PATERSON
GROUP

Geotechnical Investigation

Proposed Residential Development

800 Cedarview Road
Ottawa, Ontario

Prepared for Mattamy Homes

Report PG5600-1 Revision 1 dated June 3, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Mattamy Homes to conduct a geotechnical investigation for the proposed residential development to be located at 800 Cedarview Road in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

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

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

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

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of townhouses, single-family homes, condominiums and mixed-use buildings with associated driveways, local roadways and parking areas. Landscaped areas, park lands and stormwater management facilities are also proposed.

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

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was carried out at the subject site from March 27 to April 1, 2024, and consisted of advancing a total of 8 boreholes to a maximum depth of 6.3 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration existing site features and underground services.

A previous investigation was conducted by Paterson on December 2 and 3, 2020, and consisted of advancing a total of 14 test pits to a maximum depth of 6.0 m below existing ground surface. The approximate locations of the test holes are shown on Drawing PG5600-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two-person crew. The test pits were completed using an excavator and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure consisted of augering or excavating to the required depths at the selected locations, sampling and testing the overburden and bedrock.

Sampling and In-Situ Testing

Soil samples were collected from the boreholes either by sampling directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and rock cores were placed in cardboard boxes.

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

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

Bedrock samples were recovered from all boreholes with the exception of BH 3-24 and BH 5-24, using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. The values are indicative of the bedrock quality.

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.

Sample Storage

All samples will be stored in the laboratory for a period of 1 month after issuance of this report. They will then be discarded unless we are otherwise directed.

Groundwater

Groundwater monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels following the completion of drilling. The groundwater level readings were measured after a suitable stabilization period subsequent to the completion of the field investigation. Additionally, the depth at which water infiltration was encountered through the sidewalls of the test pits was recorded prior to the completion of excavation.

Typical monitoring well construction details are described below.

- ☐ 1.5 m of slotted 50.8 mm diameter PVC screen at the base of the boreholes within overburden.
- ☐ 3 m of slotted 31.7 mm diameter PVC screen at the base of the borehole within bedrock
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen
- ☐ Clean backfill from top of bentonite plug to the ground surface.

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

Long-Term Groundwater Level Monitoring

Submersible dataloggers (Van Essen Instrument TD-Diver Water Level Datalogger) were installed in all monitoring wells to record long-term groundwater level fluctuation by measuring hydrostatic pressure of the water above the sensor.

Continuous reading of groundwater level will be recorded from the installed datalogger for a maximum of 8-month period. Paterson will conduct a regular data logger check to ensure that equipment is functioning properly before spring freshet.

3.2 Field Survey

The test hole locations, and the ground surface elevations at each test hole were surveyed by Paterson using a handheld GPS unit, and referenced to a geodetic datum. The test hole locations and elevations are presented on Drawing PG5600-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered and visually examined in our laboratory to review the results of the field logging. Moisture content testing was conducted on all soil samples.

Additionally, 5 samples were submitted for grain size distribution analysis during the previous investigation. The results are discussed in Section 4.2 and are provided in Appendix 1 of this report.

3.4 Analytical Testing

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

3.5 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at all monitoring well locations to assist in confirming anticipated groundwater flow rates within the subsoil and /or bedrock at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951).

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden and/or bedrock aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.05 m in overburden (a diameter of 0.03 m in bedrock). 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. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin. The semi-log drawdown vs. time plots for rising and falling head at each borehole location are presented in Appendix 1.

The results of testing and hydrogeological recommendations are further discussed in Subsections 4.2.

4.0 Observations

4.1 Surface Conditions

The subject site was observed to heavily forested with several walk trails and small water streams. The ground surface across the subject site is relatively flat with a downward slope towards the north-east and south-east directions. A Hydro Ottawa corridor crosses the site from east to west within the northern portion of the site. A 2 to 4 m high fill pile is present within the central west portion of the site. Further, based on available aerial photos, and the subsurface conditions encountered in the boreholes within the central portion of the site, the property was formerly used as a quarrying operation sometime before 1964 and continued until as recently as 1991.

Further, a large volume of fill material was observed to have been placed within the central portion of the site between 1991 and 1999 and is anticipated to have been associated with construction of the adjacent Highway 416. Reference should be made to the aerial photographs in Figure 2 – Aerial Photograph – 1965 and Figure 3 – Aerial Photograph – 1999 which illustrate the site conditions described above.

The site is bordered by Highway 416 to the north and west, by residential dwellings to the east and parkland to the south.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil and/or fill material underlain by a glacial till deposit. The fill material was observed to extend to depths of 2.0 to 5.6 m at test holes BH 5-24, TP 3-20, TP5-20, and TP12-20 within the central portion of the subject site. Further, it should be noted that the thickness of fill material was not penetrated at test pit TP 11-20 which was terminated at a depth of 6.0 m. Fill material was also encountered at BH 3-24 and TP 6-20 within the southwest and northern portions of the site, and extended to depths of 0.7 and 0.4 m, respectively. The fill material was generally observed to consist of brown silty clay with sand gravel, cobbles and boulders and trace amounts of crushed stone, topsoil and organics.

An approximate 0.3 to 0.7 m thick layer of compact brown silty sand layer was encountered below the topsoil and/or fill material at test holes BH2-24, BH4-24, TP 3-20, TP6-20, and TP8-20.

A glacial till deposit was encountered below the aforementioned topsoil/fill and silty sand layers at the majority of the test hole locations. The glacial till deposit was observed to consist of a compact to very dense silty sand with clay, gravel, cobbles and boulders. The glacial till was noted to transition from brown to grey in colour at approximate depths of 2.2 and 4.1 m below the existing ground surface at boreholes BH 1-24 and BH 3-24, respectively.

Bedrock

Practical refusal to excavation and augering was encountered at approximate depths ranging from 0.1 to 3.5 m across the majority of the subject site. Bedrock outcrops were observed at surface within the northern portion of the subject site. However, at the test holes located within the aforementioned fill area (BH5-24, TP 3-20, TP 5-20, and TP 11-20), practical refusal was encountered at approximate depths of 5.3 to 5.6 m below the existing ground surface. Test pit TP 11-20 was terminated in the fill material at a depth of 6.0 m.

Further, practical refusal to augering was not encountered in borehole BH 3-24, located within the southern portion of the site. The borehole was terminated within the glacial till deposit at an approximate depth of 6.3 m below the existing ground surface.

The bedrock was cored at boreholes BH 1-24, BH 2-24, BH 4-24 and BH 6-24 to BH 8-24, and based on the recovered rock core, was observed to consist of fair to excellent quality, grey limestone. The bedrock was cored to a maximum depth of 6.5 m below the existing ground surface. At borehole BH 8-24, the upper 4.1 m of bedrock was observed to consist of a good quality grey sandstone.

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.

Grain Size Distribution Testing

Grain size distribution analysis was carried out on a total of 5 recovered soil samples. The results are summarized in Table 1 on the following page and presented in Appendix 1.

Table 1 – Summary of Grain Size Distribution Results

Test Hole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silty (%)	Clay (%)
TP 3-20	G2	2.0-2.3	20.1	29.0	24.2	26.7
TP 3-20	G4	4.4-4.7	0.3	80.3	19.4	
TP 6-20	G2	0.4-0.7	8.0	67.1	24.9	
TP 6-20	G3	0.7-1.0	23.8	50.0	26.2	
TP 11-20	G2	2.1-2.4	7.9	53.8	38.3	

Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) tests were conducted at all borehole locations 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 2 below.

Table 2 – Summary of hydraulic conductivity values.

Test Hole	Ground Surface Elevation (m)	Screen Interval (m)	K (m/sec)	Test Type	Soil Type/Bedrock
BH 1-24	104.96	4.9 to6.4	$6.41x10^{-5}$	Falling Head	Limestone Bedrock
			$7.12x10^{-5}$	Rising Head	
BH 2-24	108.74	4.8 to 6.3	$4.99x10^{-7}$	Rising Head	Limestone Bedrock
BH 3-24	110.78	4.7 to 6.2	$1.60x10^{-6}$	Rising Head	Glacial Till
BH 4-24	109.95	4.8 to 6.3	$1.65x10^{-7}$	Falling Head	Limestone Bedrock
BH 5-24	116.50	3.9 to 5.4	$2.32x10^{-5}$	Rising Head	Fill Material
BH 6-24	113.36	4.9 to 6.4	$7.90x10^{-7}$	Falling Head	Limestone Bedrock
BH 7-24	112.93	4.6 to 6.1	$4.81x10^{-6}$	Falling Head	Limestone Bedrock
			$5.65x10^{-6}$	Rising Head	
BH 8-24	110.77	4.6 to 6.1	$3.97x10^{-6}$	Falling Head	Limestone Bedrock

The hydraulic conductivity values (K) measured in the limestone bedrock ranged from 1.65 x 10^{-7} to 7.12 x 10^{-5} m/s. These values are generally consistent with similar material Paterson has encountered on other sites and typical published values for limestone bedrock. The higher hydraulic conductivity value identified at BH1-24 can be attributed to the variability in composition and consistency of the bedrock encountered at the screened interval.

The slug testing completed at borehole BH 3-24 within the glacial till deposit identified a hydraulic conductivity value of 1.60×10^{-6} m/sec which is also consistent with similar material Paterson has encountered on other sites and typical published values for glacial till.

The slug testing completed at BH5-24, screened within the fill layer, identified a hydraulic conductivity value of 2.32×10^{-5} m/sec. Reference should be made to Appendix 1 for the details of the hydraulic conductivity value at each test hole location with respect to screen depth and overburden/bedrock composition.

4.3 Groundwater

Groundwater levels were measured in the installed monitoring wells on April 17, 2024. Data loggers were installed in all monitoring wells to record seasonal fluctuations in water levels across the site. Further, groundwater infiltration into the open test pits were recorded in the field at the time of the previous investigation.

Table 3 – Summary of Groundwater Levels

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-24	104.96	At surface	-	April 17, 2024
BH 2-24	108.74	0.45	108.29	April 17, 2024
BH 3-24	110.78	0.18	110.60	April 15, 2024
BH 4-24	109.95	1.21	108.74	April 17, 2024
BH 5-24	116.50	4.25	112.25	April 17, 2024
BH 6-24	113.36	1.47	111.89	April 15, 2024
BH 7-24	112.93	0.65	112.28	April 15, 2024
BH 8-24	110.77	0.33	110.44	April 15, 2024
TP 1-20	112.09	Dry	-	December 2, 2020
TP 2-20	113.06	Dry	-	December 2, 2020
TP 3-20	113.39	Dry	-	December 2, 2020
TP 4-20	113.64	Dry	-	December 2, 2020
TP 5-20	119.76	Dry	-	December 2, 2020
TP 6-20	111.27	Dry	-	December 2, 2020

Table 3 – Summary of Groundwater Levels – Continued

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
TP 7-20	111.20	Dry	-	December 2, 2020
TP 8-20	109.81	0.30	109.51	December 2, 2020
TP 9-20	104.82	2.50	102.32	December 2, 2020
TP 10-20	115.84	Dry	-	December 2, 2020
TP 11-20	119.89	Dry	-	December 2, 2020
TP 12-20	115.44	Dry	-	December 2, 2020
TP 13-20	108.47	Dry	-	December 3, 2020
TP 14-20	113.51	Dry	-	December 3, 2020

Note: Ground surface elevations at borehole location are referenced to a geodetic datum.

The groundwater table can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table is generally expected to be located within the bedrock. Where water was observed within the overburden soils, it is generally expected to consist of water perched above the bedrock.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction. Continuous groundwater levels will be recorded via the data loggers installed within the monitoring wells.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is anticipated that the proposed buildings will be founded on conventional spread footings bearing on a clean, surface sounded bedrock, undisturbed, compact silty sand, compact to very dense glacial till, or on the existing fill surface which is prepared in accordance with the “Subgrade Improvement Program for Foundations” procedure provided in Section 5.2.

Where the footings are designed to be supported on bedrock, which is encountered below the underside of footing elevation, the overburden should be excavated to the surface of the clean, surface sounded bedrock and replaced with lean concrete to the proposed founding elevation.

It is anticipated that bedrock removal will be required for building construction and servicing installation. Therefore, all contractors should be prepared for bedrock removal within the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints. However, it is recommended that the existing fill layer be proof-rolled several times under dry conditions and above freezing temperatures, and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Bedrock Removal

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

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

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding structures.

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

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.

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

Two parameters 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).

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

Fill Placement

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

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

Non- specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

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 300 mm. Where the fill is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

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

Lean Concrete Filled Trenches

Where footings are designed to be supported on clean, surface sounded bedrock is encountered at the USF, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete (minimum 17 MPa 28-day compressive strength) to the founding elevation.

Typically, the excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of excavation. The additional width of the concrete poured will suffice in providing a direct transfer of the footing load to the underlying clean, surface sounded bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Subgrade Improvement Program for Foundations

The following subgrade improvement program is recommended for areas where fill, free of significant amounts of deleterious materials, is encountered at the underside of footing elevation for the proposed residential buildings.

☐ The bearing surface at design underside of footing level should be sub-excavated at least 500 mm below footing level, extending at least 1 m beyond the outside faces of the footing.

☐ The footing subgrade should be proof-compacted with a vibratory drum roller or large vibratory plate compactor. Any performing areas should be removed and replaced with an OPSS Granular B Type II material placed in maximum 300 mm loose lifts and compacted by a vibratory drum roller making several passes and witnessed by the geotechnical consultant.

☐ The sub-excavated area should be in-filled up to the design underside of footing elevation with engineered fill, such as OPSS Granular B Type II, placed in maximum 300 mm loose lifts and compacted to at least 98% of the material's SPMDD.

5.3 Foundation Design

Footings supported directly on clean, surface-sounded bedrock, or on lean concrete trenches which are placed directly over the clean surface-sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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

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

Footings placed on an undisturbed, compact silty sand, compact to very dense glacial till or engineered fill placed over an approved existing fill subgrade which is prepared in accordance with the "Subgrade Improvement Programs for Foundations" procedure above, can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa** incorporating a geotechnical factor of 0.5.

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

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

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 the above-noted overburden soils and engineered fill bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

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 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. Weathered bedrock will require a lateral support zone of 1H:1V (or flatter).

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements.

Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material, as shown in Figure 1 below. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

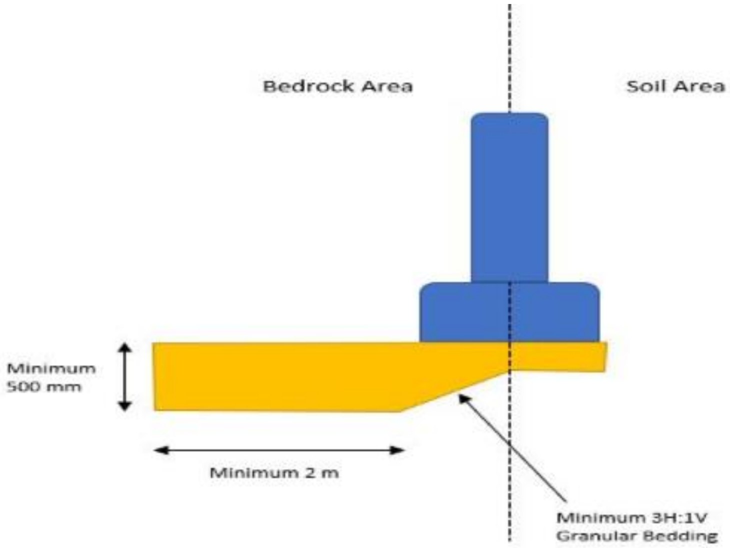


Figure 1 – Bedrock/Soil Transition Treatment

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class (Class A or B) is required for the proposed buildings which are founded on or within 3 m of the bedrock surface, a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed buildings, the native soil, bedrock, or approved engineered fill pad surface approved by Paterson will be considered an acceptable subgrade upon which to commence backfilling for basement slab or slab-on-grade construction.

Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface. The proof-compaction program should be witnessed and approved by Paterson. Any poor performing areas should be removed and reinstated with an engineered fill such as OPSS Granular A, Granular B Type II with a maximum particle size of 50 mm. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub- floor fill consists of 19 mm clear crushed stone.

5.6 Basement Wall

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

Where undrained conditions are anticipated (i.e. below groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).
The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot H^2/g$ where:

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above. The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

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

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

5.7 Pavement Design

For design purposes, the pavement structure presented in the following tables can be used for the design of car parking areas and local roadways.

Table 4 – Recommended Pavement Structure – Driveways and Car Parking Areas

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

Table 5 – Recommended Pavement Structure – Local Residential Roadways

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

Table 6 – Recommended Pavement Structure – Roadways with Bus Traffic

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material’s SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended for each proposed structure. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

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 drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

6.3 Excavation Side Slopes

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

Unsupported Excavations

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

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

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

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

Bedrock Stabilization

Where required, excavation side slopes in sound bedrock can be carried out using almost vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

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

The requirement for temporary rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineers during the design stage of the project.

6.4 Pipe Bedding and Backfill

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

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

Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A and be placed in maximum size of 25 mm. The cover material should be placed in maximum 300 mm thick lifts and compacted to 99% of the material’s SPMDD.

It should generally be possible to re-use the dry to moist (not-wet) silty sand and glacial till above the cover material if excavation and filling operations are carried out in dry weather conditions. Any stones greater than 300 mm in their longest dimension should be removed from these materials prior to placement.

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

6.5 Groundwater Control

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

Groundwater Control for Building Construction

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

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

6.6 Winter Construction

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

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

Under winter conditions, if snow and ice are present within the blast rock or other 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 personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

☐ Review detailed grading plan(s) from a geotechnical perspective once available.

☐ Review of the bedrock stabilization and excavation requirements if required.

☐ Review of the proposed foundation drainage and under-slab system.

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

☐ Sampling and testing of the concrete and fill materials.

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

☐ Observation of all subgrades prior to backfilling.

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

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess 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 Mattamy Homes, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Sok Kim, EIT.

LICENSED PROFESSIONAL ENGINEER
June 3, 2024
K. A. PICKARD
100531344
PROVINCE OF ONTARIO

Kevin A. Pickard, P.Eng.

Report Distribution:

☐ Mattamy Homes (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION TESTING RESULTS

ANAYLTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

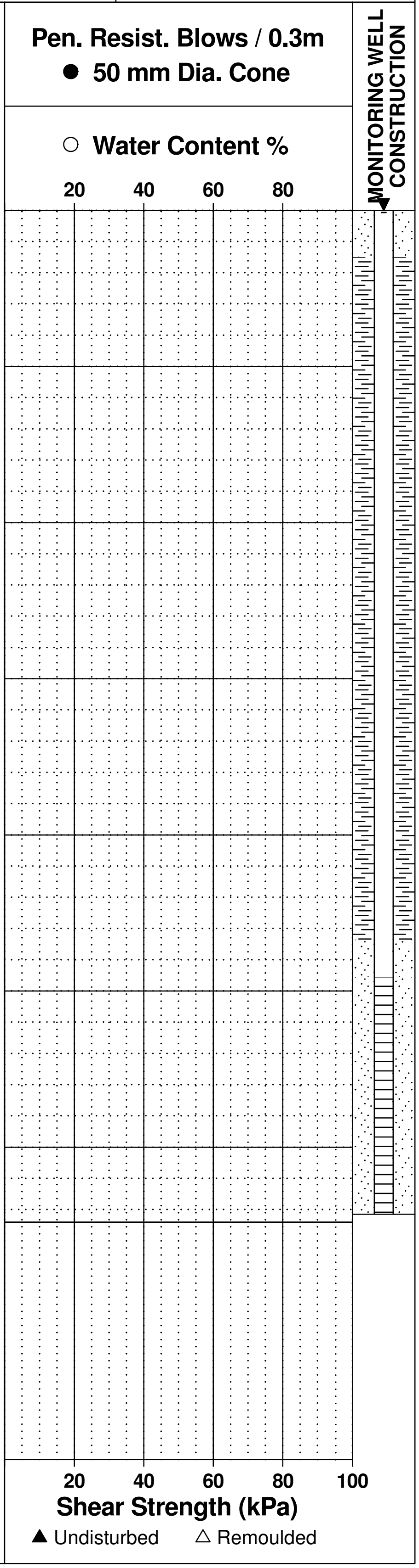
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 360042.722 **NORTHING:** 5015631.119 **ELEVATION:** 104.96
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 March 27

FILE NO. PG5600
HOLE NO. BH 1-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.96
TOPSOIL with organics — 0.28						
GLACIAL TILL: Compact, brown silty sand to sandy silt with clay and gravel, occasional cobbles	AU	1				
	SS	2	100	8	1	103.96
— 2.21	SS	3	50	14	2	102.96
GLACIAL TILL: Compact to dense, grey silty sand with gravel, cobbles and boulders	SS	4	100	28		
— 3.30	SS	5	80	+50	3	101.96
BEDROCK: Fair quality, grey limestone bedrock - Good to excellent quality by 3.6m depth	RC	1	100	60	4	100.96
	RC	2	100	100	5	99.96
— 6.48	RC	3	100	100	6	98.96
End of Borehole (GWL at surface - April 17, 2024)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359831.302 **NORTHING:** 5015735.647 **ELEVATION:** 108.74

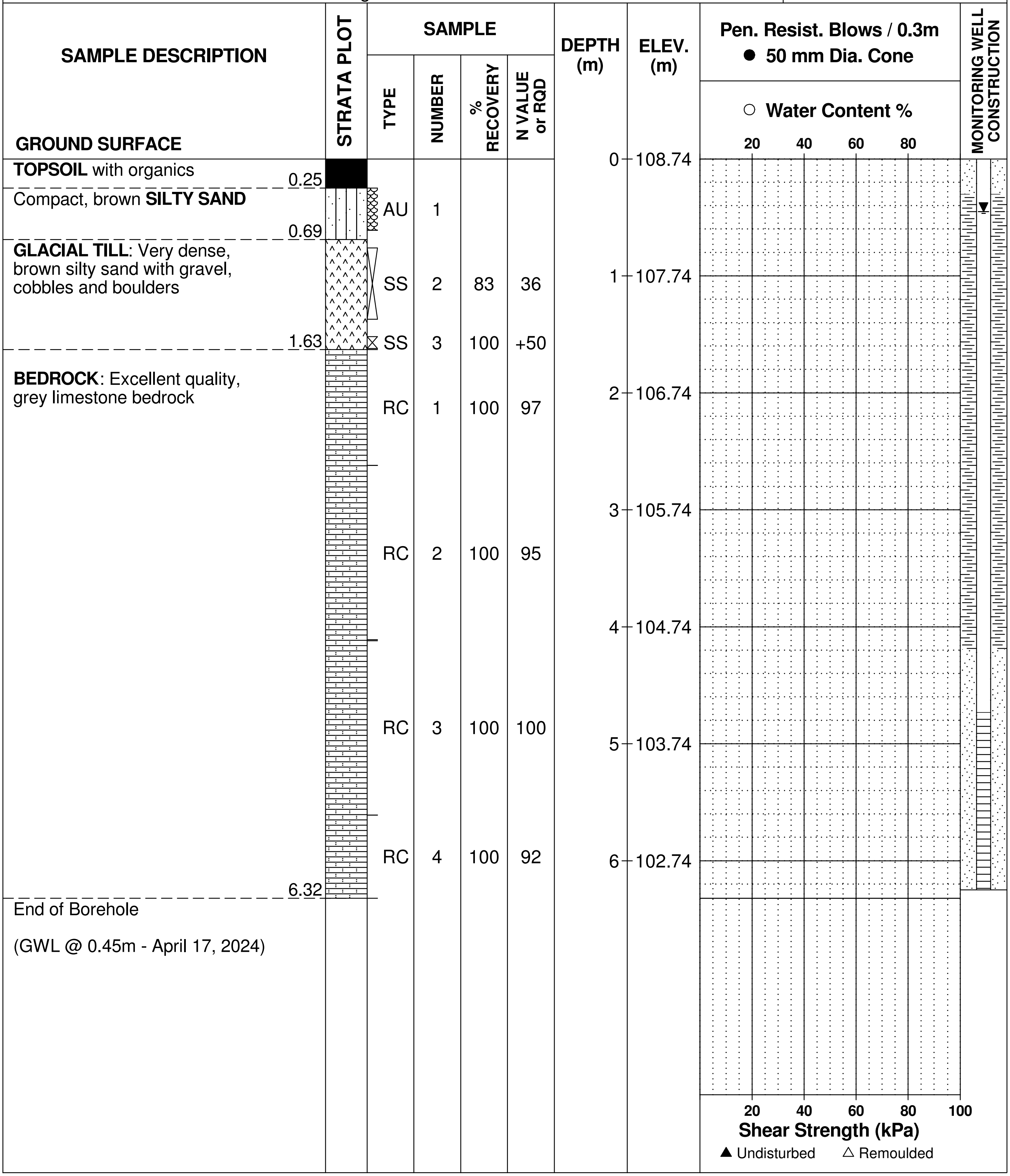
DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 March 27

FILE NO. PG5600

HOLE NO. BH 2-24



SAMPLE DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL with organics — 0.25				
Compact, brown **SILTY SAND** — 0.69	AU	1		
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	2	83	36
— 1.63	SS	3	100	+50
BEDROCK: Excellent quality, grey limestone bedrock	RC	1	100	97
	RC	2	100	95
	RC	3	100	100
	RC	4	100	92
— 6.32				
End of Borehole				
(GWL @ 0.45m - April 17, 2024)				

DEPTH (m)	ELEV. (m)
0	108.74
1	107.74
2	106.74
3	105.74
4	104.74
5	103.74
6	102.74

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

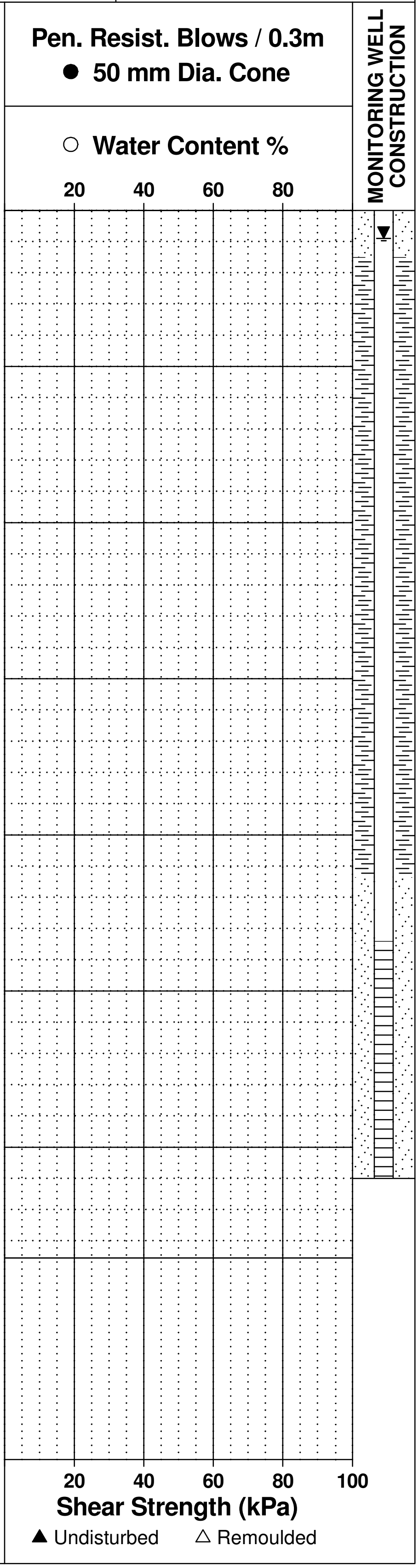
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359838.455 **NORTHING:** 5015502.999 **ELEVATION:** 110.78
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 March 27

FILE NO. PG5600
HOLE NO. BH 3-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	110.78
TOPSOIL with organics 0.28						
FILL: Brown silty sand to sandy silt with clay 0.69	AU	1				
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders	SS	2	62	+50	1	109.78
	RC	1	32		2	108.78
					3	107.78
	RC	2	29			
					4	106.78
- Grey by 4.1m depth	SS	3	55	+50		
	SS	4	80	+50		
					5	105.78
					6	104.78
6.32	SS	5	0	+50		
End of Borehole						
(GWL @ 0.18m - April 15, 2024)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359856.182 **NORTHING:** 5015913.238 **ELEVATION:** 109.95

DATUM: Geodetic

REMARKS:

