24	108.02	Dry	Dry	
BH 7-24	108.04	Dry	Dry	
PH 1-24	107.49	Dry	Dry	
BH 1-24	107.79	3.94	103.85	August 8, 2024
BH 4B-24	107.85	3.91	103.94	

Notes: The test holes were surveyed with respect to a geodetic datum.

The monitoring program was undertaken from April 2024 to August 2024. The monitoring data was compared with Environment and Natural Resources Canada precipitation data from the Ottawa International Airport over the same timeframe as part of the monitoring program. The groundwater monitoring results are presented in Appendix 1.

It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

4.4 In-Situ Infiltration Testing Results

In-situ infiltration tests were conducted at three (3) test pit locations to provide general coverage of the subject site on May 2, 2024. Field saturated hydraulic conductivity (K_{fs}) values and estimated unfactored infiltration rates are presented in Table 3 below.

Field saturated hydraulic conductivity values were determined using the Engineering Technologies Canada Ltd. (ETC) reference tables provided in the most recent ETC Pask Permeameter User Guide dated July 2018. Unfactored infiltration rates were estimated based on the methodology outlined in the Ontario Ministry of Municipal Affairs and Housing – Supplementary Guidelines to the Ontario Building Code, 1997 – SG-6 – Percolation Time and Soil Descriptions.

Table 3 – Field Saturated Hydraulic Conductivity Results and Estimated Unfactored Infiltration Rates

Test Pit ID	Ground Surface Elevation (m)	Infiltration Testing Depth (m)	Infiltration Testing Elevation	K_{fs} (m/sec)	Infiltration Rate (mm/hr)	Soil Type
TP 1-24	108.03	1.40	106.63	$1.6x10^{-5}$	97	Fill
		2.40	105.63	$5.3x10^{-6}$	72	Fill
		3.40	104.63	$2.7x10^{-7}$	33	Glacial Till
TP 2-24	108.05	1.50	106.55	$1.7x10^{-8}$	15	Fill
		2.50	105.55	$2.3x10^{-8}$	17	
TP 4-24	107.92	1.40	106.52	$5.3x10^{-7}$	39	Fill
		1.90	106.02	$5.3x10^{-7}$	39	Fill
		2.40	105.52	$1.1x10^{-5}$	88	Fill

The observed K_{fs} values and unfactored infiltration rates of the shallow unsaturated soils at the subject site ranged between 1.73×10^{-8} to 1.60×10^{-5} m/sec and 15 to 97 mm/hr, respectively.

The large range of observed K_{fs} values and unfactored infiltration rates are due to the variability in composition and consistency of the material encountered across the subject site but are generally consistent with similar material Paterson has encountered on other sites and typical published values.

It is important to note that the estimated infiltration rates derived from the K_{fs} values are unfactored. Prior to use for design purposes, a safety correction factor will need to be applied to the above infiltration rates. It should also be noted that for most LID measures, the invert of the system should be planned to be in accordance with the latest and pertinent City of Ottawa design guidelines, which are anticipated to require a minimum separation of 1 m above the seasonally high groundwater table and bedrock surface. Additional testing may be required depending on the depth and size of the proposed LID system.

4.5 Hydraulic Conductivity (Slug) Testing Results

Hydraulic conductivity (slug) tests were conducted at two (2) monitoring well locations on April 2, 2024, to provide information regarding the hydraulic properties of the overburden material and bedrock at the subject site. The hydraulic conductivity results are shown in Table 4 below and summarized in Appendix 1.

Table 4 – Summary Of Hydraulic Conductivity Testing Results.

Borehole ID	Ground Surface Elevation (m)	Slug Testing Depth (m)	Slug Testing Elevation (m)	K (m/sec)	Test Type	Material at Testing Depth
BH 1-24	107.79	6.2-7.7	101.59-100.09	$2.84x10^{-5}$	Falling Head	Bedrock
				$3.18x10^{-5}$	Rising Head	
BH 4B-24	107.85	3.8-5.3	104.05-102.55	$2.33x10^{-4}$	Rising Head	Glacial Till
				$1.26x10^{-4}$	Rising Head	

The measured hydraulic conductivity (K) values of the bedrock and glacial till ranged between 2.84 x 10^{-5} to 3.18 x 10^{-5} m/sec and 1.26 x 10^{-4} to 2.33 x 10^{-4} m/sec, respectively. The results are consistent with similar materials Paterson has encountered on other sites and typical published values for bedrock and glacial till with a sandy matrix.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be supported by conventional spread footing foundations founded upon compact to dense, undisturbed, in-situ glacial till and/or placed directly upon a clean, surface sounded bedrock. Consideration may also be given to indirectly placing footings on the bedrock surface by extending a near-vertical trench of lean concrete between the underside of footing depth and the bedrock surface where overburden is encountered at the design founding depth for footings.

Some bedrock removal is anticipated to be required to complete the basement level and/or site servicing work. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing significant amounts of organic or deleterious materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Consideration may be given for leaving in-situ fill in place at the subgrade level of paved areas provided it is reviewed in the field at the time of construction by Paterson personnel and subsequently proof-roller by a suitably-sized sheepsfoot roller. Proof-rolling should be completed under dry and above-freezing conditions and under the supervision of Paterson personnel.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming and/or controlled blasting will be required to remove sound bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming in conjunction with conventional excavation techniques, such as the use of a hydraulic excavator.

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by Paterson field personnel at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

Overbreak in Bedrock

Sedimentary bedrock formation, such as 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 and variable formation nature.

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. Where overbreak will be accommodated by leaving footings at the design founding elevation (i.e., not lowering or thickening to accommodate site conditions) 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 bedrock bearing surfaces be reviewed and approved by Paterson once the bedrock surface has attained the design founding elevation and should not be lowered to a deeper depth until reviewed and approved by Paterson field personnel at the time of construction.

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.

Vibration Considerations

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

Two parameters determine the recommended vibration limit: 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).

These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

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

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD (or as deemed appropriate by visual inspection of Paterson field staff experienced in assessing the compaction of soil fill). Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 50 mm and matching the gradation of an OPSS Granular B Type I or Type II crushed stone. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted in 300 mm thick loose lifts and compacted to a minimum of 98% of the materials SPMDD using a suitably sized smooth drum vibratory roller. Site-generated blast rock fill may be used for preparing the subgrade for the basement slab throughout the building footprint provided the fill is considered suitable by Paterson at the time of construction.

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 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 95% of its SPMDD.

Under winter conditions, if snow and ice is present within the blast rock or other imported fill placed below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized. Providing a heat source during winter construction may be recommended if compacted fill material is intended to be exposed for long periods of time.

5.3 Foundation Design

Bearing Resistance Values

Conventional spread footings placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **375 kPa**.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in dry conditions, prior to the placement of concrete for footings.

Footings placed on a clean, surface sounded limestone bedrock surface could be designed for a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

Lean Concrete Filled Trenches

Alternatively, where bedrock is not encountered at the design underside of footing elevation for footings where a bedrock bearing resistance value and bearing medium is sought as part of the foundation design, consideration may be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (minimum **15 MPa** 28-day compressive strength).

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying sound bedrock. The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Settlement

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above 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 value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

Seismic Shear Wave Velocity Testing

Shear wave velocity testing was completed to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012 and 2024.

The results of the shear wave velocity testing are provided in Figure 2 and Figure 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location is presented on Drawing PG7040-1 - 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 a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound 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 located 20, 3, and 2 m away from the first geophone, 20, 3, and 2 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods.

The interpretation is performed by recovering arrival times from direct 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 building foundations. 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 shear wave velocity due to the increasing quality of bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Determination of Vs_{30}

It is expected footings are placed directly or indirectly (i.e., using lean-concrete in-filled trenches) upon a clean, sounded bedrock surface. Based on our testing results, the bedrock shear wave velocity is **2,270 m/s**. Based on the above, the Vs_{30} was calculated considering the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\,m}{\left(\frac{30\,m}{2{,}270\,m/s}\right)}$$

$$V_{s30=}\ 2{,}270\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed building founded directly or indirectly upon a bedrock bearing surface is **2,270 m/s**. Therefore, a **Site Class A or Site Designation X_{2270}** is applicable for the proposed building, as per Table 4.1.8.4.A of the OBC 2012 and OBC 2024, respectively. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the in-situ soil and/or bedrock surfaces will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill should consist of OPSS Granular A. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes should be advised by Paterson during the design phase and once the footing and sump pit locations are known. The footprint would be confirmed at the time of construction once groundwater infiltration can be best assessed, if any. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions could 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.

A portion of the basement walls are expected to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m^3 (effective unit weight of 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. The seismic earth pressure is expected to be transferred to the underground floor slabs, which should be designed to accommodate the pressures.

Undrained conditions are anticipated (i.e., below the groundwater level). Therefore, the applicable effective unit weight of the retained soil should be 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight for the overburden.

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

Static 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 $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 0.32 g according to the OBC 2012 and 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 2012 and 2024.

5.7 Pavement Design

The recommended pavement structures for the subject site are shown in Table 5, Table 6 and Table 7.

Table 5 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 6 – Recommended Pavement Structure – Local Residential Roadways, Access Lanes and Heavy Truck Parking Areas

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

Table 7 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**32 MPa Concrete**
200	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-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 SPMDD with suitable vibratory equipment.

If consideration is given to providing a rigid pavement structure for the ramp portion of the underground parking garage, it is suggested Paterson review and advise on the potential associated pavement design, drainage and frost protection considerations once those details are known.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Foundation Drainage System

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure where double-sided pours will be undertaken. In areas where blind-sided pours will be considered, the perimeter drainage pipe should be placed along the interior side of the foundation wall and connected to sleeves placed within the foundation wall at a 6 m center-to-center spacing. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

It is anticipated that underfloor drainage will be required to provide an outlet for water captured by the buildings drainage system since it is assumed external gravity outlets will not be able to be accommodated by the sewer design. The layout of the perimeter and underfloor drainage systems should be determined by Paterson during the design phase once the foundation structure and sump pit locations are known. The perimeter drainage pipe would connect to a series of underfloor drainage lines which would direct water to sump pit(s) within the lower basement area.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts and pools located within the lowest basement level. A continuous PVC waterstop should be installed within the interface between the concrete base slab below the elevator shaft foundation walls. It is recommended that Paterson review and advise on all basement waterproofing/drainage system designs during the design phase.

Foundation Backfill

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 composite drainage system, such as CCW MiraDRAIN 2000 or Delta-Teraxx or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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 or current structural and architectural drawings for the proposed building to provide specific waterproofing and drainage design recommendations for design and tender. 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 of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) 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 Side Slopes

Unsupported Excavations

Excavation side slopes above the groundwater level extending to a maximum vertical height 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.

In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors or other means determined by Paterson at the time of construction.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by Paterson in order to detect if the slopes are exhibiting signs of distress.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 2 m centres.

Soil stockpiles, debris, and other forms of weight should not be considered for the purpose of securing the tarpaulins along the top of the slope. However, consideration may be given to restraining the tarpaulins with soil, sandbags, stone, etc. along the bottom of the side-slope. The tarpaulins should extend beyond the overburden and onto the bedrock surface.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A minimum of 1 m horizontal ledge should remain between the unsupported excavation and bedrock surface.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system, or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of 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 included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Given the sandy nature of the soils present throughout the subject site, the designer should consider provisions to mitigate the potential for excessive losses of retained soil during the lagging installation process if consideration is given to using a soldier pile and lagging system. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The 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

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

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

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

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes.

The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A crushed stone. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 99% of the SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that surface water infiltration into the excavations undertaken above the groundwater table should be manageable through the sides of the excavation and controllable using open sumps. It is further anticipated that groundwater infiltration into the excavations may be moderate to high throughout the overburden located below the groundwater table and/or bedrock surface. Further, bedrock removal can lead to increased fracturing (hydraulic pathways), resulting in highly variable groundwater conditions. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

It is recommended that Paterson review detailed design drawings and construction timelines associated with excavation works prior to tendering the earthworks portion of the project. It is recommended that Paterson review and advise at that time if additional recommendations are required with regards to planning temporary dewatering and groundwater management efforts during the construction phase.