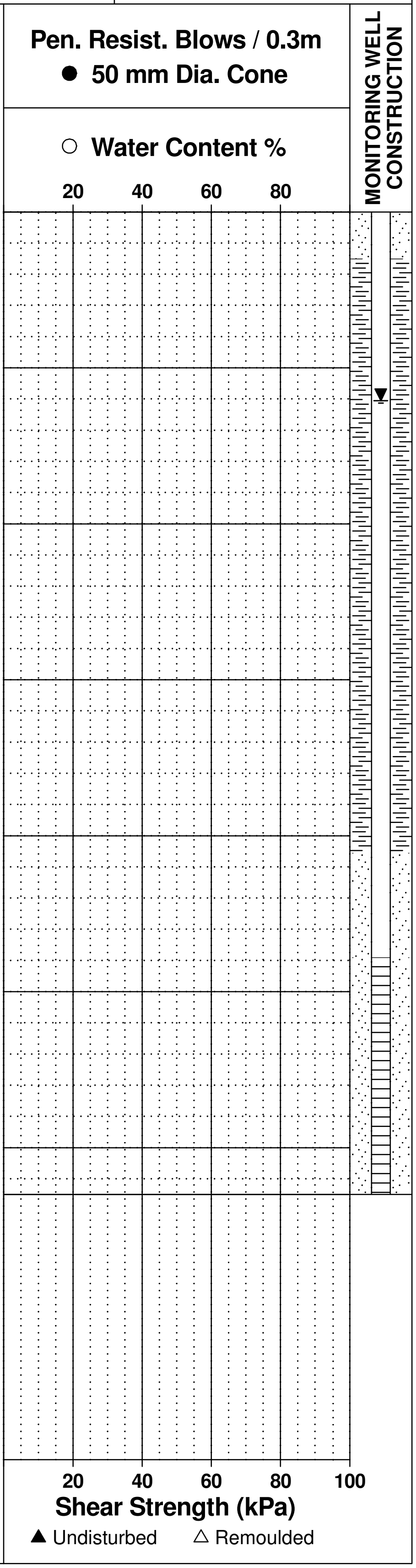
BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 March 28

FILE NO. PG5600

HOLE NO. BH 4-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL with organics	0.25				
Brown **SILTY SAND**, trace gravel	0.69	AU	1		
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	0.86	SS	2	0	+50
		RC	1	100	100
BEDROCK: Excellent quality, grey limestone bedrock - Fair to good quality from 1.4m to 2.3m depth		RC	2	100	64
		RC	3	100	86
		RC	4	100	100
		RC	5	100	100
End of Borehole (GWL @ 1.21m - April 17, 2024)	6.30				

DEPTH (m)	ELEV. (m)
0	109.95
1	108.95
2	107.95
3	106.95
4	105.95
5	104.95
6	103.95



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359634.713 **NORTHING:** 5015846.98 **ELEVATION:** 116.50

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 March 28

FILE NO. PG5600

HOLE NO. BH 5-24

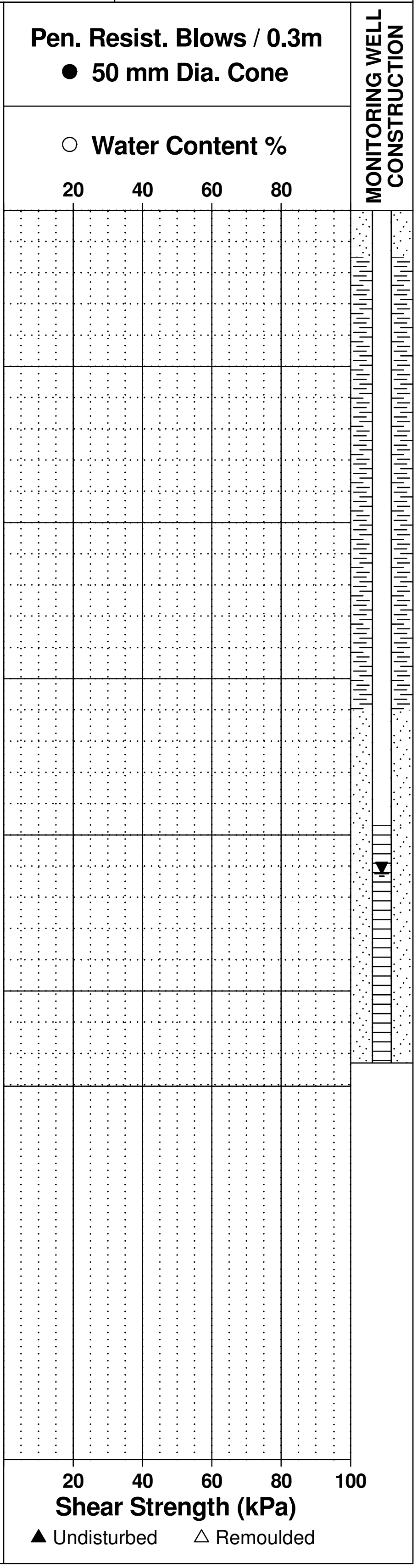
SAMPLE DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL with organics	0.05				
FILL: Brown silty clay, trace topsoil and organics		AU	1		
		SS	2	58	6
	1.45				
FILL: Brown silty clay with gravel, trace sand, wood and organics		SS	3	50	11
	2.21				
FILL: Brown silty clay with sand, gravel and cobbles, trace organics and crushed stone		SS	4	100	+50
	2.97				
FILL: Brown silty sand with gravel, cobbles and boulders, trace crushed stone		SS	5	50	39
		SS	6	60	+50
		SS	7	80	+50
	5.61				

End of Borehole

Practical refusal to augering at 5.61m depth

(GWL @ 4.25m - April 17, 2024)

DEPTH (m)	ELEV. (m)
0	116.50
1	115.50
2	114.50
3	113.50
4	112.50
5	111.50



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

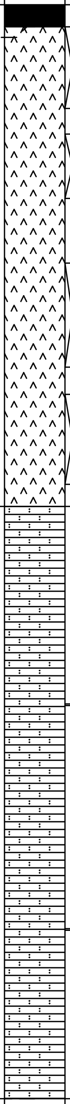
Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359420.374 **NORTHING:** 5016406.742 **ELEVATION:** 113.36
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 April 1

FILE NO. PG5600
HOLE NO. BH 6-24

SAMPLE DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	113.36
TOPSOIL with organics — 0.13						
GLACIAL TILL: Compact, brown silty sand with gravel, trace clay, cobbles and boulders	SS	1	75	7		
	SS	2	67	+50	1	112.36
	SS	3	17	9	2	111.36
— 2.95	SS	4	48	15		
BEDROCK: Fair quality, grey limestone bedrock					3	110.36
	RC	1	100	63	4	109.36
	RC	2	100	46	5	108.36
— 6.43	RC	3	100	67	6	107.36
End of Borehole (GWL @ 1.47m - April 15, 2024)						

STRATA PLOT



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

○ Water Content %

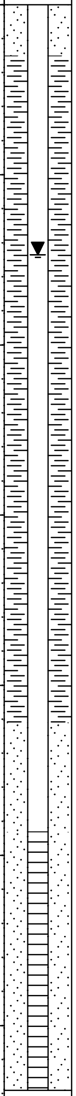
20 40 60 80

Shear Strength (kPa)

20 40 60 80 100

▲ Undisturbed △ Remoulded

MONITORING WELL CONSTRUCTION



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SOIL PROFILE AND TEST DATA

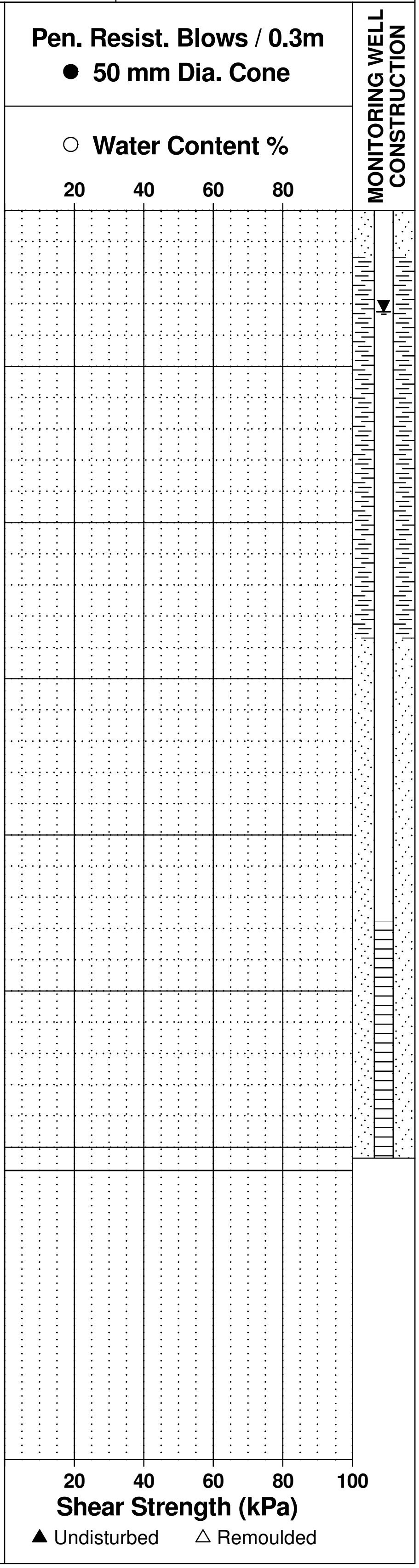
Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359561.736 **NORTHING:** 5016749.605 **ELEVATION:** 112.93
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 April 1

FILE NO. PG5600
HOLE NO. BH 7-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL with organics	0.20	SS	1	50	+50
GLACIAL TILL: Very dense, brown silty sand with gravel and cobbles	0.28				
BEDROCK: Fair to good quality, grey sandstone bedrock - Vertical fractures from 0.8m to 1.1m depth and from 1.3m to 2.1m depth	4.17	RC RC RC	1 2 3	100 100 100	42 81 90
BEDROCK: Good to excellent quality, grey limestone bedrock	6.15	RC RC	4 5	100 100	64 94
End of Borehole (GWL @ 0.65m - April 15, 2024)					

DEPTH (m)	ELEV. (m)
0	112.93
1	111.93
2	110.93
3	109.93
4	108.93
5	107.93
6	106.93



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 800 Cedarview Road
Ottawa, Ontario

EASTING: 359686.062 **NORTHING:** 5016930.791 **ELEVATION:** 110.77

DATUM: Geodetic

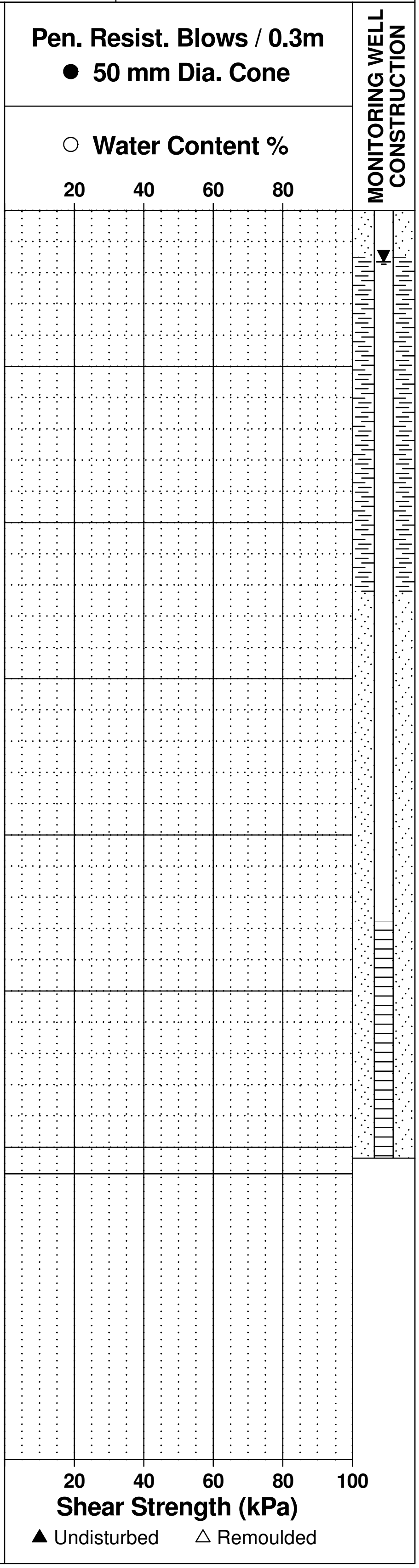
REMARKS:

BORINGS BY: CME 55 Mechanical Earth Auger on Track **DATE:** 2024 April 1

FILE NO. PG5600

HOLE NO. BH 8-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL with organics	0.10	SS	1	100	+50	0	110.77
GLACIAL TILL: Very dense, brown silty sand with gravel	0.30						
BEDROCK: Good quality, grey sandstone bedrock		RC	1	100	71	1	109.77
		RC	2	100	86	2	108.77
		RC	3	100	80	3	107.77
	4.29					4	106.77
BEDROCK: Good to excellent quality, grey limestone bedrock		RC	4	100	78	5	105.77
	6.17	RC	5	100	100	6	104.77
End of Borehole							
(GWL @ 0.33m - April 15, 2024)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

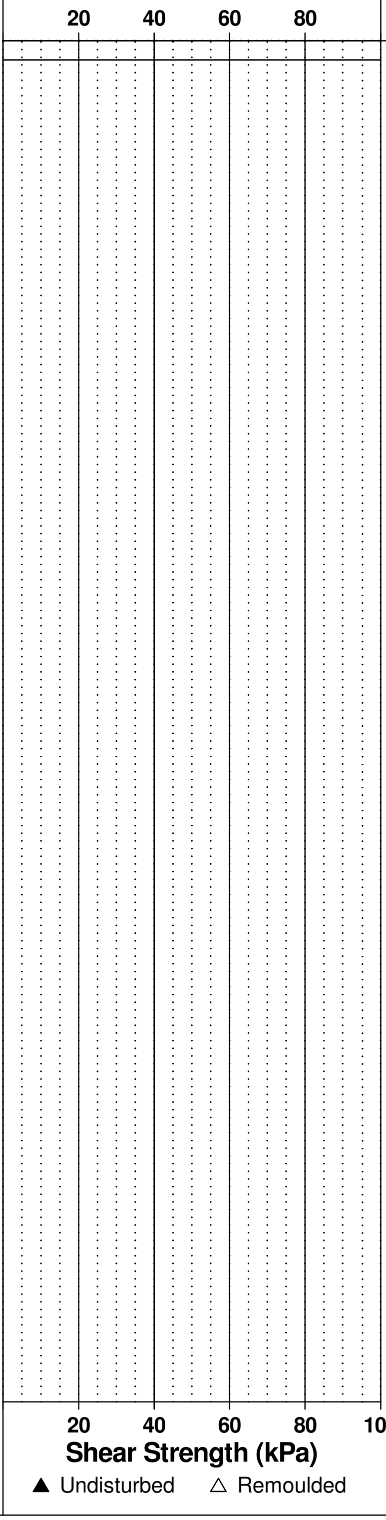
BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 1-20

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 %	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.10	■	G	1			0	112.09		
End of Test Pit TP terminated on bedrock surface at 0.10m depth. (TP dry upon completion)									



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 2-20

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									
TOPSOIL 0.20		G	1			0	113.06		
GLACIAL TILL: Compact, brown sandy silt, trace gravel and cobbles 1.20		G	2			1	112.06		
End of Test Pit TP terminated on bedrock surface at 1.20m depth. (TP dry upon completion)									

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

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Excavator
DATE December 2, 2020
FILE NO. PG5600
HOLE NO. TP 3-20