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.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In particular, where a shoring system is constructed, 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 documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice is present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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 (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to severely aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Based on our review, the proposed will be founded on non-cohesive soils and/or bedrock. Since the structures are not anticipated to be founded upon silty clay soils, tree planting restrictions as based on the City of Ottawa *Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)* are not considered applicable for the subject site from a geotechnical perspective.

7.0 Recommendations

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

- Review grading, servicing and structural plan(s) from a geotechnical perspective.
- Review of architectural plans pertaining to the buildings foundation drainage and/or waterproofing system and associated drainage systems.

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

- Observation of all bearing surfaces prior to the placement of concrete.
- Observation of all waterproofing membranes, sub-slab drainage system and all associated systems and assemblies.
- 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 undertaken by Paterson.

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 Seymour Pacific Developments (Ontario) Ltd. 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.

N Versolato

Nicholas F. R. Versolato, CPI, B. Eng.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
February 6, 2025
D. J. PETAHTEGOOSE
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PROVINCE OF ONTARIO

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

ANALYTICAL TESTING RESULTS

MONITORING WELL WATER ELEVATION PLOTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349145.37 **NORTHING:** 5016613.415 **ELEVATION:** 107.79

DATUM: Geodetic

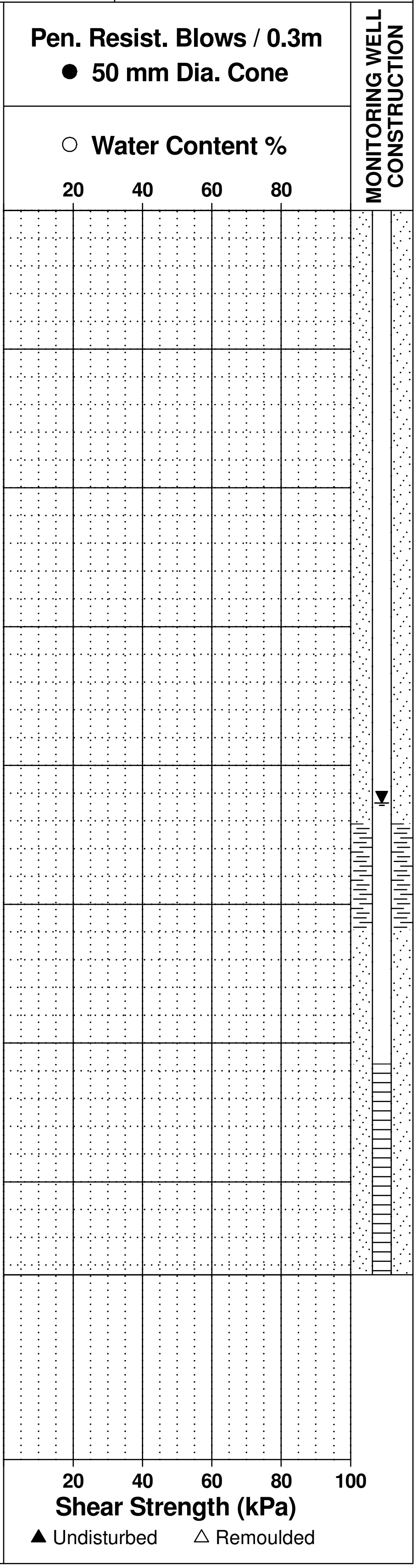
REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 25, 2024

FILE NO. PG7040

HOLE NO. BH 1-24

SAMPLE DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	107.79
FILL: Brown silty sand with gravel, cobbles, boulders and organics	0.10		AU	1				
FILL: Very stiff, brown silty clay, trace gravel, sand and organics - gravel content increases with depth	1.22		SS	2	27	+50	1	106.79
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders			SS	3	13	17	2	105.79
			SS	4	89	+50		
			SS	5	42	30	3	104.79
	4.45		SS	6	46	+50	4	103.79
BEDROCK: Excellent quality, limestone bedrock			RC	1	100	91	5	102.79
							6	101.79
- vertical seams from 7.0m to 7.5m depth	7.67		RC	2	100	91	7	100.79
End of Borehole (GWL @ 4.27m - April 1, 2024)								



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349095.799 **NORTHING:** 5016673.602 **ELEVATION:** 107.78

DATUM: Geodetic

REMARKS:

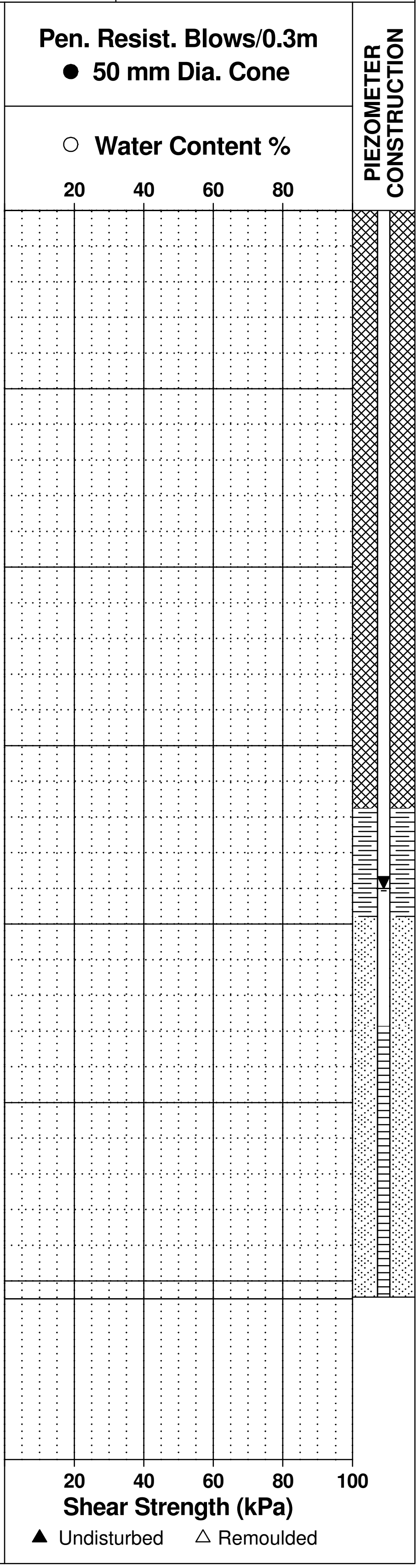
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 25, 2024

FILE NO. PG7040

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown silty sand with topsoil and organics, some clay, gravel and cobbles		AU	1		
0.61					
FILL: Firm, brown silty clay, some gravel, trace topsoil		SS	2	58	4
1.68					
GLACIAL TILL: Compact to very dense, brown silty sand to sandy silt, some gravel, boulders, occasional cobbles, trace clay		SS	3	75	16
		SS	4	100	+50
2.79					
BEDROCK: Fair to good quality, limestone bedrock - good quality by 3.1m depth		RC	1	100	45
		RC	2	100	76
		RC	3	100	80
6.10					
End of Borehole (GWL @ 3.80m - April 1, 2024)					

DEPTH (m)	ELEV. (m)
0	107.78
1	106.78
2	105.78
3	104.78
4	103.78
5	102.78
6	101.78



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

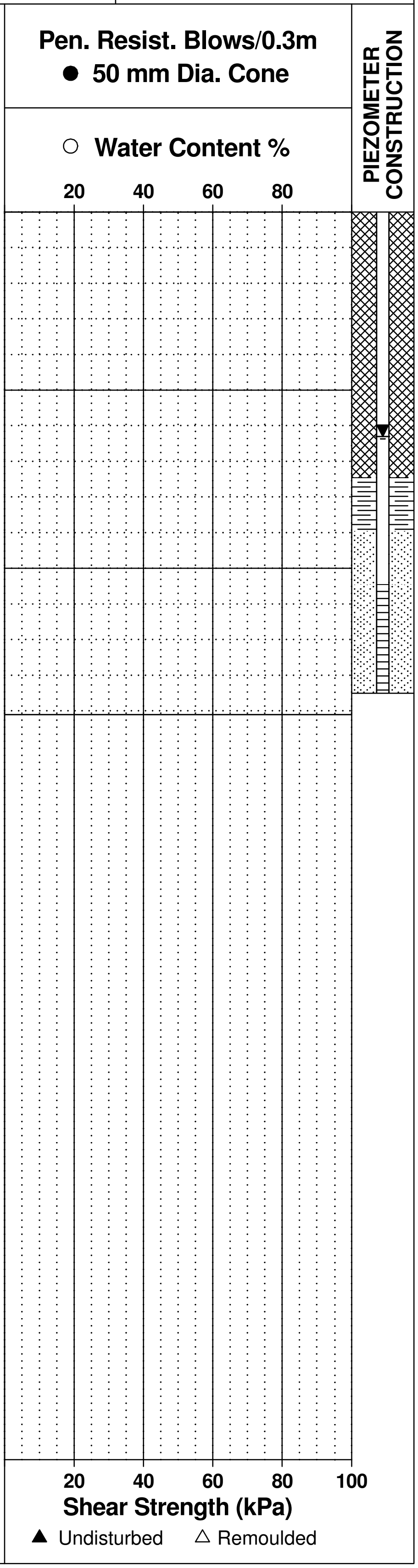
EASTING: 349148.741 **NORTHING:** 5016670.638 **ELEVATION:** 107.75
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 25, 2024

FILE NO. PG7040
HOLE NO. BH 3-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	107.75
FILL: Brown silty clay wih organics, gravel, cobbles and debris (0.30)	▨	AU	1				
FILL: Firm, brown silty clay, trace organics, wood and sand	▨	SS	2	67	8	1	106.75
1.45							
FILL: Stiff, brown silty clay, trace gravel	▨	SS	3	83	12	2	105.75
2.21							
GLACIAL TILL: Very dense, brown silty sand to sandy silt with gravel, boulders, occasional cobbles, trace clay		SS	4	67	+50		
2.82							
End of Borehole							

Practical refusal to augering at 2.82m depth

(GWL @ 1.26m - April 1, 2024)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349149.925 **NORTHING:** 5016648.712 **ELEVATION:** 107.85
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 25, 2024

FILE NO. PG7040
HOLE NO. BH 4-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone ○ Water Content %	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	107.85		
FILL: Brown silty clay with organics, some gravel, cobbles and sand		AU	1						
0.61									
FILL: Firm, grey silty clay to clayey silt, trace topsoil, gravel, wood and organics						1	106.85		
		SS	2	50	7				
		SS	3	8	10				
						2	105.85		
2.44									
GLACIAL TILL: Very dense to compact, brown silty sand to sandy silt with gravel, boulders, occasional cobbles		SS	4	100	+50				
						3	104.85		
		SS	5	42	48				
						4	103.85		
		SS	6	25	18				
		SS	7	82	37				
						5	102.85		
5.18									
End of Borehole									

Water Content %: 20 40 60 80

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349147.597 **NORTHING:** 5016648.386 **ELEVATION:** 107.77

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 26, 2024

FILE NO. PG7040

HOLE NO. BH 4A-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	MONITORING WELL CONSTRUCTION
GROUND SURFACE									
OVERBURDEN						0	107.77		
						1	106.77		
						2	105.77		
3.15						3	104.77		
End of Borehole Practical refusal to augering at 3.15m depth									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349149.925 **NORTHING:** 5016648.712 **ELEVATION:** 107.85
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 26, 2024

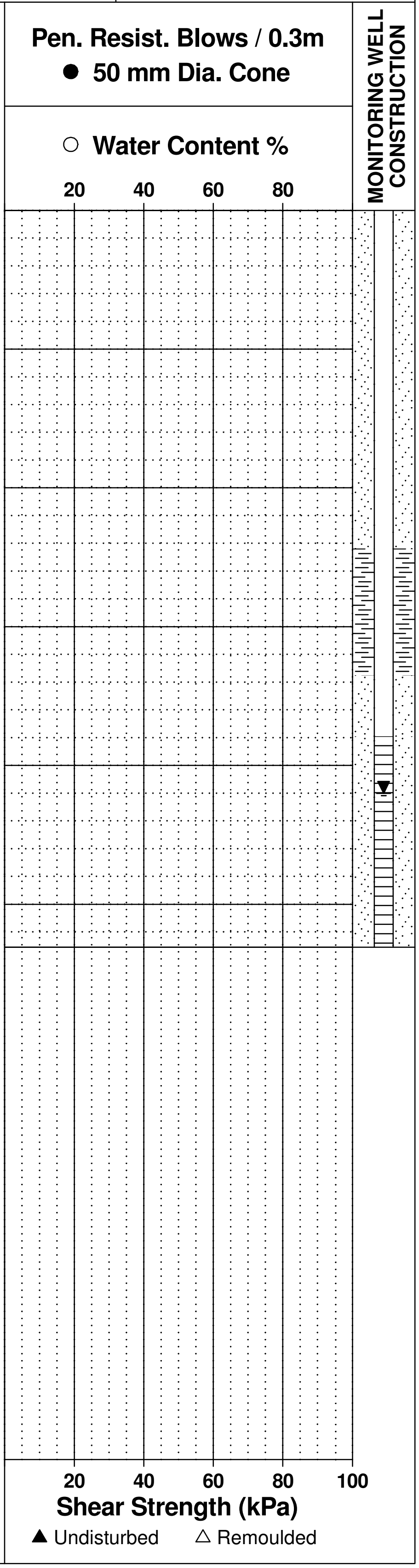
FILE NO. PG7040
HOLE NO. BH 4B-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
OVERBURDEN						0	107.85
						1	106.85
						2	105.85
						3	104.85
						4	103.85
						5	102.85
5.31							

End of Borehole

Practical refusal to augering at 5.31m depth

(GWL @ 4.20m - April 1, 2024)



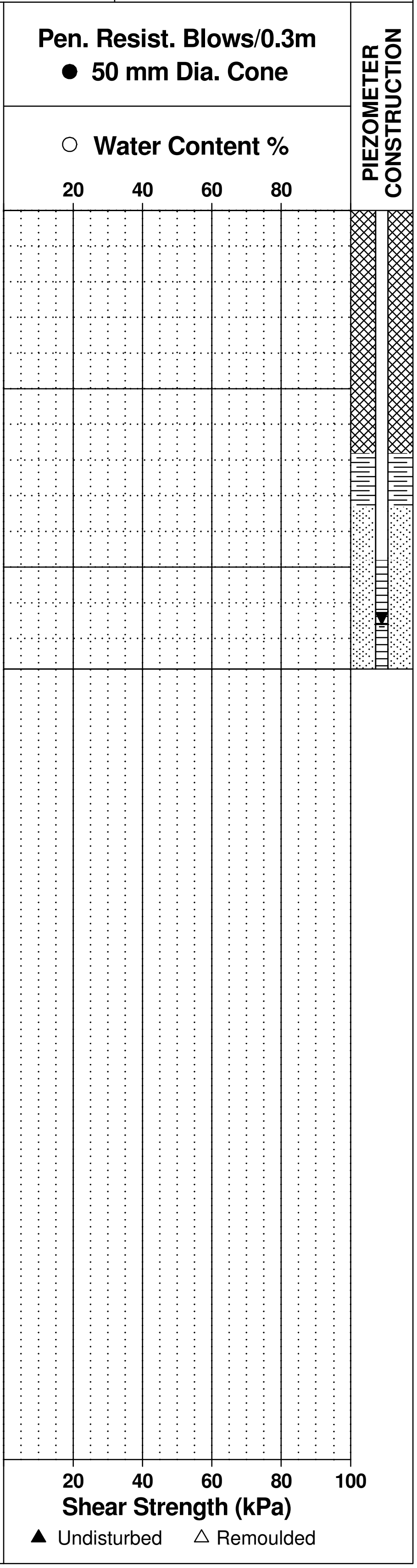
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349120.339 **NORTHING:** 5016659.338 **ELEVATION:** 107.59
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 25, 2024

FILE NO. PG7040
HOLE NO. BH 5-24

SAMPLE DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	107.59
FILL: Brown silty clay with organics, gravel, cobbles, debris	0.08						
FILL: Firm, brown silty clay, trace gravel		AU	1				
		SS	2	75	4	1	106.59
		SS	3	50	3	2	105.59
	2.21						
GLACIAL TILL: Very dense, brown silty clay to clayey silt with sand and shale fragments	2.57	SS	4	60	+50		
End of Borehole Practical refusal to augering at 2.57m depth (GWL @ 2.32m - April 1, 2024)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349121.373 **NORTHING:** 5016699.77 **ELEVATION:** 108.02

DATUM: Geodetic

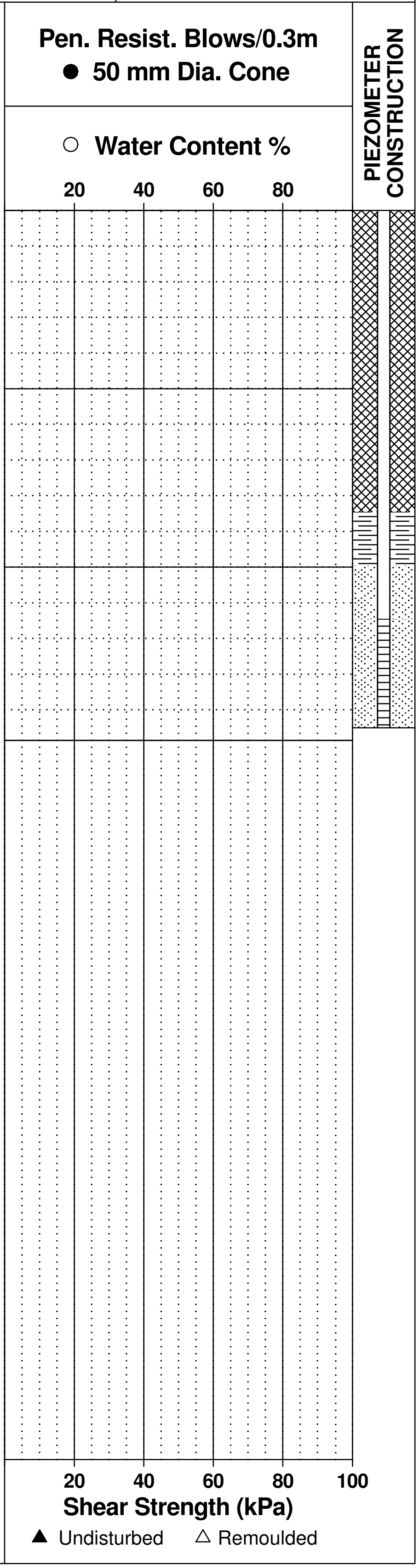
REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 26, 2024

FILE NO. PG7040

HOLE NO. BH 6-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	108.02
FILL: Compact, brown silty sand to sandy silt, some gravel, cobbles, organics, trace clay		AU	1				
- clay content increases with depth		SS	2	58	12	1	107.02
- trace topsoil by 1.52m depth		SS	3	58	10	2	106.02
		SS	4	33	8		
2.97							
End of Borehole Practical refusal to augering at 2.97m depth (Borehole Dry - April 1, 2024)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349177.081 **NORTHING:** 5016637.846 **ELEVATION:** 108.04

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 26, 2024

FILE NO. PG7040

HOLE NO. BH 7-24

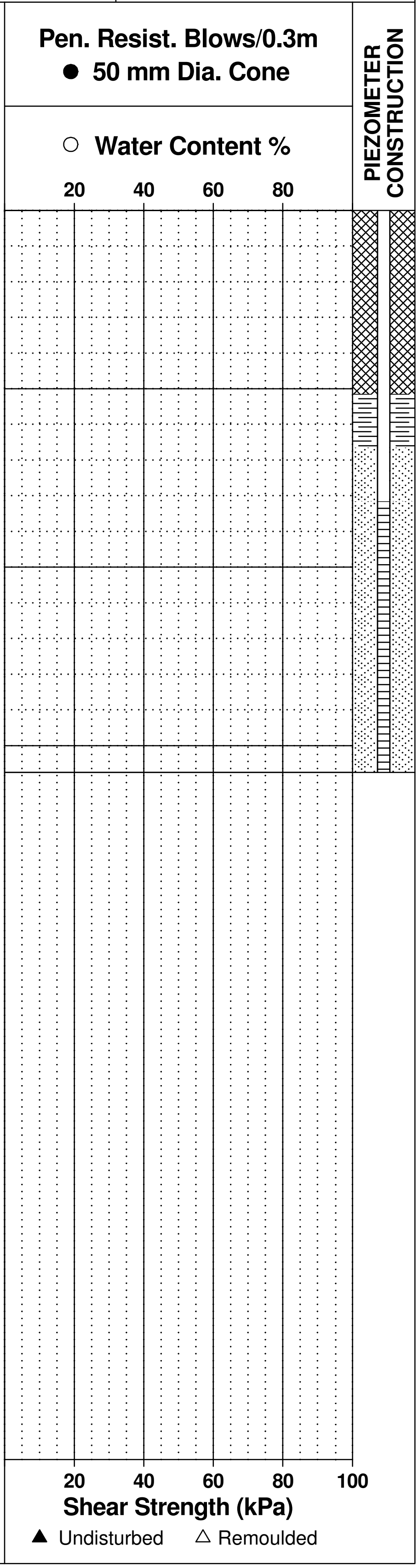
SAMPLE DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Compact, brown silty sand to sandy silt with gravel, some clay and organics	0.61	AU	1		
FILL: Very stiff, brown silty clay to clayey silt with gravel, some topsoil, occasional cobbles	1.45	SS	2	58	25
FILL: Stiff, grey silty clay to clayey silt, some gravel and topsoil		SS	3	42	8
- trace sand by 2.3m depth	2.97	SS	4	42	6
GLACIAL TILL: Very dense, brown silty sand to sandy silt, trace gravel, occasional cobbles	3.15	SS	5	100	+50

End of Borehole

Practical refusal to augering at 3.15m depth

(Borehole Dry - April 1, 2024)

DEPTH (m)	ELEV. (m)
0	108.04
1	107.04
2	106.04
3	105.04



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349182.005 **NORTHING:** 5016646.561 **ELEVATION:** 108.03
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
Ground Surface						0	108.03		
FILL: Compact brown silty sand to sandy silt with gravel, cobbles and boulders 0.70	▨	G	1						
FILL: Very stiff brown silty clay to clayey silt with gravel, cobbles and boulders	▨	G	2						
						1	107.03		
1.20 **FILL:** Very stiff brown to grey silty clay to clayey silt with gravel, cobbles and boulders, trace topsoil	▨	G	3						
	▨	G	4			2	106.03		
	▨	G	5						
3.10						3	105.03		
GLACIAL TILL: Very dense brown silty sand to sandy silt with gravel, cobbles and boulders	^^^	G	6						
4.00	^^^	G	7			4	104.03		
End of Test Pit Practical Refusal @ 4.0 m on bedrock									

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349135.715 **NORTHING:** 5016706.052 **ELEVATION:** 108.05

DATUM: Geodetic

REMARKS:

BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040

HOLE NO. TP 2-24

SAMPLE DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	PIEZOMETER CONSTRUCTION
Ground Surface							0	108.05		
FILL: Compact brown silty sand to sandy silt, trace topsoil	0.20		G	1						
FILL: Stiff brown silty clay to clayey silt with sand, gravel, cobbles and blast rock			G	2						
							1	107.05		
	1.20									
FILL: Stiff dark brown silty clay with gravel, cobbles and blast rock, trace wood and topsoil			G	3						
			G	4			2	106.05		
			G	5						
	2.90									
GLACIAL TILL: Very dense brown silty sand to sandy silt with gravel, cobbles and boulders	3.05		G	6			3	105.05		
End of Test Pit Practical refusal @ 3.05 m on bedrock										

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349106.993 **NORTHING:** 5016686.169 **ELEVATION:** 107.92
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 3-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	PIEZOMETER CONSTRUCTION
Ground Surface						0	107.92		
FILL: Compact brown silty clay to clayey silt with gravel, cobbles and boulders, trace clay and topsoil 0.40		G	1						
FILL: Stiff brown to grey silty clay with gravel, cobbles and boulders, trace topsoil, sand, concrete and bricks		G	2						
						1	106.92		
		G	3						
		G	4						
						2	105.92		
		G	5						
2.80									
GLACIAL TILL: Very dense brown silty sand to sandy silty with gravel, cobbles and boulders 2.95		G	6						
End of Test Pit Practical refusal @ 2.95 m on bedrock									