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	113.39		
FILL: Brown silty clay, trace sand, gravel, cobbles and roots	▨	G	1						
	▨					1	112.39		
	▨	G	2			2	111.39		
	▨					3	110.39		
4.00	▨					4	109.39		
TOPSOIL 4.30	■	G	3						
Firm, brown **CLAYEY SILTY FINE SAND** 5.00	▧	G	4			5	108.39		
End of Test Pit TP terminated on bedrock surface at 5.300m depth. (TP dry upon completion)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5600**

REMARKS

HOLE NO. **TP 4-20**

BORINGS BY Excavator

DATE December 2, 2020

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	113.64		
TOPSOIL 0.30		G	1						
Weathered **BEDROCK** 1.30		G	2			1	112.64		
End of Test Pit (TP dry upon completion)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	119.76	
	▨	G	1					
	▨					1	118.76	
FILL: Brown silty clay, trace sand, gravel, roots and concrete	▨					2	117.76	
	▨	G	2					
	▨					3	116.76	
	▨					4	115.76	
4.30								
TOPSOIL	■	G	3					
4.50								
GLACIAL TILL: Compact, brown sandy silt, trace gravel	^	G	4					
	^					5	114.76	
5.30								
End of Test Pit TP terminated on bedrock surface at 5.30m depth. (TP dry upon completion)								

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 6-20

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 %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
FILL: Dark brown silty clay , trace gravel 0.40		G	1			0	111.27		
Compact, brown **SILTY SAND,** some gravel 0.70		G	2						
GLACIAL TILL: Brown clayey silty fine sand, some gravel 1.10		G	3			1	110.27		
End of Test Pit TP terminated on bedrock surface at 1.10m depth. (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 7-20

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	111.20		
TOPSOIL 0.15	■	G	1						
End of Test Pit TP terminated on bedrock surface at 0.15m depth. (TP dry upon completion)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP 8-20

SOIL 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	109.81		
TOPSOIL 0.30	■	G	1						∇
Compact, brown **SILTY SAND** 0.70		G	2						
End of Test Pit TP terminated on bedrock surface at 0.70m depth. (Groundwater infiltration at 0.3m depth)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5600**

REMARKS

HOLE NO. **TP 9-20**

BORINGS BY Excavator

DATE December 2, 2020

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 %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
TOPSOIL 0.30		G	1			0	104.82		
		G	2						
						1	103.82		
GLACIAL TILL: Dense, brown silty sand, some clay, trace gravel, occational cobbles and boulders									
						2	102.82		
									∇
		G	3						
						3	101.82		
3.50									
End of Test Pit TP terminated on bedrock surface at 3.50m depth. (Groundwater infiltration at 2.5m depth)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP10-20

SOIL 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	115.84		
TOPSOIL 0.20	■	G	1						
End of Test Pit TP terminated on bedrock surface at 0.20m depth. (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 2, 2020

FILE NO. PG5600

HOLE NO. TP11-20

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	119.89		
	▨	G	1						
	▨					1	118.89		
	▨					2	117.89		
	▨	G	2						
FILL: Brown silty sand, some gravel, cobbles and boulders, trace organics	▨								
	▨					3	116.89		
	▨					4	115.89		
	▨	G	3						
	▨					5	114.89		
6.00						6	113.89		
End of Test Pit (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5600

REMARKS

HOLE NO. TP12-20

BORINGS BY Excavator

DATE December 2, 2020

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	115.44		
FILL: Brown silty clay, trace sand, gravel and organics, occasional cobbles and boulders	▨	G	1						
						1	114.44		
		G	2						
2.00						2	113.44		
End of Test Pit TP terminated on bedrock surface at 2.00m depth. (TP dry upon completion)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE December 3, 2020

FILE NO. PG5600

HOLE NO. TP13-20

SOIL 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
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
TOPSOIL 0.40		G	1			0	108.47		
End of Test Pit TP terminated on bedrock surface at 0.40m depth. (TP dry upon completion)									

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

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 800 Cedarview Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

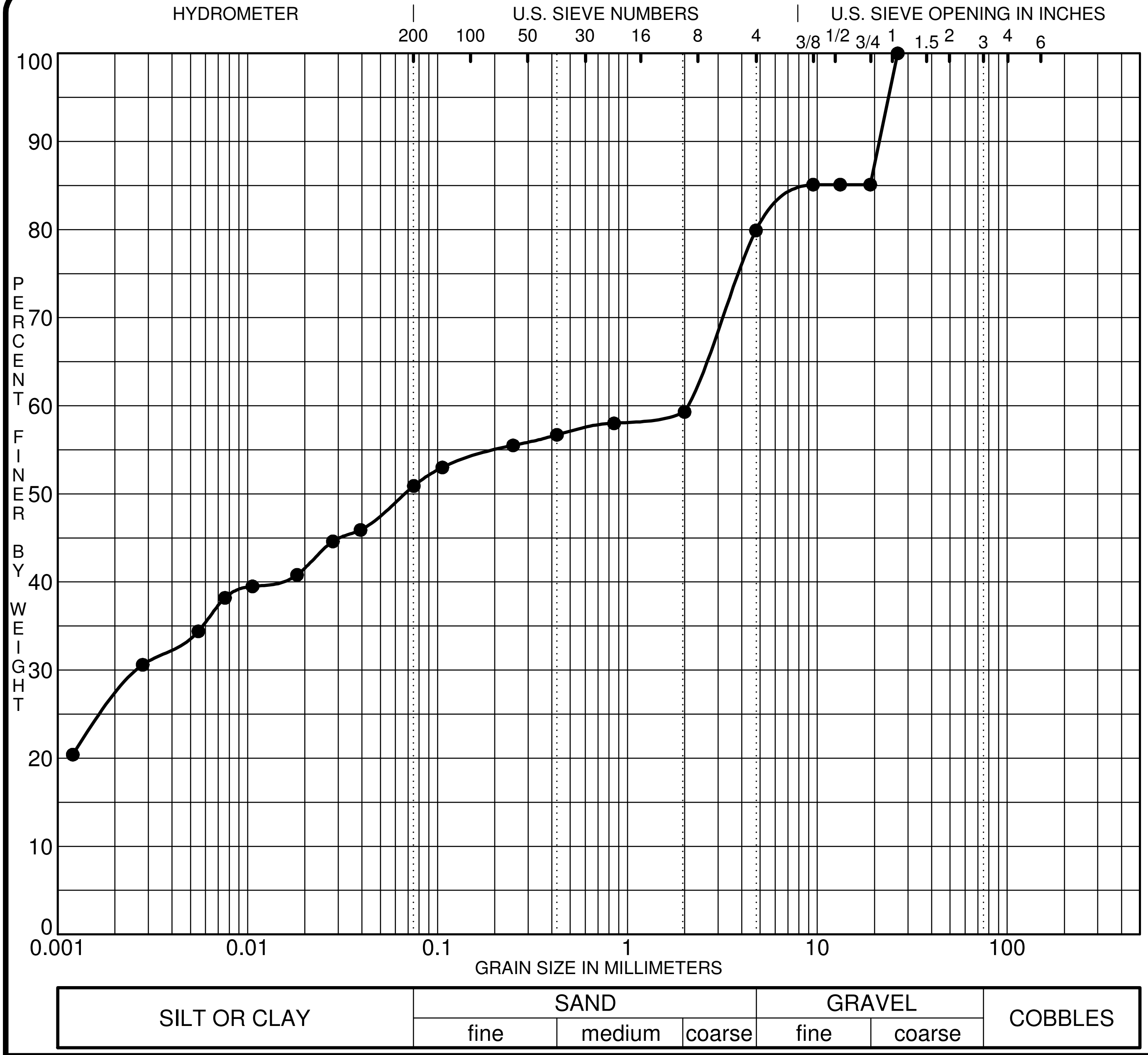
DATE December 3, 2020

FILE NO. PG5600

HOLE NO. TP14-20

SOIL 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									
TOPSOIL 0.10	▇	G	1			0	113.51		
End of Test Pit TP terminated on bedrock surface at 0.10m depth. (TP dry upon completion)									

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



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**TP 3-20**	**G 2**	**CL - Inorganic clays of low plasticity**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP 3-20**	**G 2**	**26.50**	**2.06**	**0.003**		**20.1**	**29.0**	**50.9**	

CLIENT **Mattamy Homes**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 800 Cedarview Road**

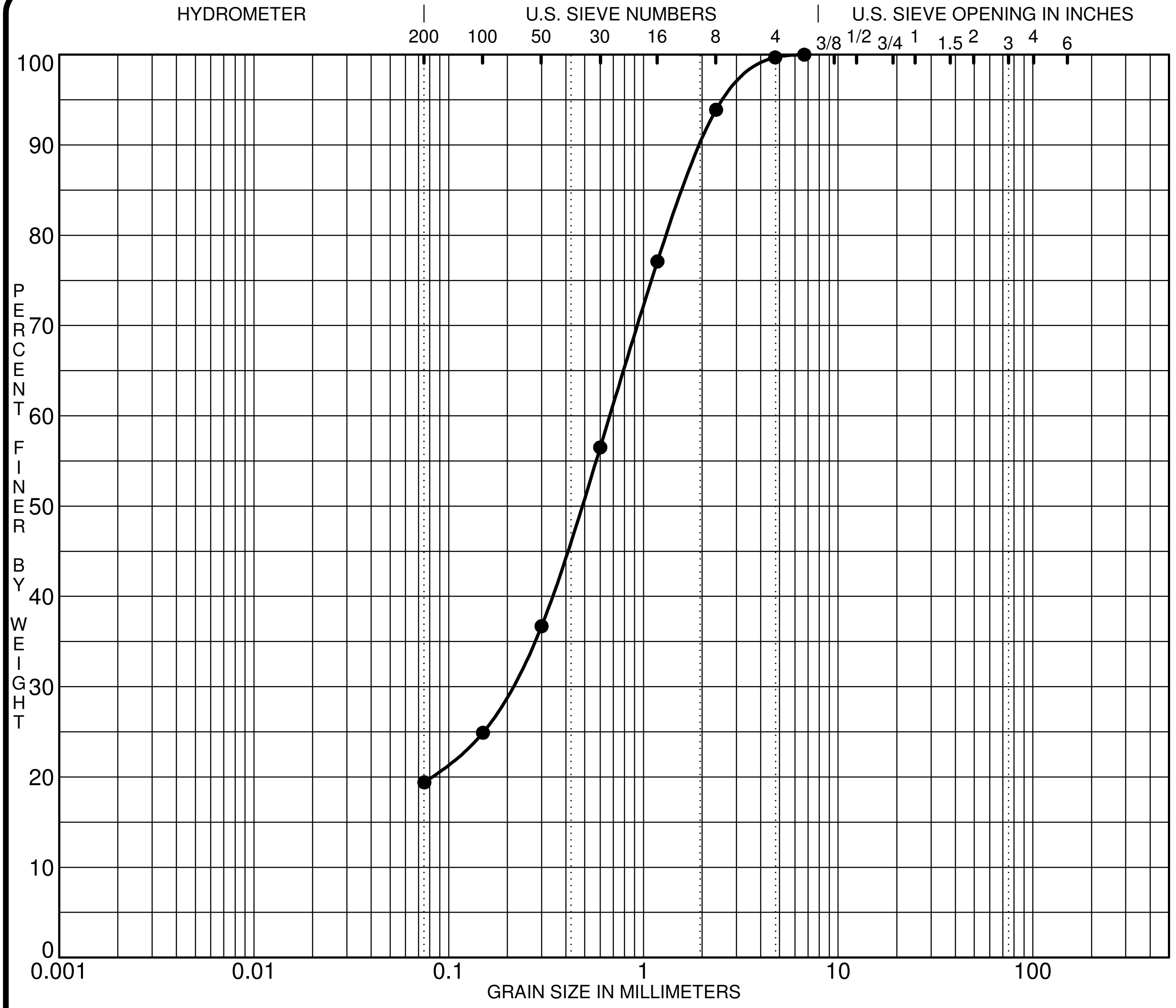
FILE NO. **PG5600**

DATE **2 Dec 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
● TP 3-20	**G 4**	**SILTY CLAYEY SAND**						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP 3-20	**G 4**	**6.70**	**0.67**	**0.202**		**0.3**	**80.3**	**19.4**	

CLIENT **Mattamy Homes**

FILE NO. **PG5600**

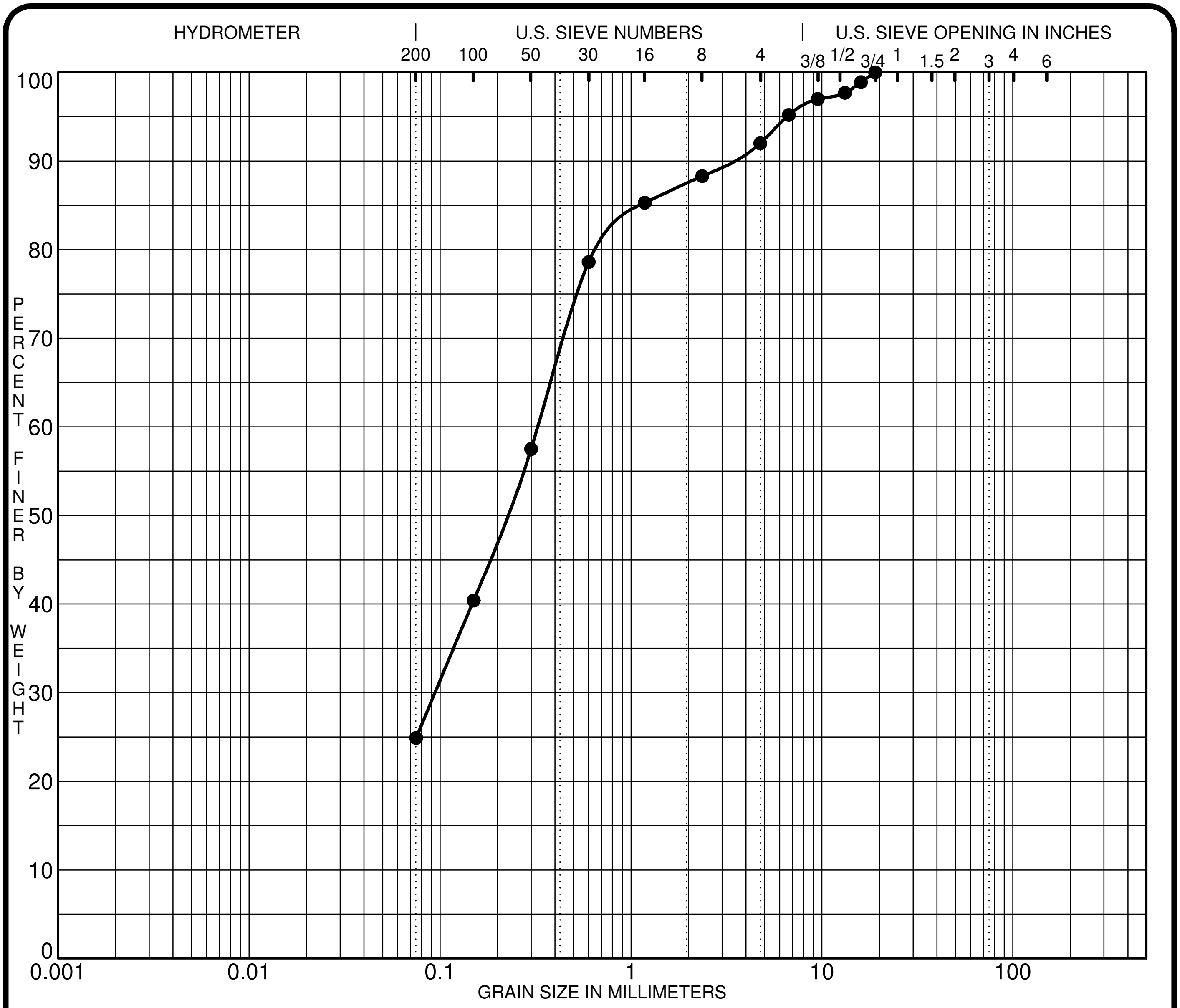
PROJECT **Geotechnical Investigation - Prop. Residential**
Development - 800 Cedarview Road

DATE **2 Dec 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
● **TP 6-20**	**G 2**	**SILTY SAND with gravel**						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **TP 6-20**	**G 2**	**19.00**	**0.33**	**0.094**		**8.0**	**67.1**	**24.9**	

CLIENT **Mattamy Homes**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 800 Cedarview Road**

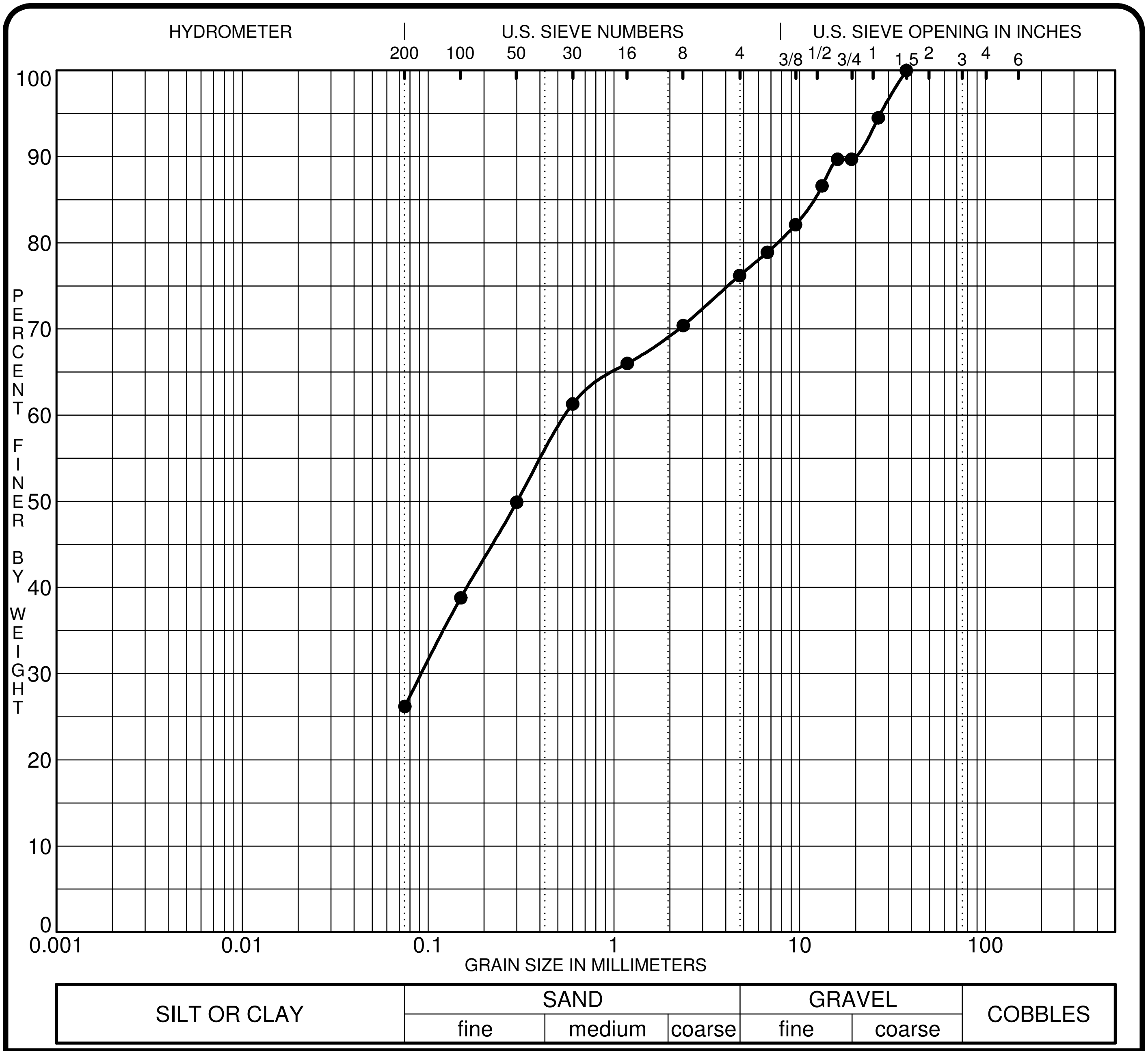
FILE NO. **PG5600**

DATE **2 Dec 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● **TP 6-20** **G 3**	**SILTY CLAYEY SAND with gravel**						

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **TP 6-20** **G 3**	**37.50**	**0.55**	**0.092**		**23.8**	**50.0**	**26.2**	

CLIENT **Mattamy Homes**

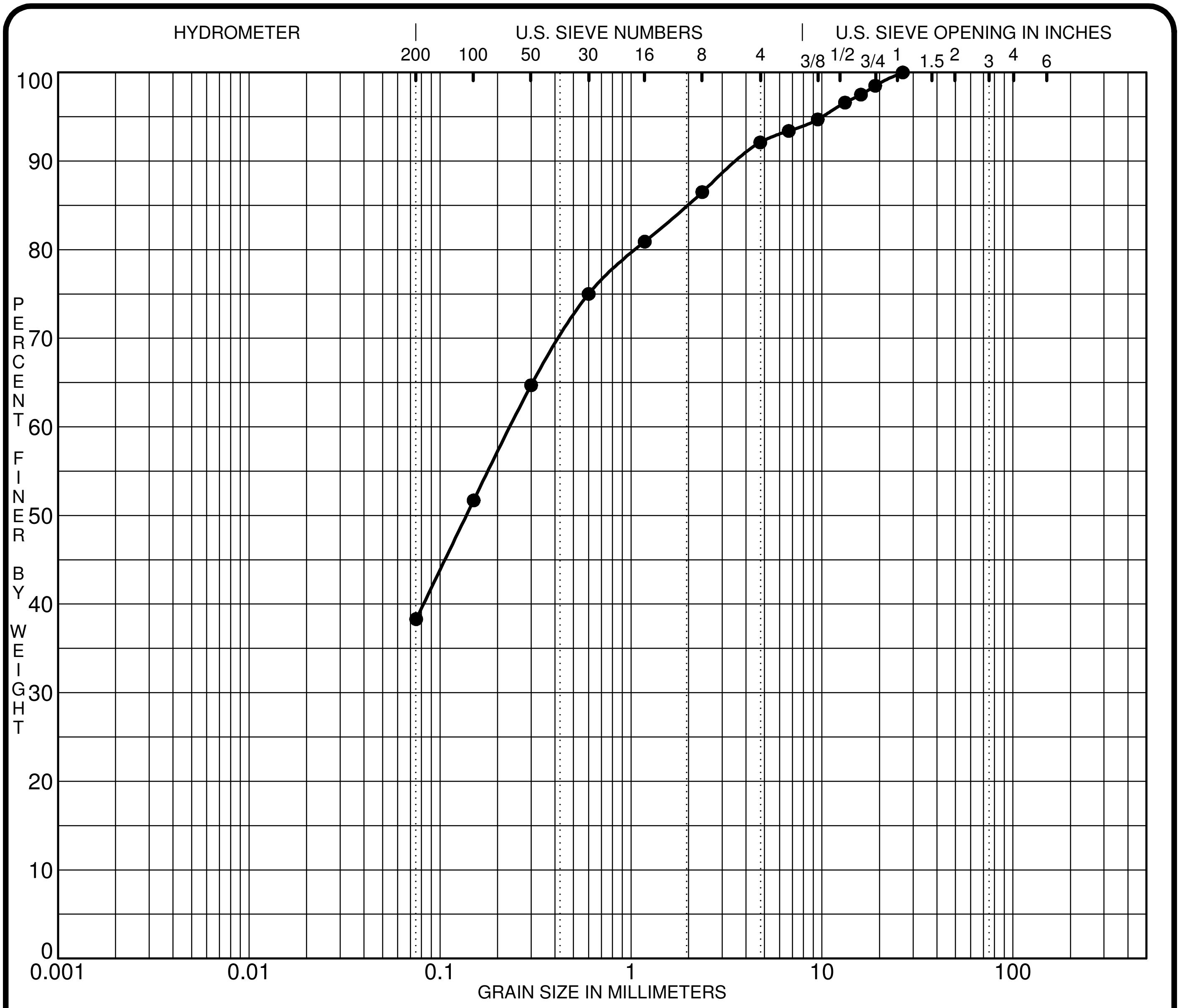
FILE NO. **PG5600**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 800 Cedarview Road**

DATE **2 Dec 20**

patersongroup **Consulting Engineers**
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**TP11-20**	**G 2**	**SILTY SAND with gravel**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP11-20**	**G 2**	**26.50**	**0.23**			**7.9**	**53.8**	**38.3**	

CLIENT **Mattamy Homes**

FILE NO. **PG5600**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 800 Cedarview Road**

DATE **2 Dec 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Report Date: 09-Apr-2024

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

Order Date: 3-Apr-2024

Client PO: 59841

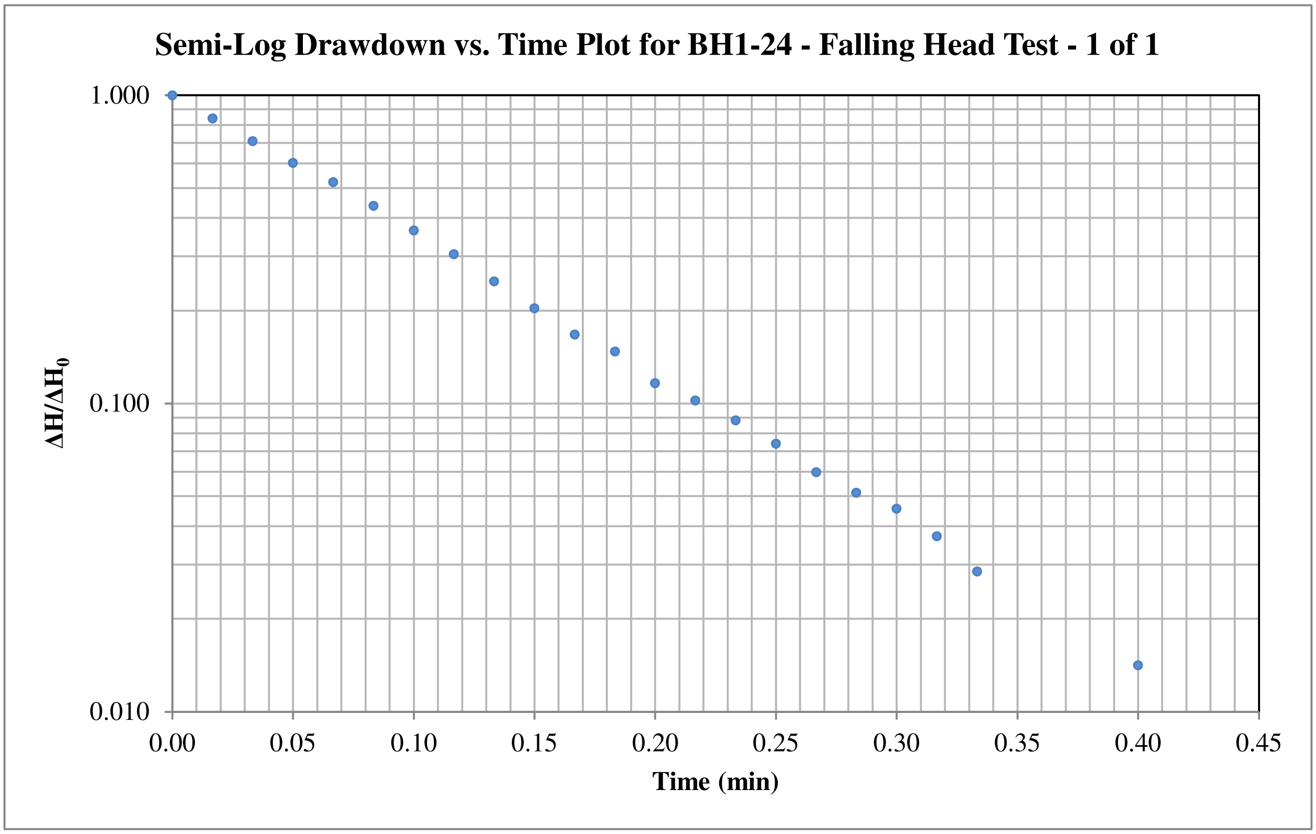
Project Description: PG5600

	Client ID:	BH1-24-SS3	-	-	-		
	Sample Date:	27-Mar-24 09:00	-	-	-	-	-
	Sample ID:	2414242-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	89.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.43	-	-	-	-	-
Resistivity	0.1 Ohm.m	78.9	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	14	-	-	-	-	-

Report: PG5600-1

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH1-24
Test: Falling Head - 1 of 1
Date: April 17, 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