20 40 60 80

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349156.3 **NORTHING:** 5016681.709 **ELEVATION:** 107.92
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 4-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	107.92
FILL: Stiff brown silty clay with sand, gravel, cobbles and blast rock, trace topsoil	▨	G	1				
	▨	G	2				
	▨					1	106.92
	▨	G	3				
	▨	G	4				
- Dark brown @ 2.0 m	▨					2	105.92
	▨	G	5				
2.90							
End of Test Pit Practical refusal @ 2.9 m on bedrock							

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

○ **Water Content %**
20 40 60 80

PIEZOMETER CONSTRUCTION

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349131.734 **NORTHING:** 5016672.68 **ELEVATION:** 107.65
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 5-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	107.65
FILL: Stiff brown silty clay with sand, gravel and cobbles, trace topsoil	▨	G	1				
	▨	G	2				
0.70							
FILL: Compact brown silty sand to sandy silt with gravel and cobbles, trace topsoil	▨					1	106.65
	▨	G	3				
	▨					2	105.65
	▨	G	4				
2.85							
End of Test Pit Practical refusal @ 2.85 m on bedrock							

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

○ Water Content %
20 40 60 80

PIEZOMETER CONSTRUCTION

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349099.4 **NORTHING:** 5016653.18 **ELEVATION:** 107.51

DATUM: Geodetic

REMARKS:

BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040

HOLE NO. TP 6-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	107.51
FILL: Stiff brown silty clay with gravel, cobbles and blast rock, trace sand and topsoil		G	1				
		G	2				
						1	106.51
		G	3				
1.60							
TOPSOIL 1.62							
GLACIAL TILL: Compact to dense reddish brown silty sand to sandy silt with gravel, cobbles and boulders		G	4				
		G	5				
						2	105.51
- Light brown by 1.95 m 2.20							
End of Test Pit Practical refusal @ 2.2 m on bedrock							

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

○ Water Content %

20 40 60 80

PIEZOMETER CONSTRUCTION

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

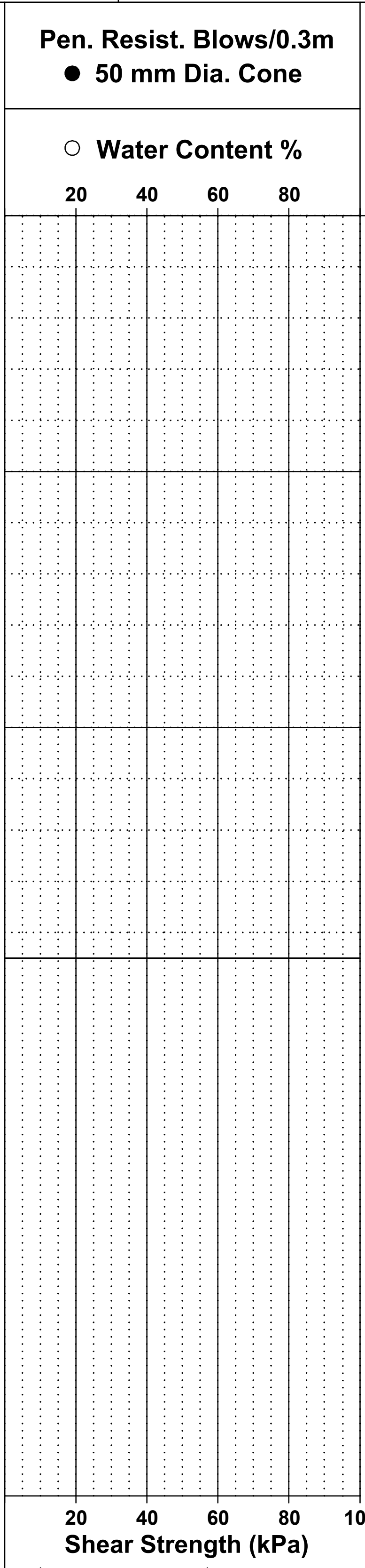
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349177.08 **NORTHING:** 5016637.85 **ELEVATION:** 107.94
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 7-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
Ground Surface						0	107.94	
FILL: Compact brown silty sand to sandy silt with gravel and cobbles, trace clay	▨	G	1					
0.50								
FILL: Very stiff brown to grey silty clay to clayey silt with sand, gravel, cobbles, trace topsoil	▨	G G	2 3			1	106.94	
1.95								
TOPSOIL and organics	■	G	4			2	105.94	
2.25								
GLACIAL TILL: Dense brown silty sand to sandy silt with gravel and cobbles	^	G	5					
2.90								
End of Test Pit Practical refusal @ 2.9 m on bedrock								



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349127.253 **NORTHING:** 5016621.445 **ELEVATION:** 107.58
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 8-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	107.58
FILL: Stiff brown silty clay with sand, gravel, cobbles and blast rock	▨	G	1				
	▨	G	2				
1.10						1	106.58
GLACIAL TILL: Dense brown silty sand to sandy silt with gravel, cobbles and boulders, trace clay	^^^	G	3				
						2	105.58
- Light brown by 2.2 m, no clay	^^^	G	4				
	^^^	G	5				
2.80							
End of Test Pit Practical refusal @ 2.8 m on bedrock							

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

PIEZOMETER CONSTRUCTION

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

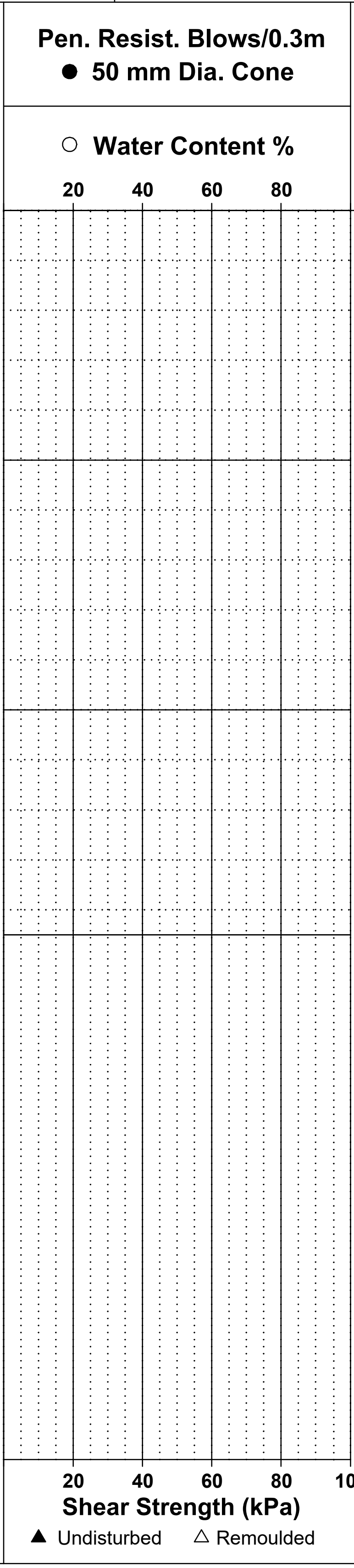
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Bldg., 425 Culdaff Road
Ottawa, Ontario

EASTING: 349110.78 **NORTHING:** 5016624.138 **ELEVATION:** 107.80
DATUM: Geodetic
REMARKS:
BORINGS BY: Excavator **DATE:** May 2, 2024

FILE NO. PG7040
HOLE NO. TP 9-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	107.80
FILL: Compact brown silty sand to sandy silt with gravel, trace clay		G	1				
0.40							
TOPSOIL							
0.50							
FILL: Stiff brown silty clay with gravel, cobbles and blast rock, trace sand		G	2			1	106.80
1.65							
FILL: Loose brown silty sand with topsoil, trace gravel		G	3			2	105.80
2.15							
GLACIAL TILL: Dense brown silty sand to sandy silt with gravel, cobbles and boulders - Light brown @ 2.5 m		G G	4 5				
2.90							
End of Test Pit Practical refusal @ 2.9 m on bedrock							



PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Development - Palladium Dr. & Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Cavanaugh Construction.

REMARKS

BORINGS BY Hydraulic Excavator

DATE March 9, 2018

FILE NO. PG3520

HOLE NO. TP 1-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	106.18		
TOPSOIL 0.20	■								
Compact, brown **SILTY FINE SAND to SANDY SILT,** trace clay and gravel 1.00		G	1			1	105.18		
GLACIAL TILL: Compact, brown silty sand, some clay, gravel, cobbles and boulders 1.50	^^^	G	2						
End of Test Pit									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

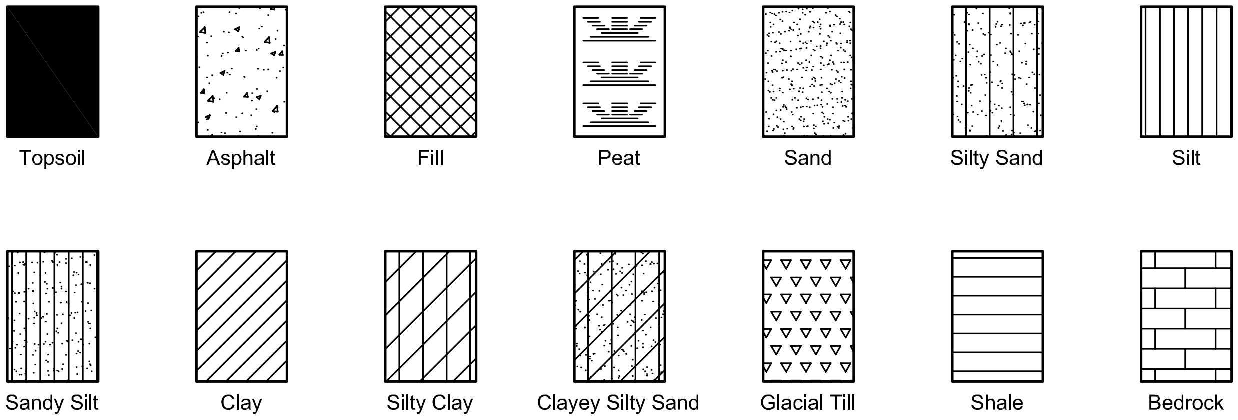
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

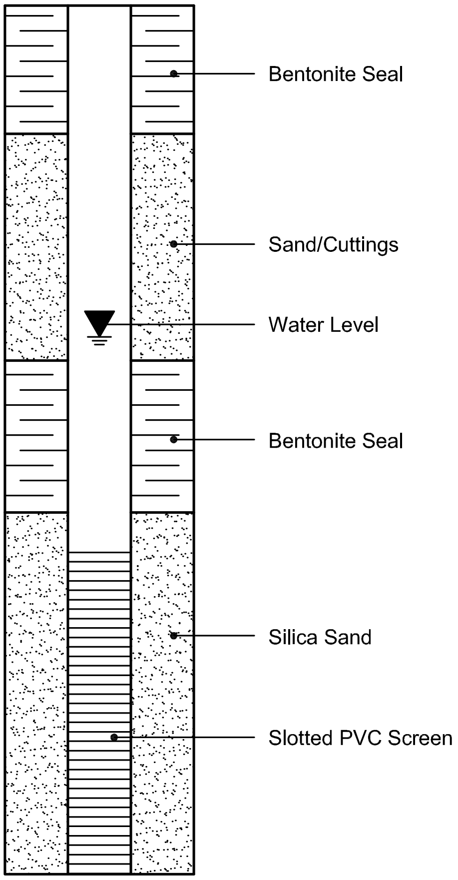
SYMBOLS AND TERMS (continued)