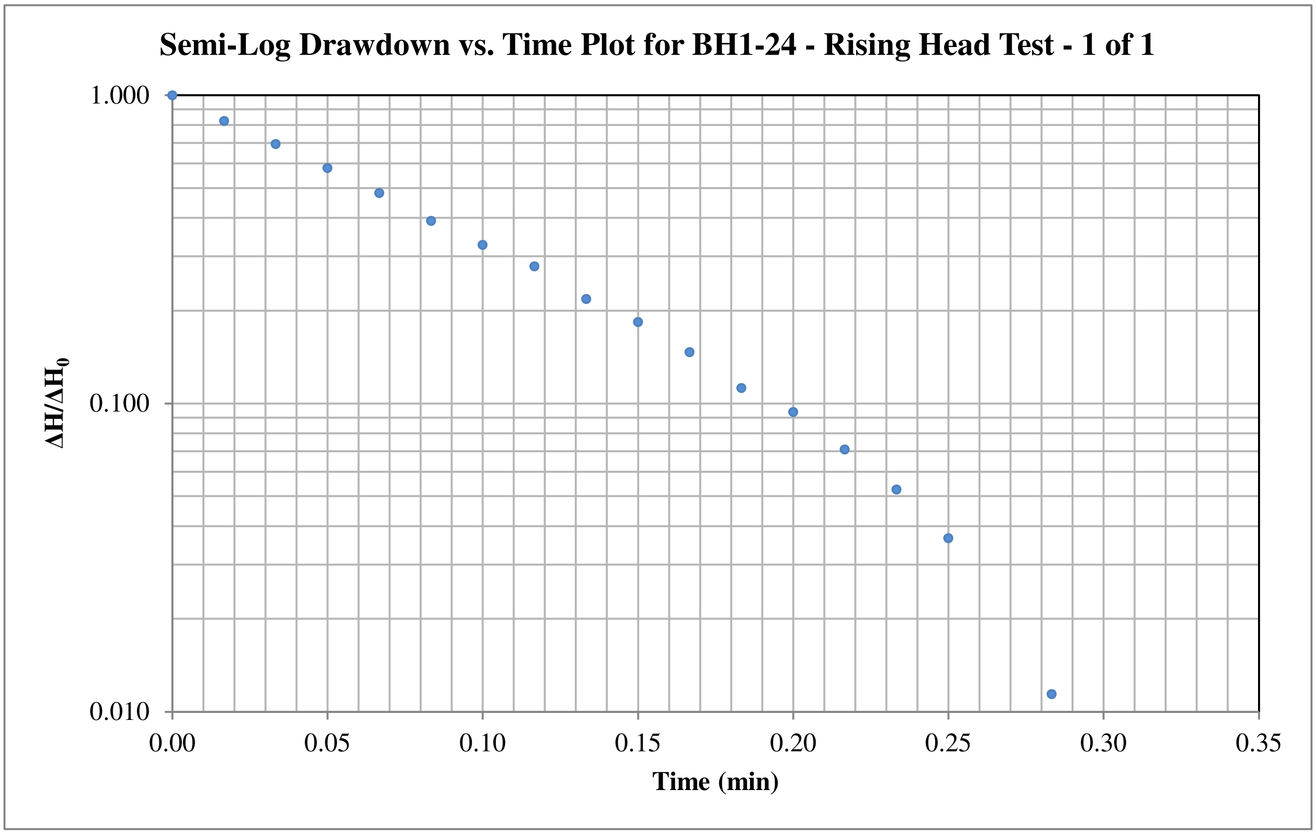
Data Points (from plot):

t*: 0.099 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 6.41E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH1-24
Test: Rising Head - 1 of 1
Date: April 17, 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

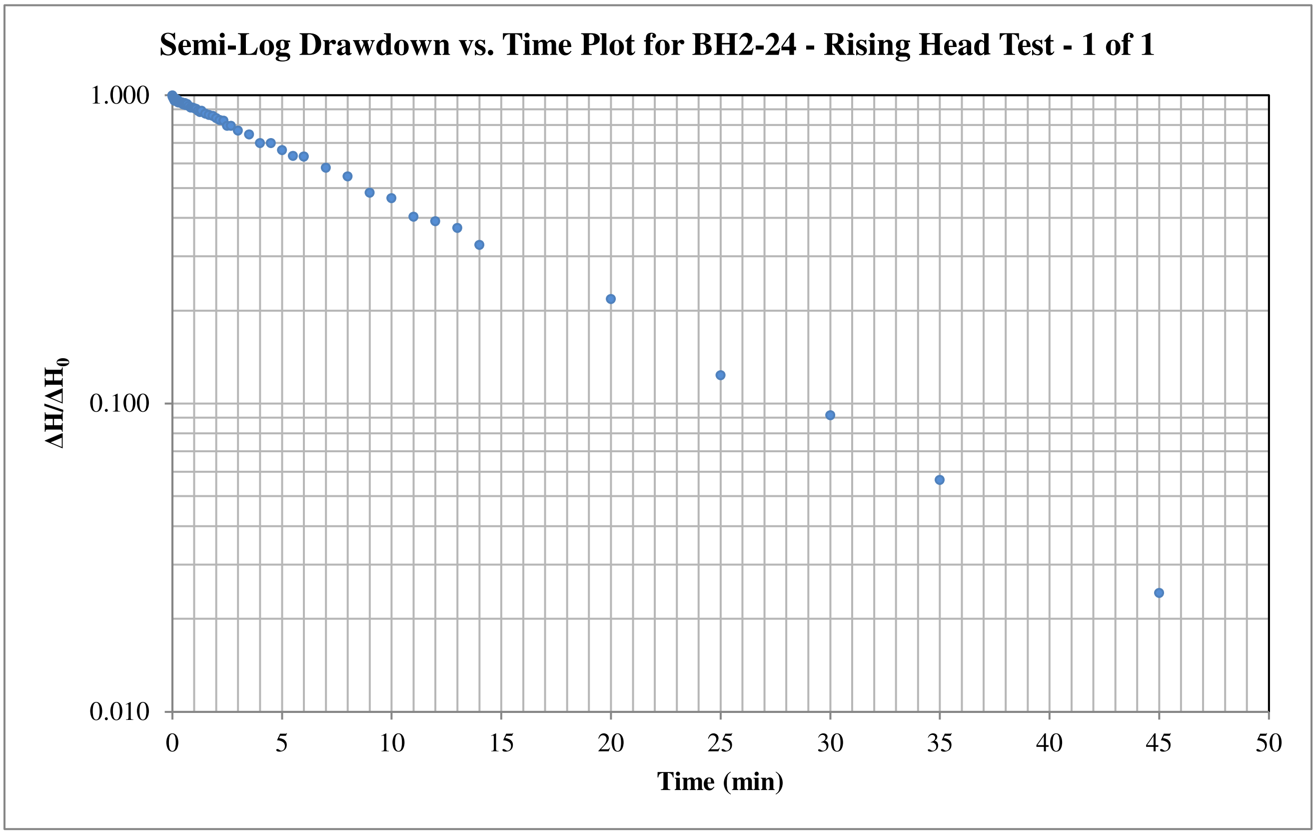
Data Points (from plot):

t*: 0.089 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 7.12E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH2-24
Test: Rising Head - 1 of 1
Date: April 17, 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

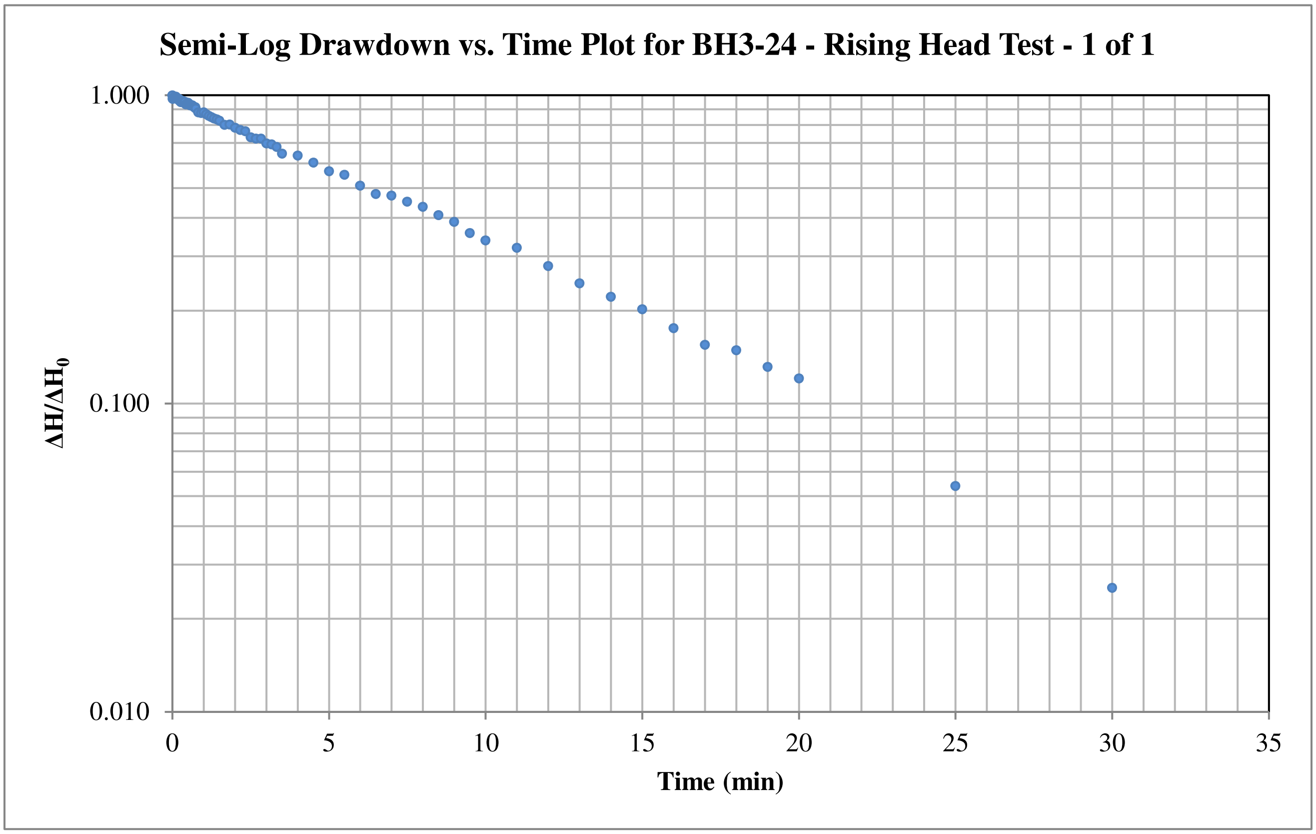
Data Points (from plot):

t*: 12.694 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.99E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH3-24
Test: Rising Head - 1 of 1
Date: April 15, 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: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

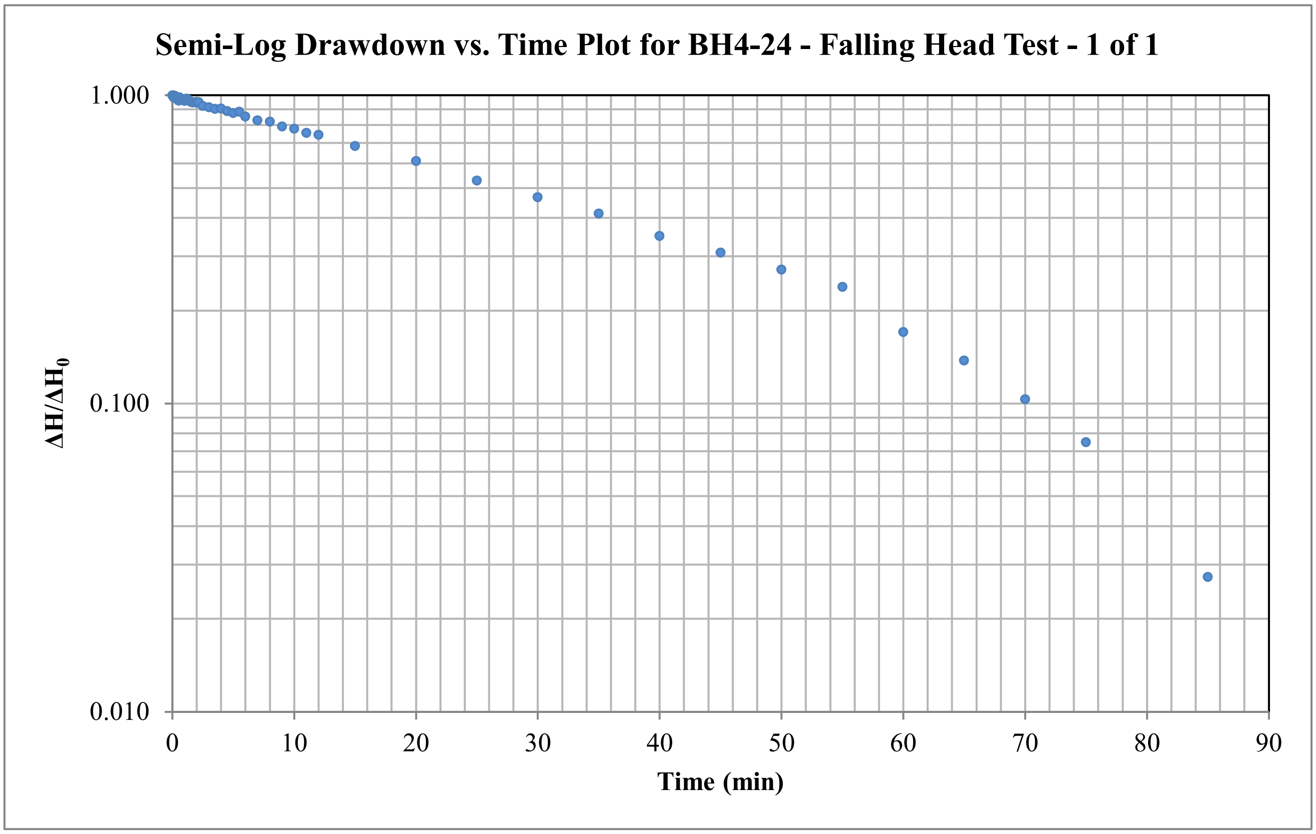
Data Points (from plot):

t*: 9.065 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.60E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH4-24
Test: Falling Head - 1 of 1
Date: April 17, 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