STRATA PLOT

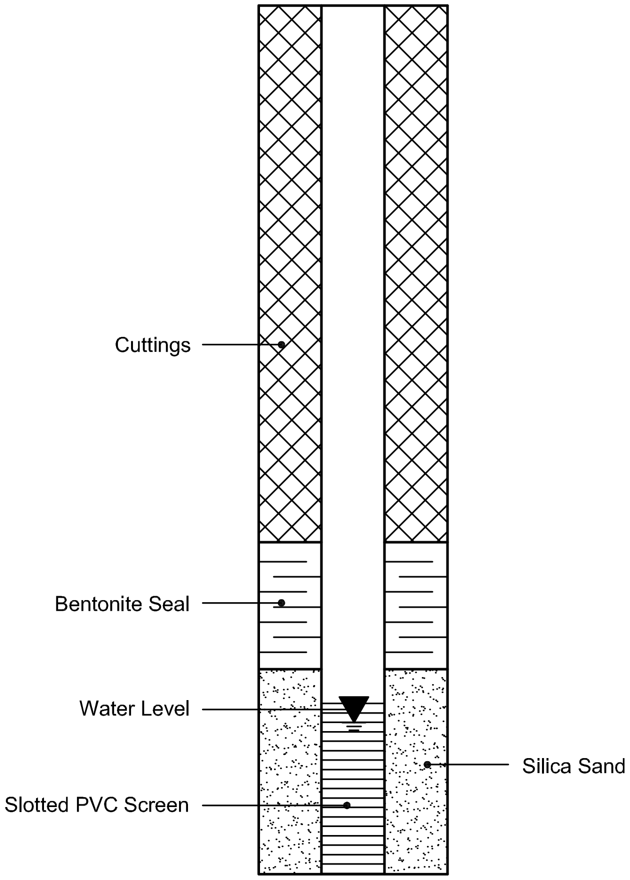


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PATERSON
GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**Seymour Pacific Developments**	FILE No.:	**PG7040**
PROJECT:	Lab Testing - Grey Limestone	REPORT No.:	1
SITE ADDRESS:	**425 Culdaff Rd.**	DATE REPT'D:	10-May-24
STRUCTURE TYPE & LOCATION:	BH2-24 / 12' - 12'5"		

SAMPLE INFORAMTION			
LAB NO.:	**51885**		
SAMPLE NO.:	RC2		
LOCATION:	12' - 12'5"		
SAMPLE DATES			
DATE CAST	28-Mar-24		
DATE CORED	-		
DATE RECEIVED	9-May-24		
DATE TESTED	10-May-24		
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00		
HEIGHT (mm)	82.00		
WEIGHT (g)	380		
AREA (mm^2)	1735		
VOLUME (cm^3)	142		
UNIT WEIGHT (kg/m^3)	**2671**		
TEST RESULTS			
H / D RATIO	1.74		
CORRECTION FACTOR	0.979		
LOAD (lbs)	17800		
GROSS Mpa	45.6		
MPa CORRECTED	**44.7**		
FORM OF BREAK	**Type A**	**Type A**	**Type A**
DIRECTION OF LOADING	PARALLEL	PARALLEL	PARALLEL
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		
REMARKS			

TECHNICAL PERSONNEL

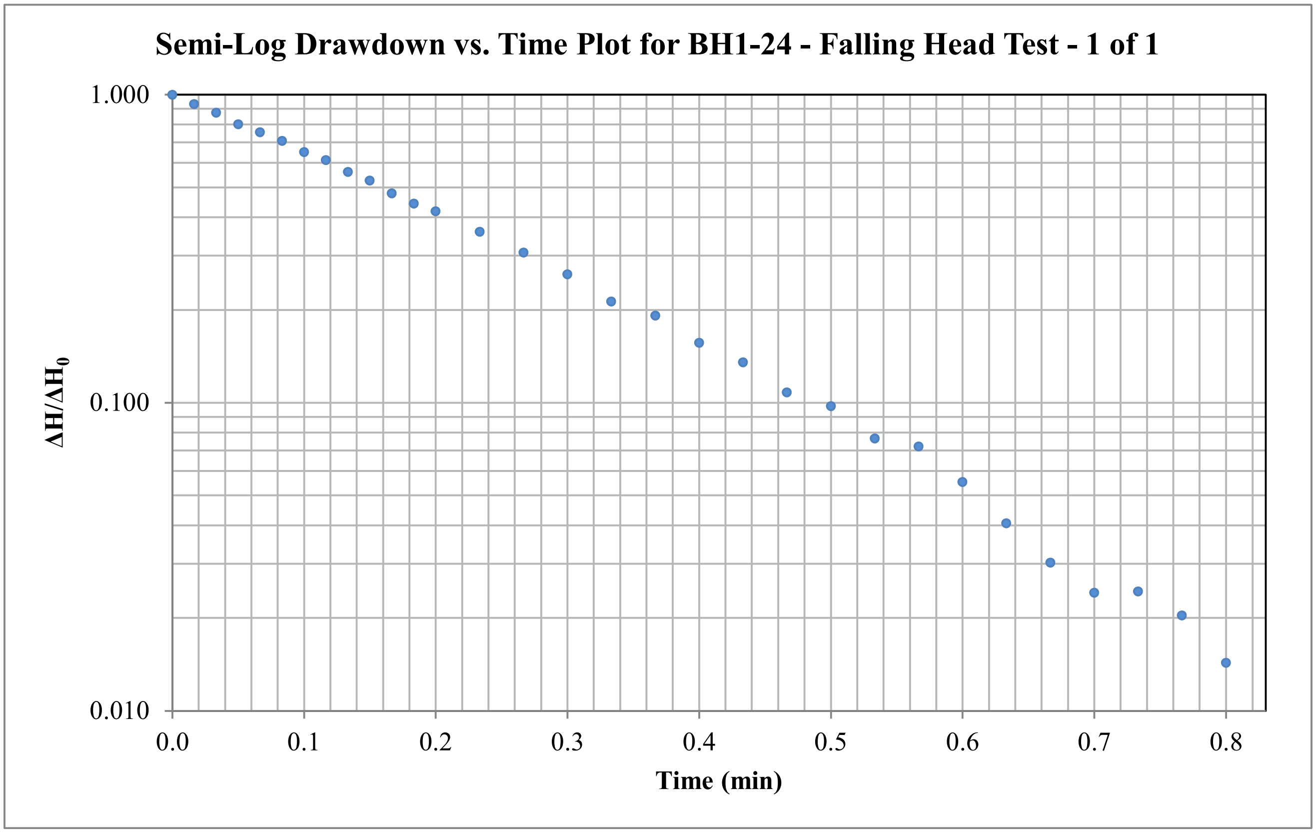
TECHNICIAN:	**VERIFIED BY:**	**C. Beadow**	**APPROVED BY:**	**Joe Forsyth, P. Eng.**

CERTIFIED LAB

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

Hvorslev Hydraulic Conductivity Analysis

Project: Seymour Pacific Developments (Ontario) Ltd. - 425 Culdaff Road
Test Location: BH1-24
Test: Falling Head - 1 of 1
Date: April 2, 2024


Semi-Log Drawdown vs. Time Plot for BH1-24 - Falling Head Test - 1 of 1

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):

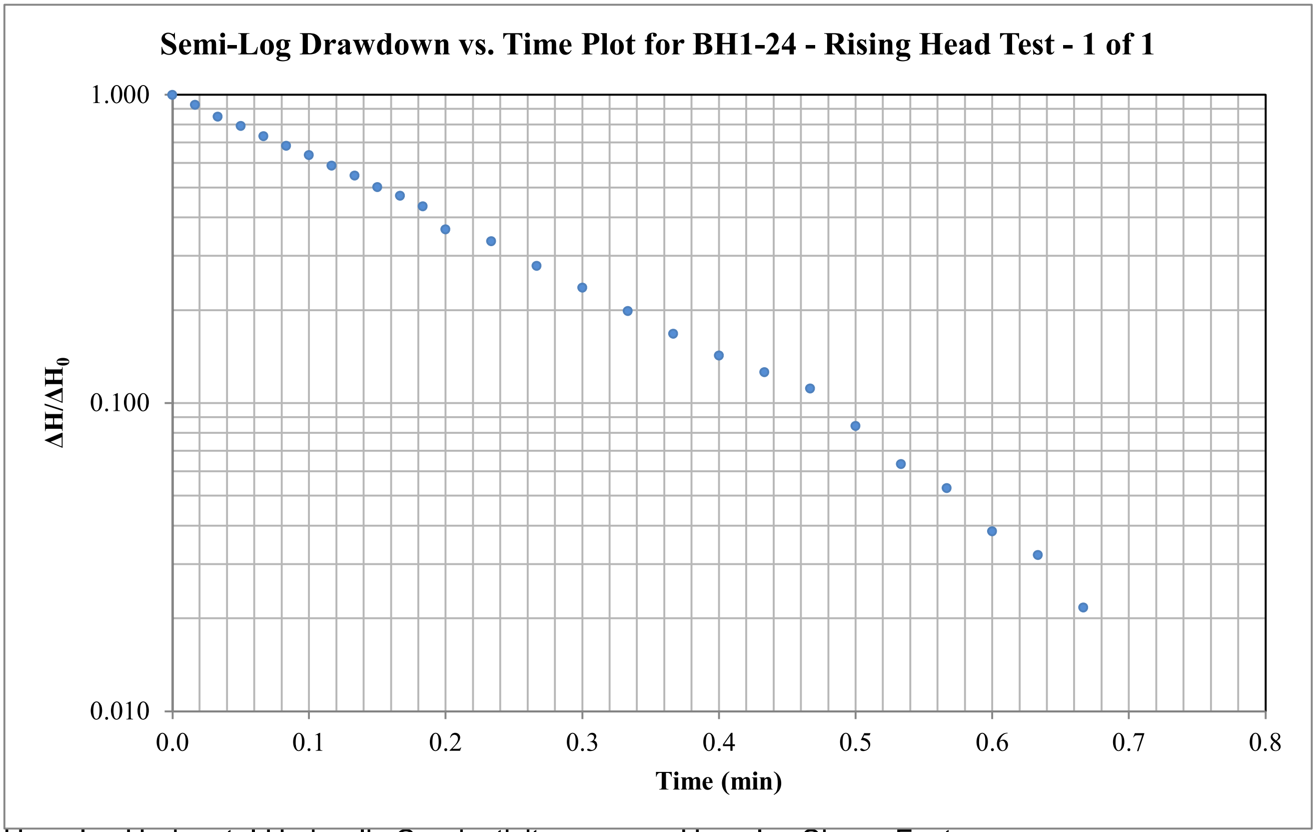
t*: 0.223 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.84E-05 m/sec

File: PG7040

Hvorslev Hydraulic Conductivity Analysis

Project: Seymour Pacific Developments (Ontario) Ltd. - 425 Culdaff Road
Test Location: BH1-24
Test: Rising Head - 1 of 1
Date: April 2, 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

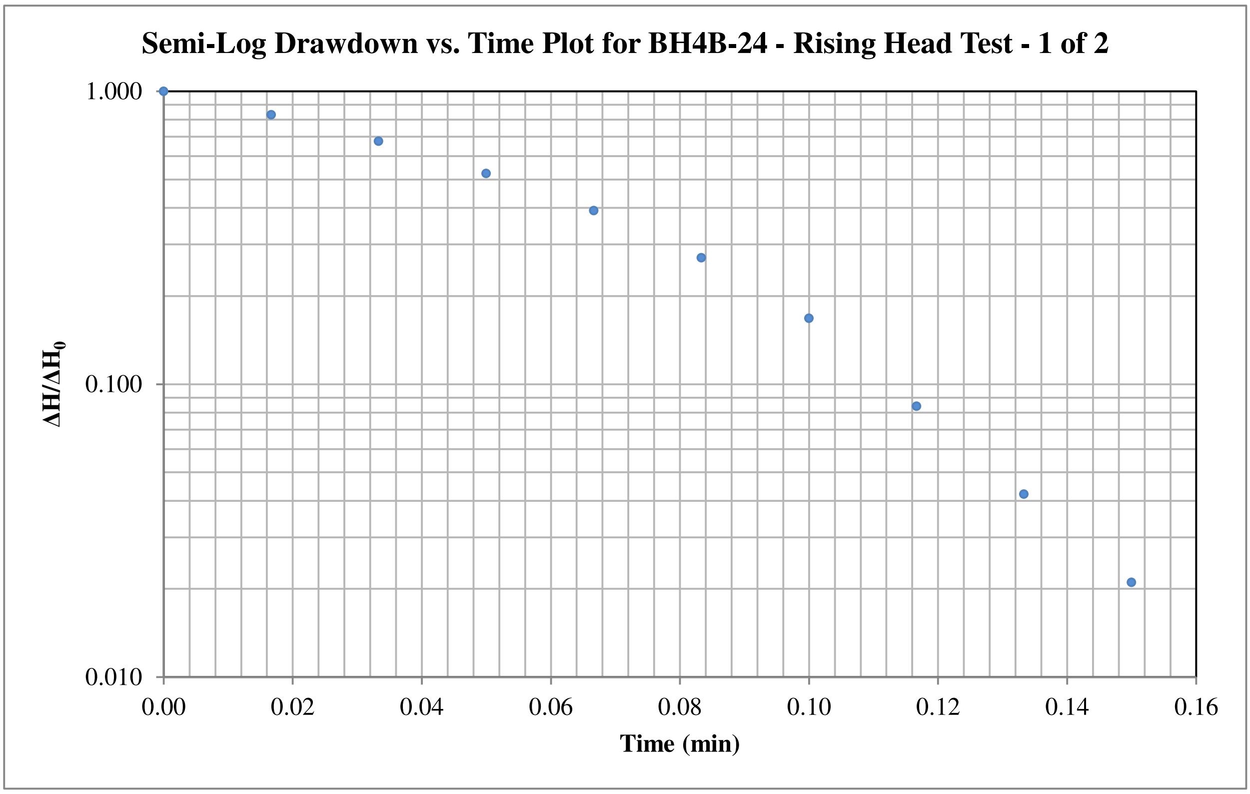
Data Points (from plot):

t*: 0.199 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.18E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Seymour Pacific Developments (Ontario) Ltd. - 425 Culdaff Road
Test Location: BH4B-24
Test: Rising Head - 1 of 2
Date: April 2, 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: 2.07557