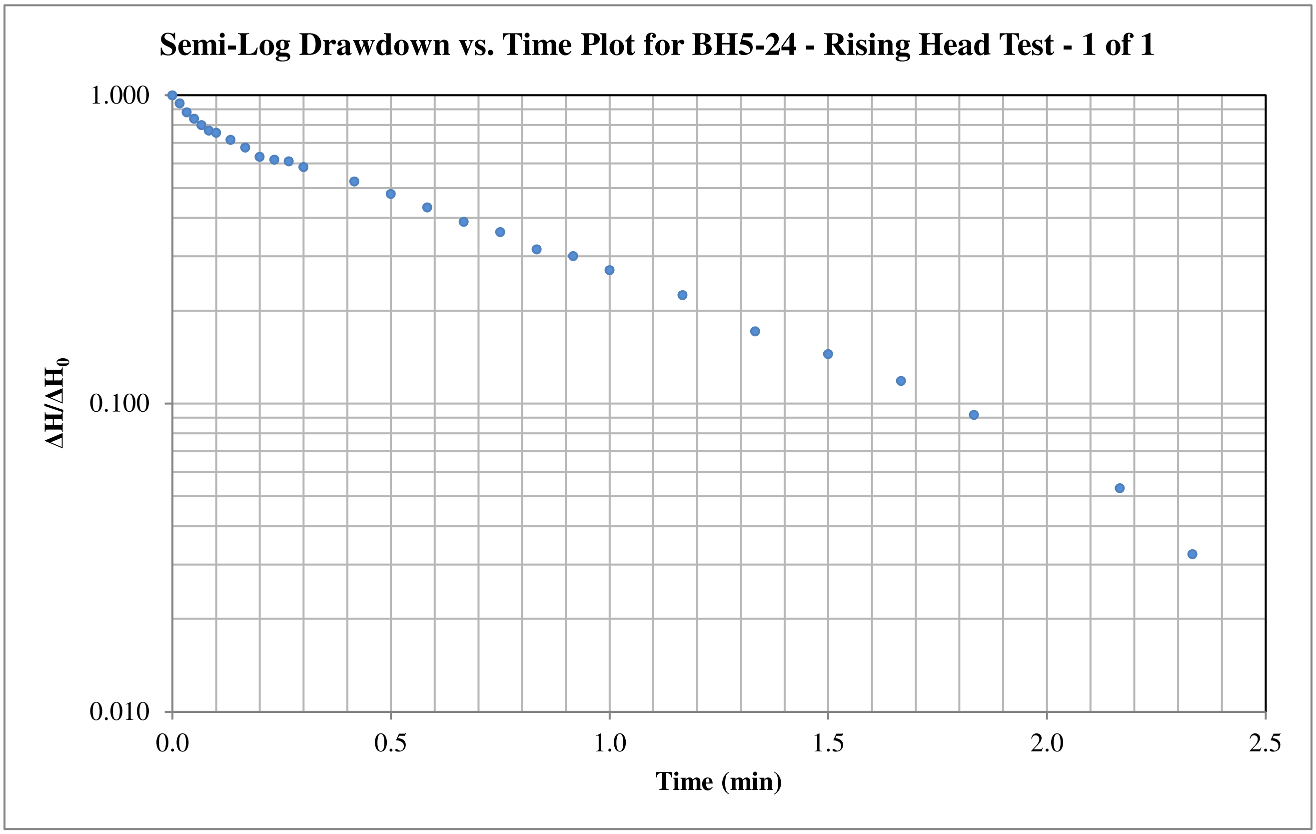
Data Points (from plot):

t*: 38.419 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.65E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH5-24
Test: Rising Head - 1 of 1
Date: April 17, 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: 1.99182

Well Parameters:

L	1.23	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

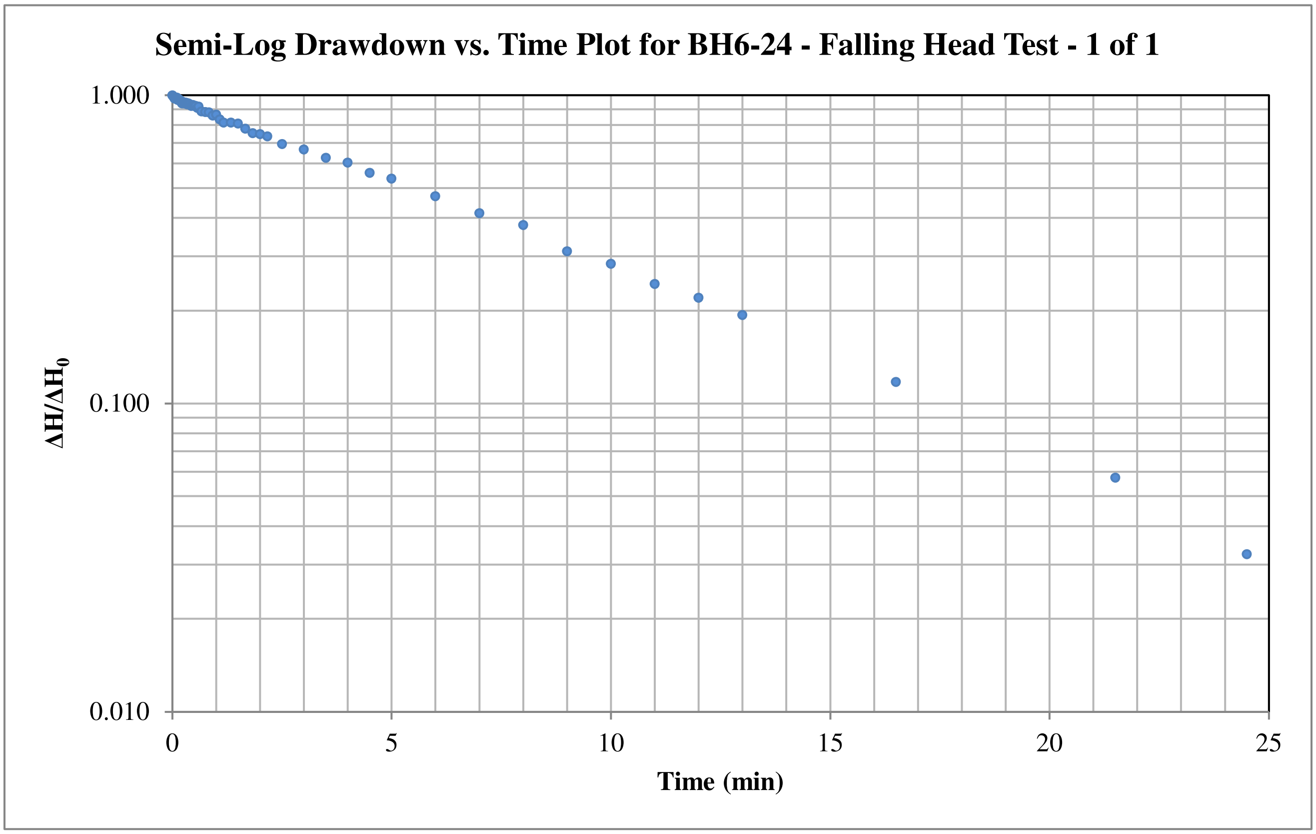
Data Points (from plot):

t*: 0.725 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.32E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH6-24
Test: Falling Head - 1 of 1
Date: April 15, 2024



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

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

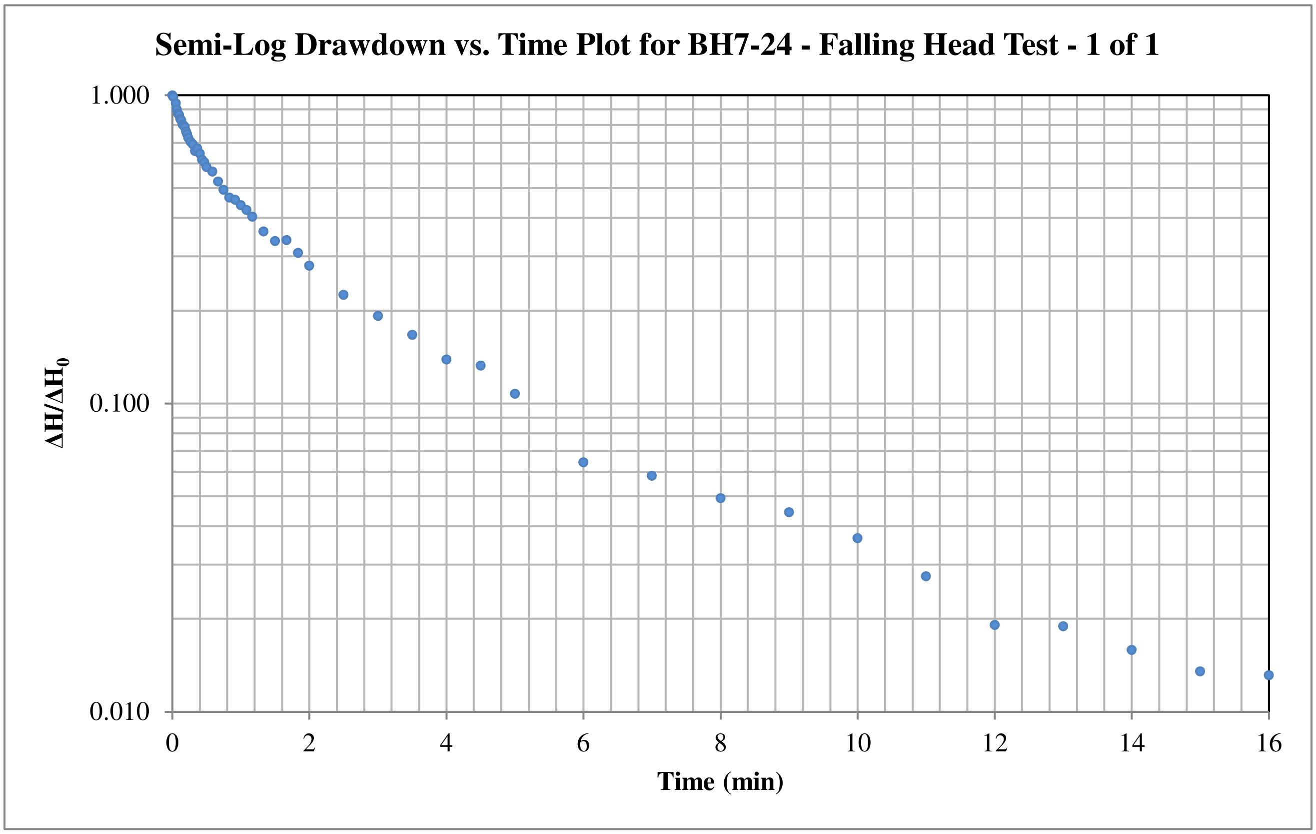
Data Points (from plot):

t*: 8.012 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 7.90E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH7-24
Test: Falling Head - 1 of 1
Date: April 15, 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

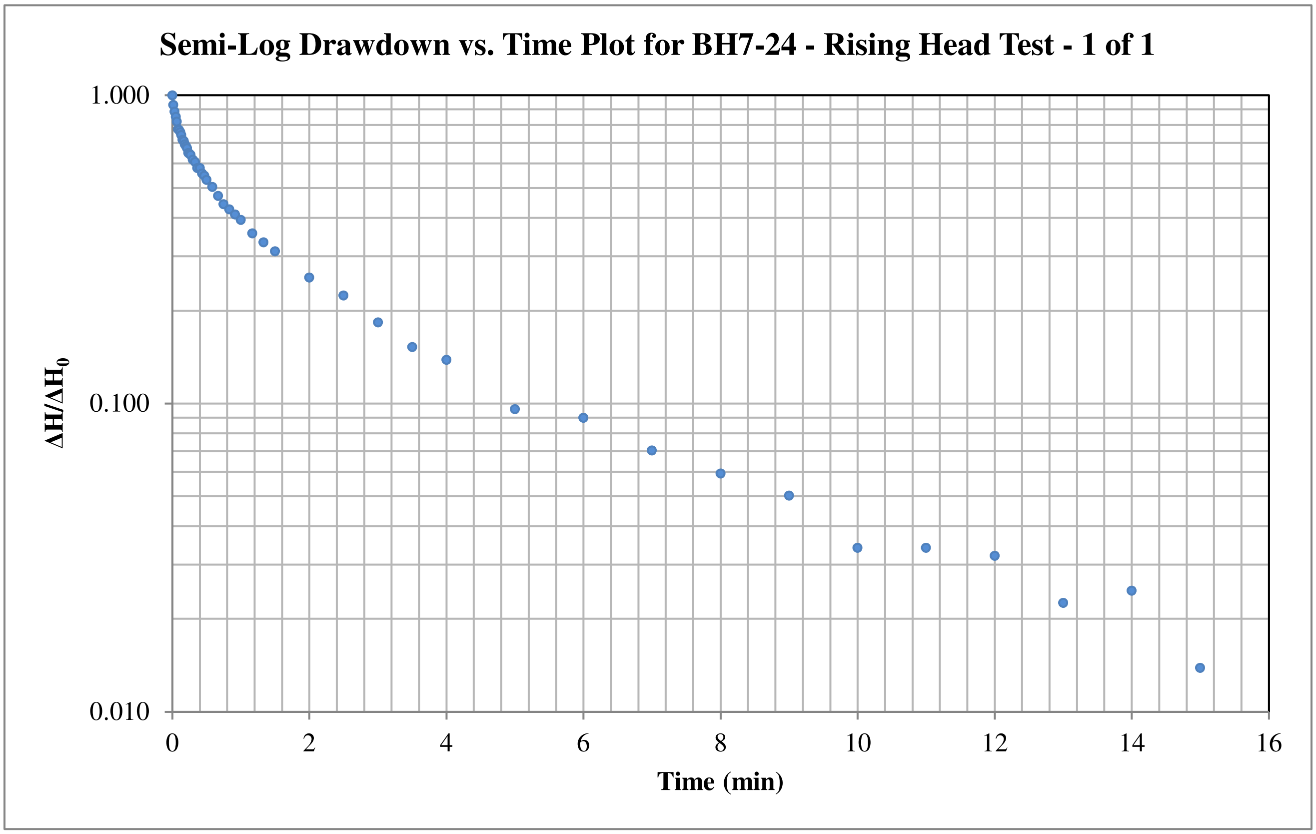
Data Points (from plot):

t*: 1.316 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.81E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH7-24
Test: Rising Head - 1 of 1
Date: April 15, 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