Well Parameters:

L	1.3 m	Saturated length of screen or open hole
D	0.0508 m	Diameter of well
r_c	0.0254 m	Radius of well

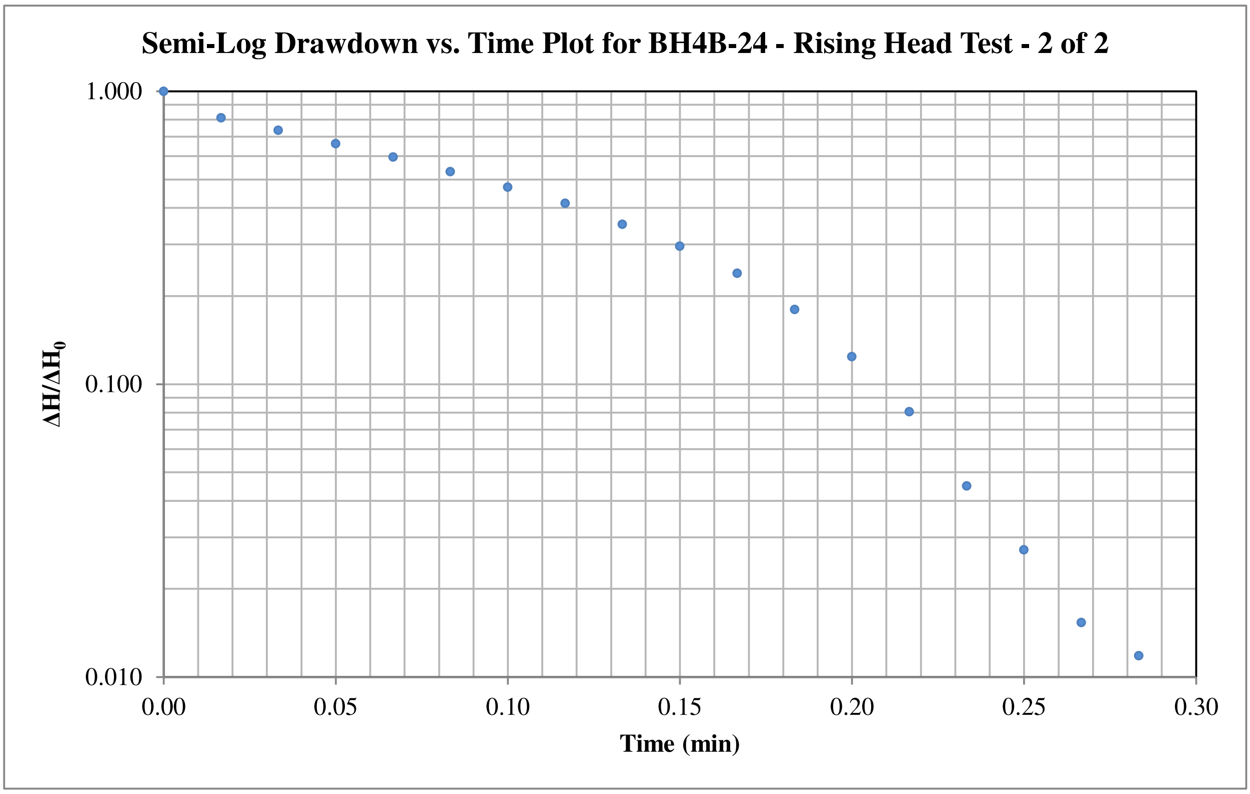
Data Points (from plot):

t*: 0.070 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.33E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Seymour Pacific Developments (Ontario) Ltd. - 425 Culdaff Road
Test Location: BH4B-24
Test: Rising Head - 2 of 2
Date: April 2, 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: 2.07557

Well Parameters:

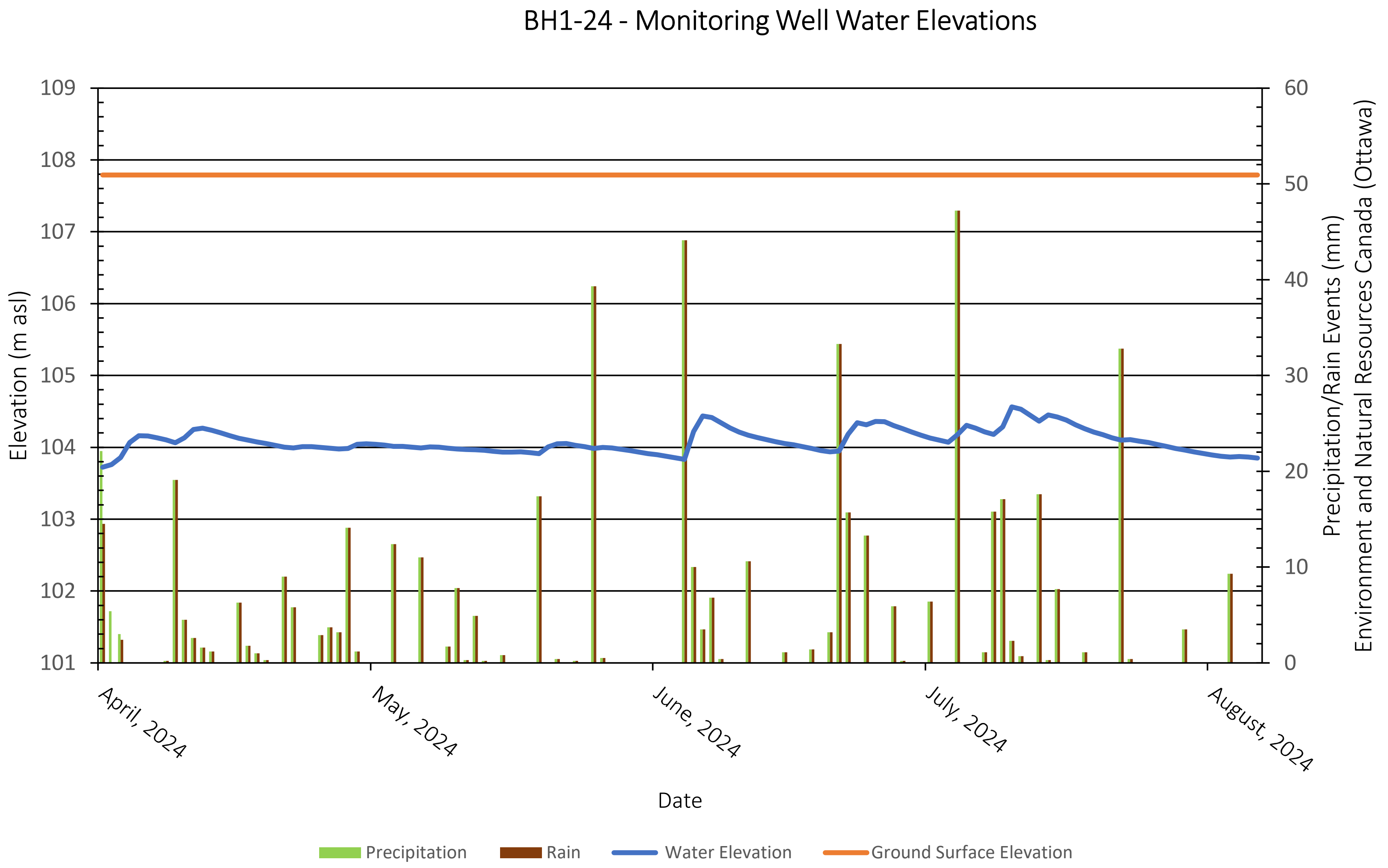
L	1.3 m	Saturated length of screen or open hole
D	0.0508 m	Diameter of well
r_c	0.0254 m	Radius of well

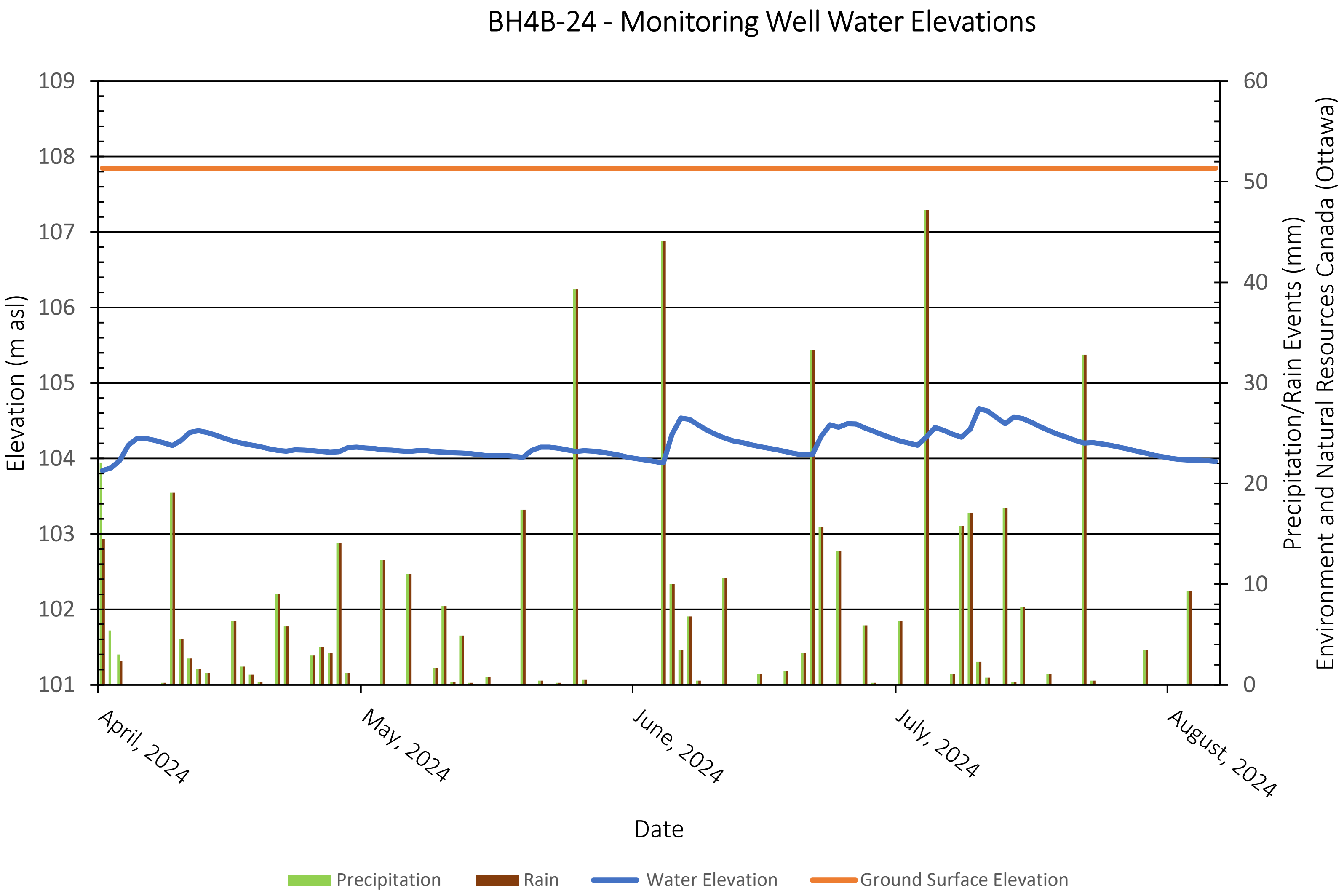
Data Points (from plot):

t*: 0.129 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.26E-04 m/sec

Client ID:		BH4 - 24 - SS7	-	-	-		
Sample Date:		26-Mar-24 09:00	-	-	-	-	-
Sample ID:		2413325-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	87.7	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.45	-	-	-	-	-
Resistivity	0.1 Ohm.m	19.5	-	-	-	-	-
Anions							
Chloride	10 ug/g	16	-	-	-	-	-
Sulphate	10 ug/g	368	-	-	-	-	-