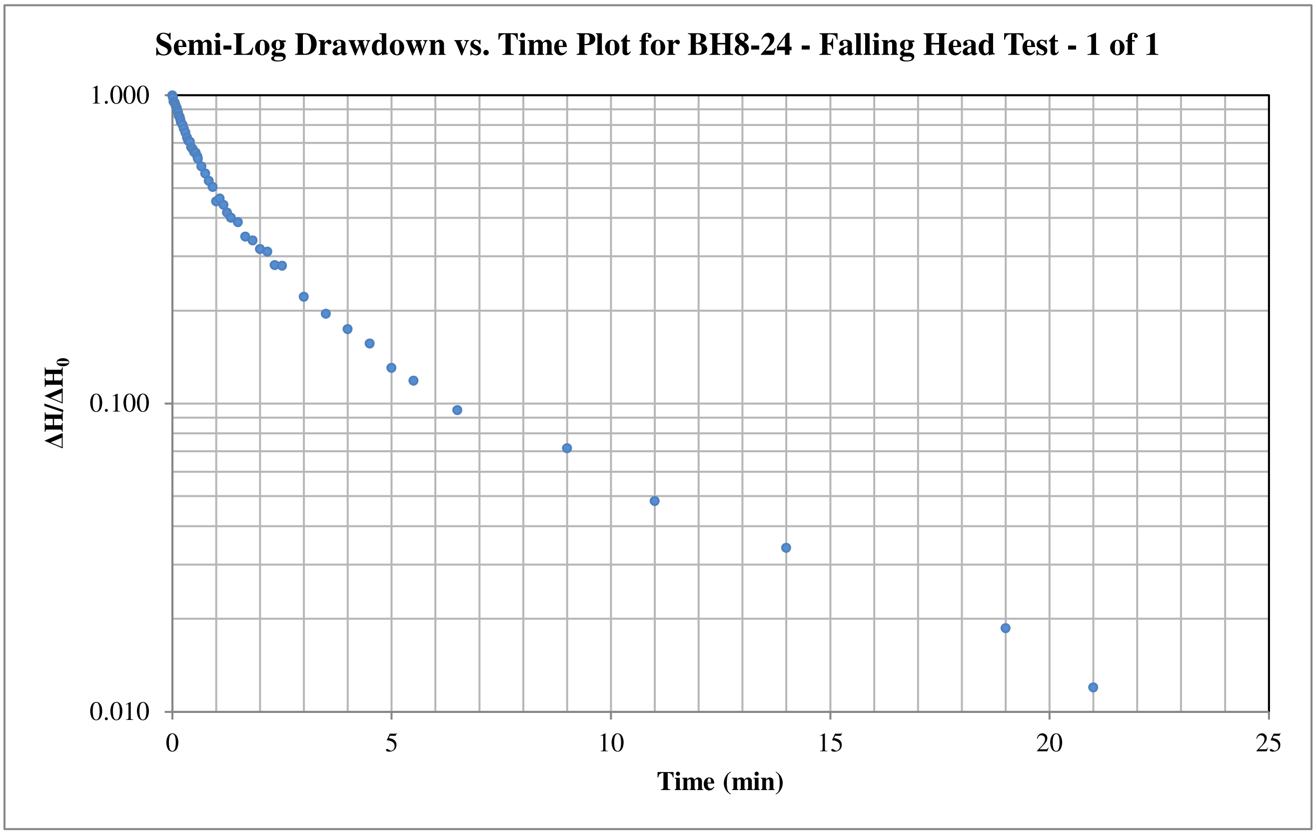
Data Points (from plot):

t*: 1.121 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.65E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Mattamy Homes - 800 Cedarview Road
Test Location: BH8-24
Test: Falling Head - 1 of 1
Date: April 15, 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*: 1.594 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.97E-06 m/sec

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 – AERIAL PHOTOGRAPH – 1965

FIGURE 3 – AERIAL PHOTOGRAPH – 1999

DRAWING PG5600-1 - TEST HOLE LOCATION PLAN

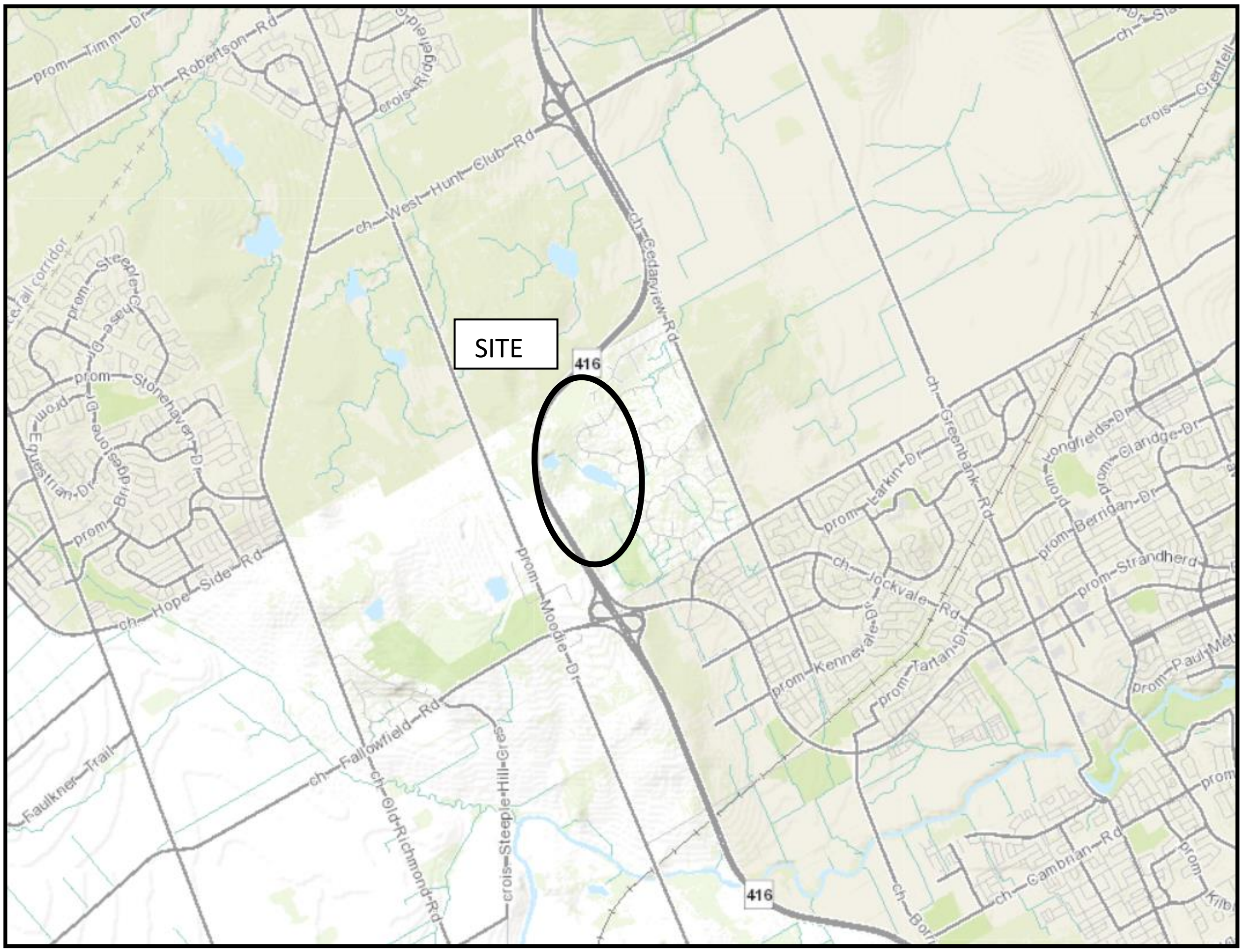


FIGURE 1

KEY PLAN

PATERSON GROUP

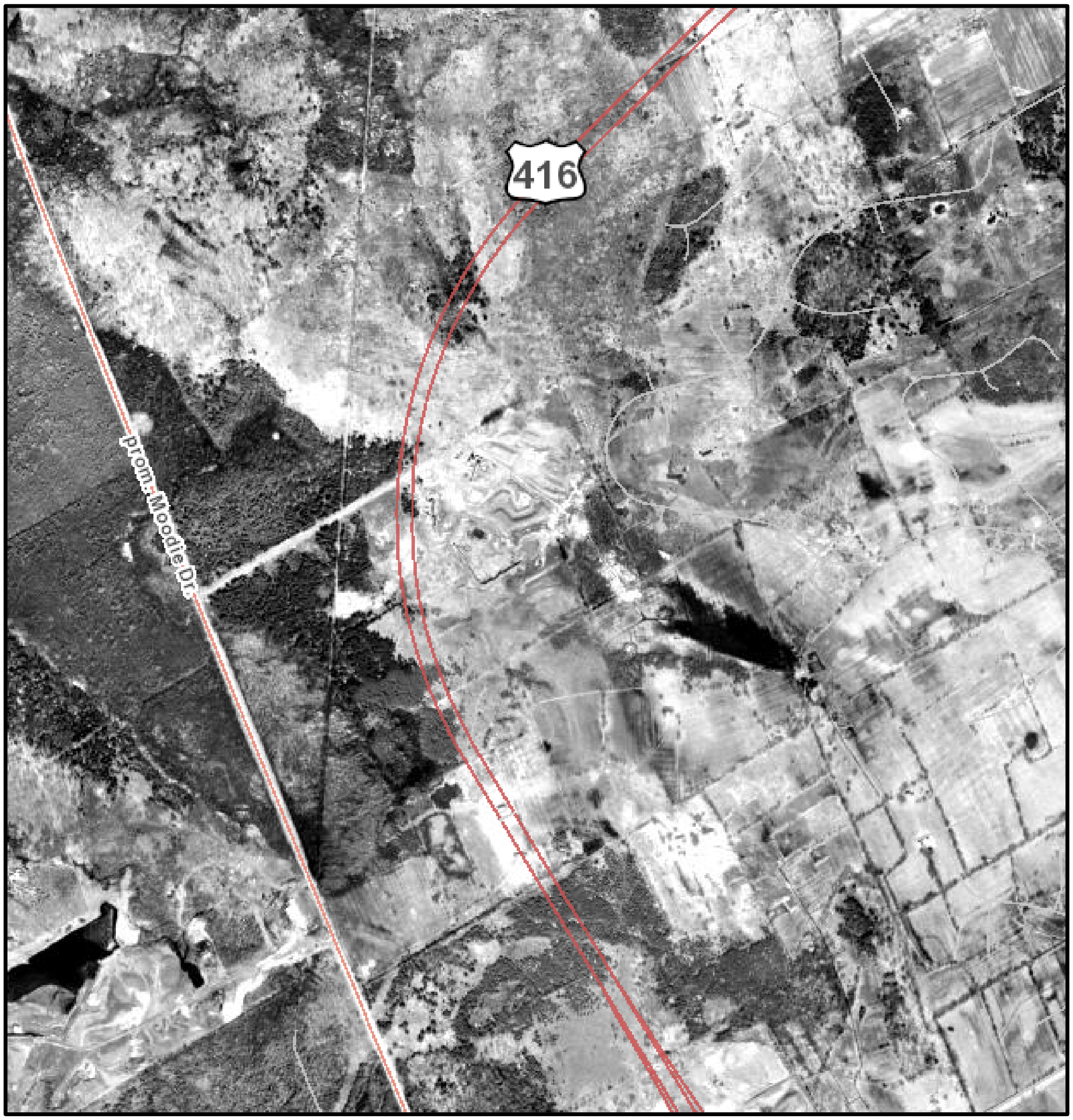
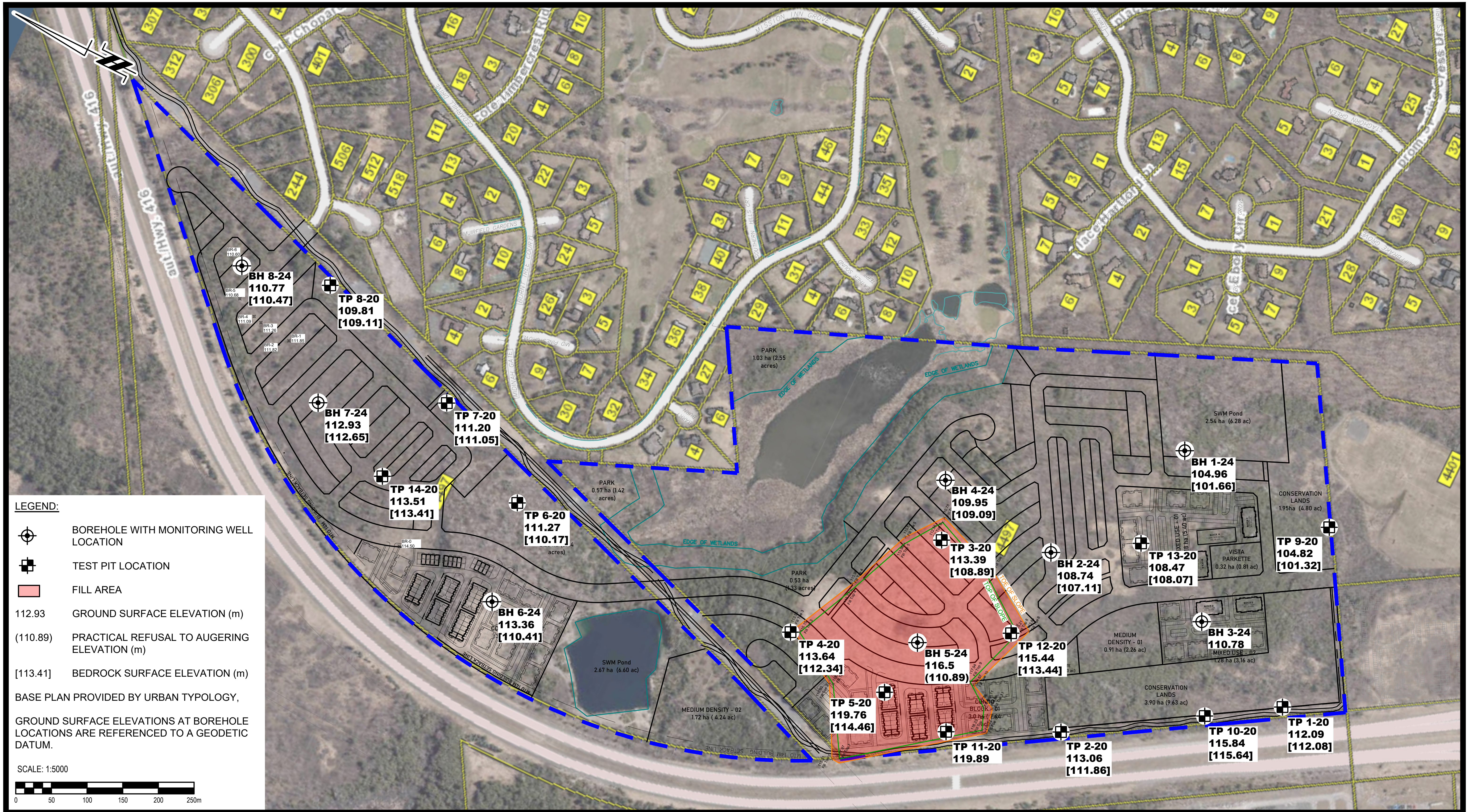


FIGURE 2

Aerial Photograph - 1965

FIGURE 3

Aerial Photograph - 1999



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
⊞	TEST PIT LOCATION
🟥	FILL AREA
112.93	GROUND SURFACE ELEVATION (m)
(110.89)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[113.41]	BEDROCK SURFACE ELEVATION (m)

BASE PLAN PROVIDED BY URBAN TYPOLOGY,

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

SCALE: 1:5000

0 50 100 150 200 250m

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED BASE PLAN AND ADDED BOREHOLES BH 1-24 TO BH 8-24	09\05\2024	SK

MATTAMY HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
800 CEDARVIEW ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:5000	Date: 12/2020
Drawn by: YA	Report No.: PG5600-1
Checked by: SK	Dwg. No.: PG5600-1
Approved by: KP	Revision No.: 1

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11x17