BH1-24 - Monitoring Well Water Elevations
Elevation (m asl)
109
108
107
106
105
104
103
102
101
Precipitation/Rain Events (mm)
Environment and Natural Resources Canada (Ottawa)
60
50
40
30
20
10
0
April, 2024
May, 2024
June, 2024
July, 2024
August, 2024
Date
Precipitation
Rain
Water Elevation
Ground Surface Elevation


BH4B-24 - Monitoring Well Water Elevations
109
108
107
106
105
104
103
102
101
Elevation (m asl)
60
50
40
30
20
10
0
Precipitation/Rain Events (mm)
Environment and Natural Resources Canada (Ottawa)
April, 2024
May, 2024
June, 2024
July, 2024
August, 2024
Date
Precipitation
Rain
Water Elevation
Ground Surface Elevation

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG7040-1 – TEST HOLE LOCATION PLAN

DRAWING PG7040-2 – BEDROCK CONTOUR PLAN

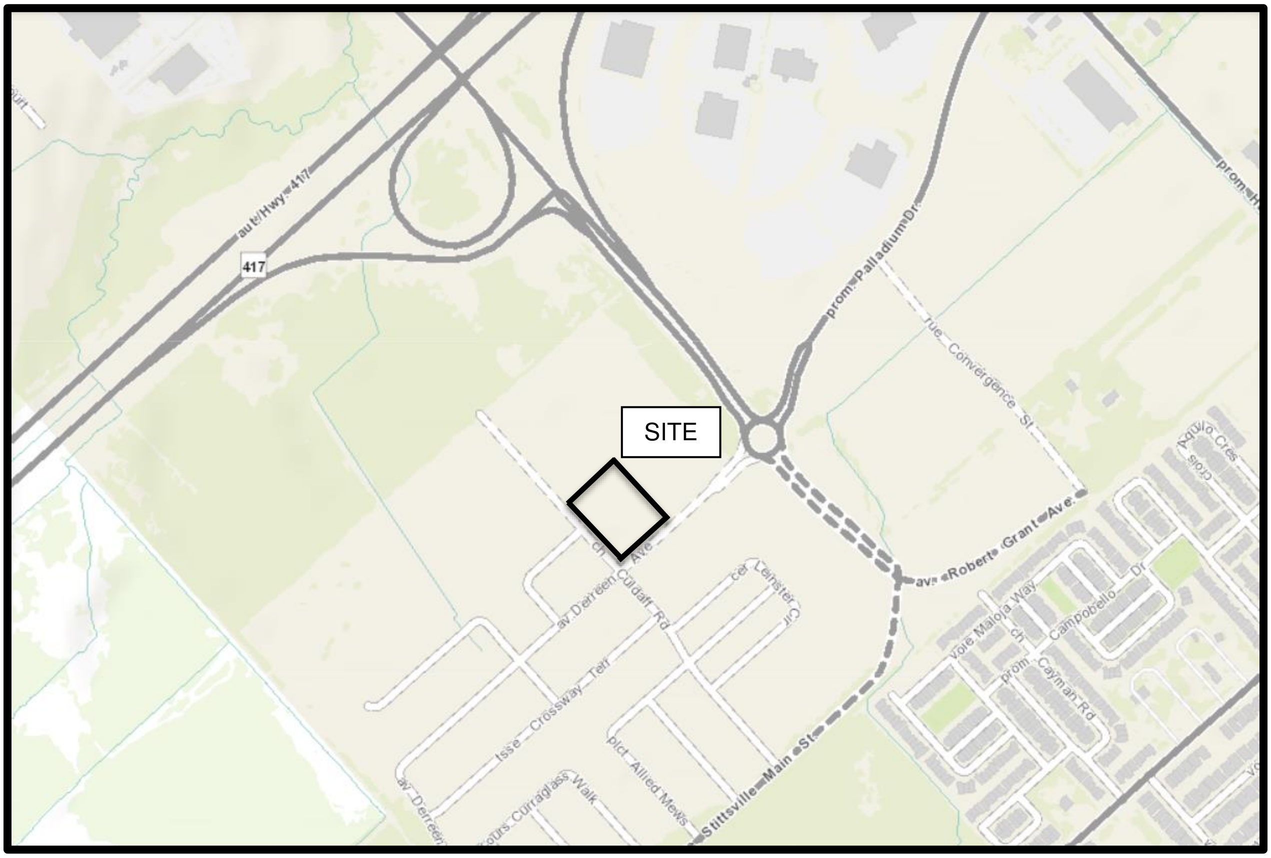


FIGURE 1

KEY PLAN

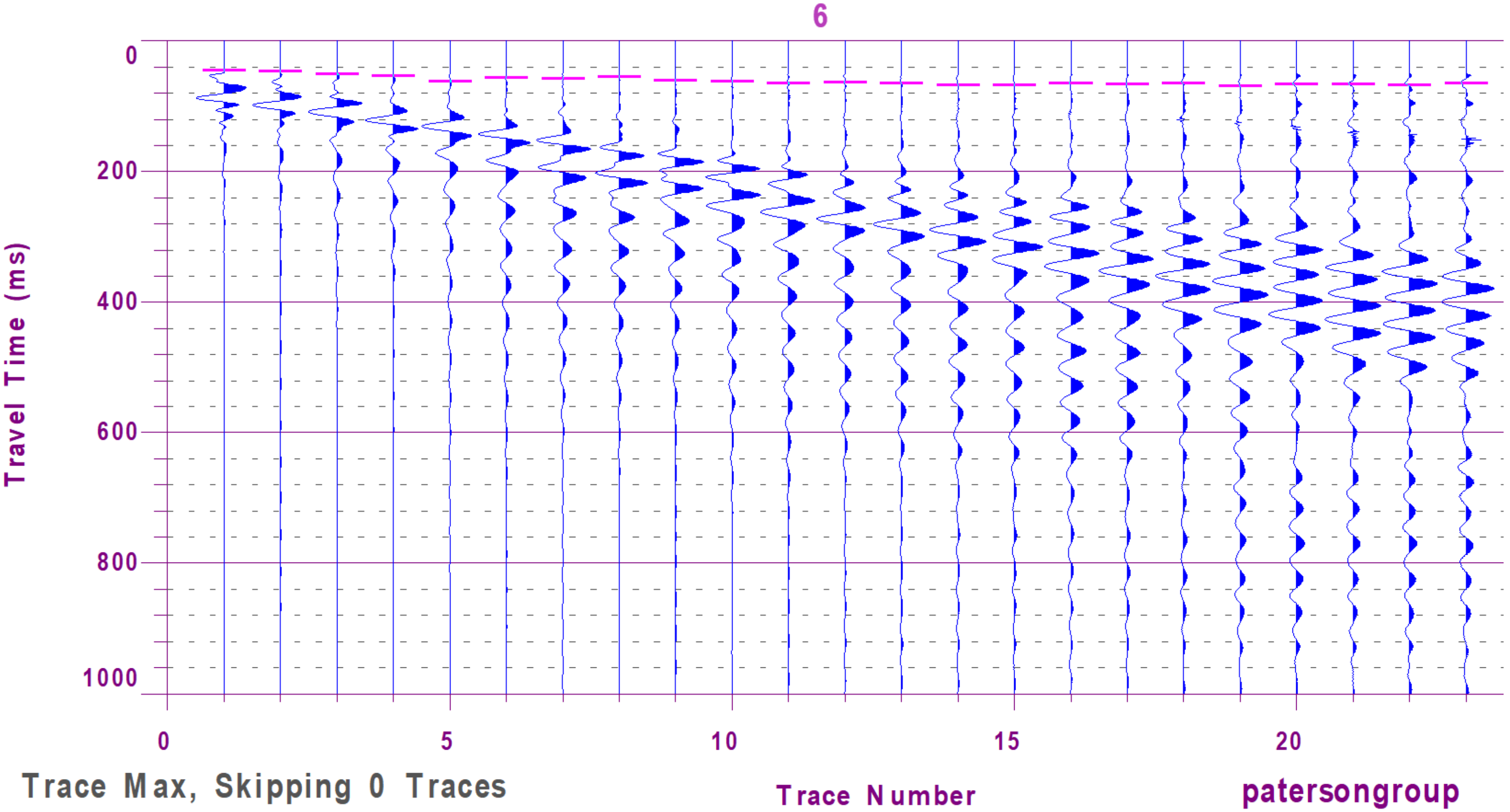


Figure 2 - Shear Wave Velocity Profile at Shot Location -3 m

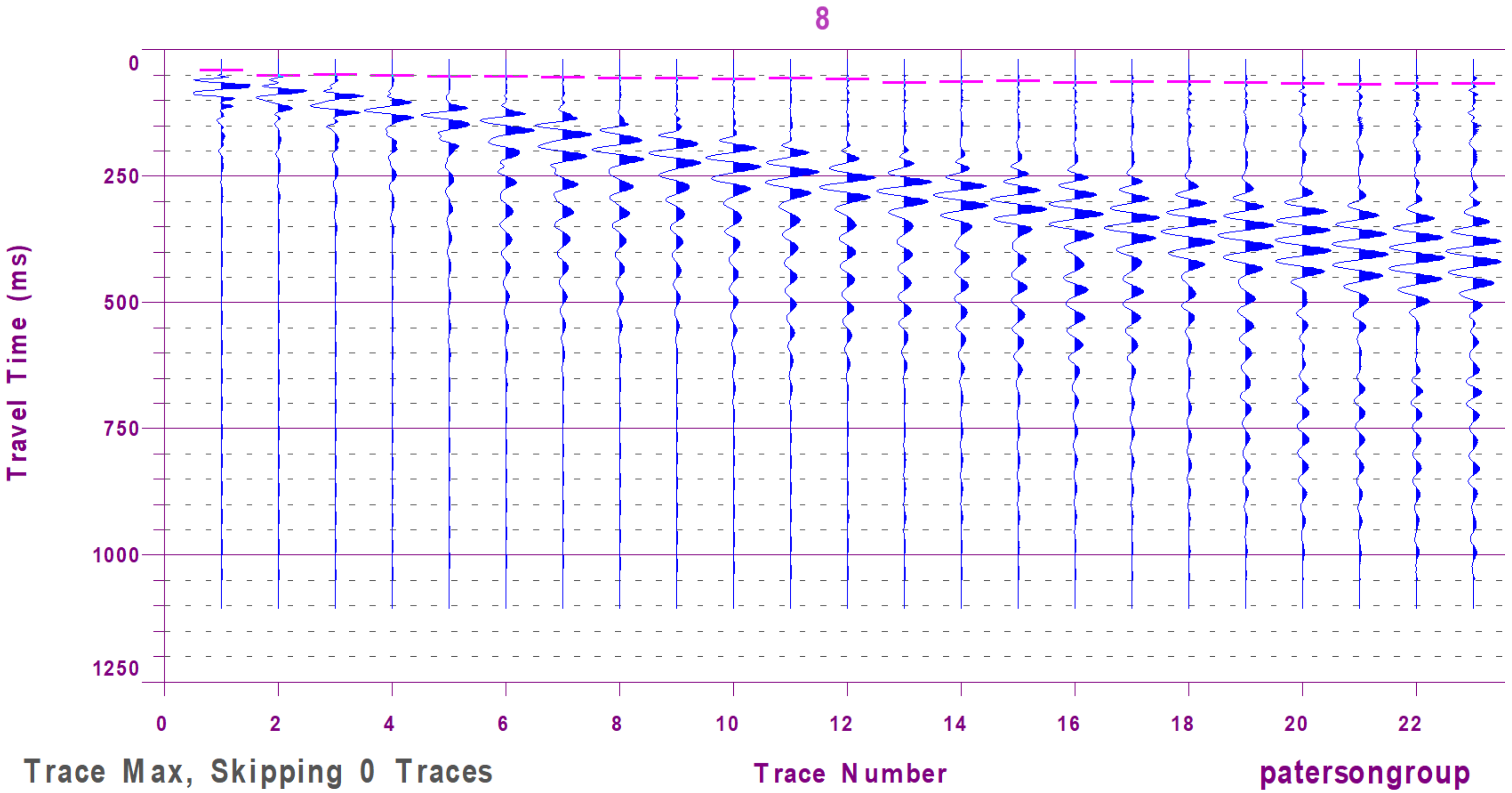


Figure 3 - Shear Wave Velocity Profile at Shot Location -2 m

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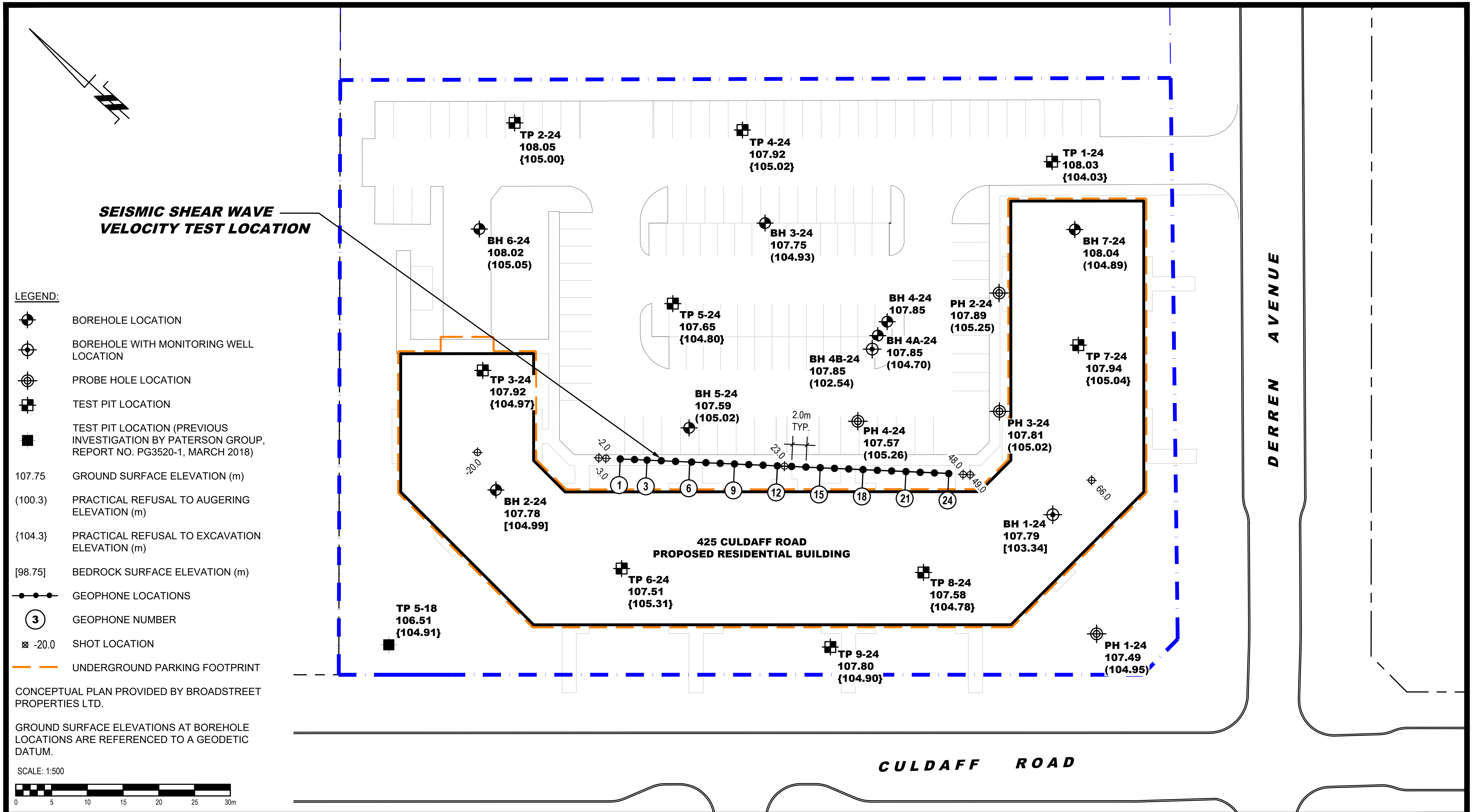

LEGEND:

⊕	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
◎	PROBE HOLE LOCATION
▣	TEST PIT LOCATION
■	TEST PIT LOCATION (PREVIOUS INVESTIGATION BY PATERSON GROUP, REPORT NO. PG3520-1, MARCH 2018)
107.75	GROUND SURFACE ELEVATION (m)
(100.3)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
{104.3}	PRACTICAL REFUSAL TO EXCAVATION ELEVATION (m)
[98.75]	BEDROCK SURFACE ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⊠ -20.0	SHOT LOCATION
— —	UNDERGROUND PARKING FOOTPRINT

CONCEPTUAL PLAN PROVIDED BY BROADSTREET PROPERTIES LTD.

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

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

NO.	REVISIONS	DATE	INITIAL

SEYMOUR PACIFIC DEVELOPMENTS (ONTARIO) LTD
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
425 CULDAFF ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:500
Drawn by: ZS
Checked by: NFRV
Approved by: DP
Date: 05/2024
Report No.: PG7040-1
Dwg. No.: PG7040-1
Revision No.:

11x17

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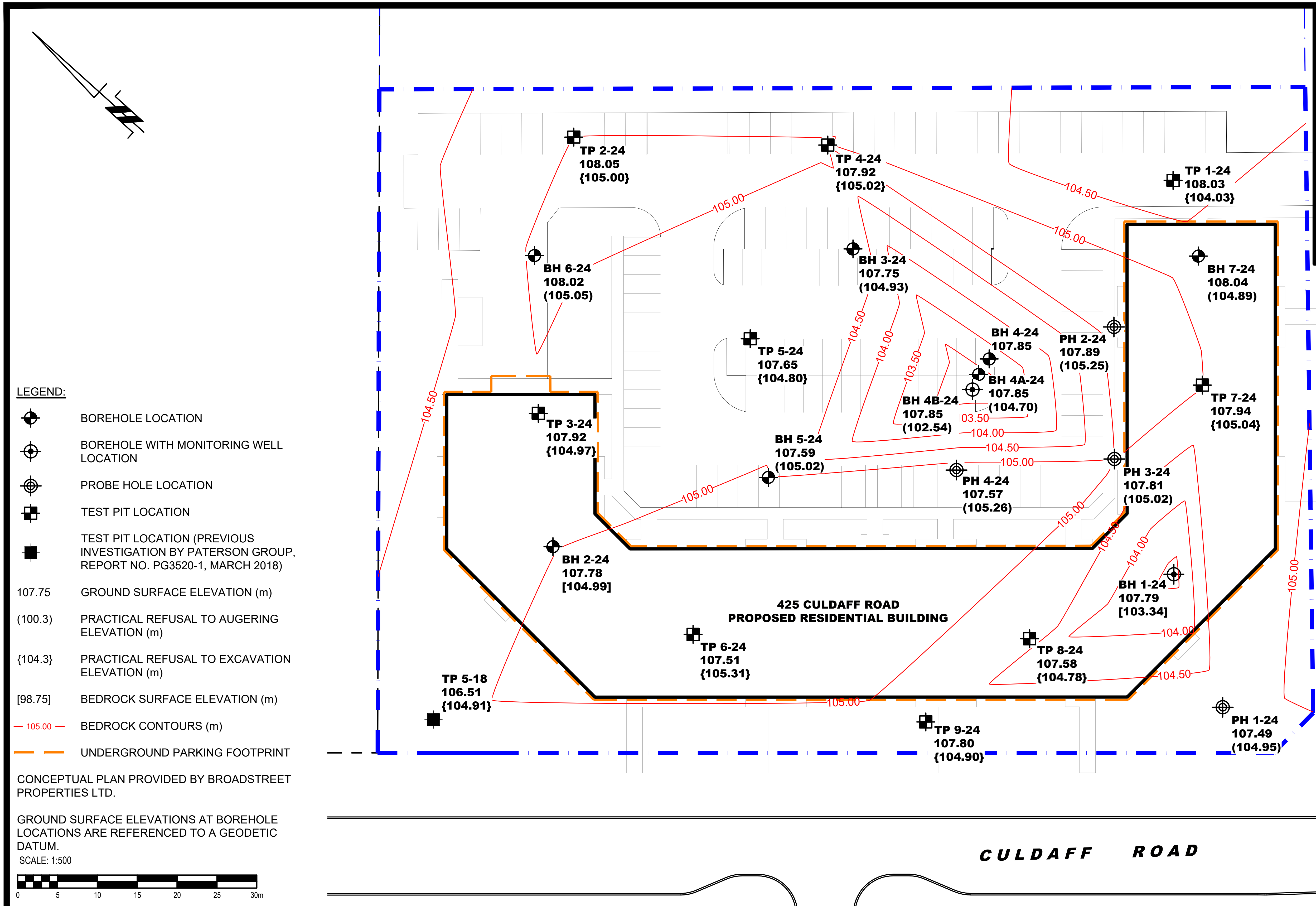

NOTE FOR BEDROCK CONTOUR LINES:

BEDROCK CONTOURS DEPICTED HEREIN ARE BASED ON LINEAR INTERPOLATION BETWEEN TEST HOLES WHERE BEDROCK HAS BEEN ENCOUNTERED BY PATERSON THROUGHOUT THE SUBJECT SITE AND IS LIMITED TO THAT INFORMATION.

ACTUAL SITE CONDITION AND BEDROCK DEPTHS/ELEVATIONS MAY VARY BEYOND TEST HOLE LOCATIONS AND AS DEPICTED BY THE CONTOUR LINES.

BASED ON THIS, IT SHOULD BE UNDERSTOOD THAT THE BEDROCK SURFACE MAY VARY WITHIN PLUS OR MINUS 0.5 TO 1.5m AT CONTOUR LINE LOCATIONS WHERE THE BEDROCK SURFACE HAS NOT BEEN DISCRETELY CONFIRMED BY A TEST HOLE.

LEGEND:

⯐	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
◎	PROBE HOLE LOCATION
⊞	TEST PIT LOCATION
■	TEST PIT LOCATION (PREVIOUS INVESTIGATION BY PATERSON GROUP, REPORT NO. PG3520-1, MARCH 2018)
107.75	GROUND SURFACE ELEVATION (m)
(100.3)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
{104.3}	PRACTICAL REFUSAL TO EXCAVATION ELEVATION (m)
[98.75]	BEDROCK SURFACE ELEVATION (m)
— 105.00 —	BEDROCK CONTOURS (m)
— —	UNDERGROUND PARKING FOOTPRINT

CONCEPTUAL PLAN PROVIDED BY BROADSTREET PROPERTIES LTD.

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

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

SEYMOUR PACIFIC DEVELOPMENTS (ONTARIO) LTD
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
425 CULDAFF ROAD

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:500	Date: 05/2024
Drawn by: ZS	Report No.: PG7040-1
Checked by: NFRV	Dwg. No.: PG7040-2
Approved by: DP	Revision No.:

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