PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

East Parcel - 1770 Shea Road & 1820 Shea Road, Ottawa, Ontario

Prepared for Caivan (Stittsville South) Inc. & Caivan (Stittsville West) Ltd.

Report PG5570-3 dated June 3, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Caivan (Stittsville South) Inc. & Caivan (Stittsville West) Ltd. to conduct a geotechnical investigation for the proposed residential development to be located at 1770 Shea Road & 1820 Shea Road in the City of Ottawa (refer to Figure 1 – Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- ❒ Determine the subsoil and groundwater conditions at this site by means of 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.

The following report should be read in conjunction with the following Paterson Group reports:

- ❒ Hydrogeological Study and Water Budget Assessment Report PH4681-1 Revision 2 dated August 7, 2024
- ❒ Hydrogeological Existing Conditions Report PH4625-1 Revision 3 dated May 15, 2024.

2.0 Proposed Development

Based on available drawings, it is understood that the proposed development will consist of a series of low-rise single and townhouse style residential dwellings with associated driveways, local roadways and landscaped areas. A stormwater management facility is proposed to be located within the southeastern portion of the site. It is anticipated that the development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between December 14, 2021 and January 10, 2022. At that time, a total of fifteen (15) boreholes were advanced to a maximum depth of 10.2 m below the existing ground surface. The test holes were distributed in a manner to provide general coverage of the subject site taking into consideration site features. The test hole locations are shown on Drawing PG5570-4 – Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of drilling to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Rock core samples were recovered from boreholes BH22A-21, BH24-21 and BH33-21 drilled during the investigations using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

Groundwater

PVC monitoring wells and data loggers were installed at boreholes BH22A-21, BH24-21 and BH33-21 to permit long-term groundwater level measurement. The remaining boreholes were fitted with flexible piezometers to allow groundwater level monitoring following completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

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 PG5570-4 – 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. A total of three (3) grain size distribution tests were completed on selected soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution Results sheets presented in Appendix 1.

3.5 Permeameter Testing

In-situ permeameter testing was conducted using a Pask (Constant Head Well) Permeameter to confirm infiltration rates of the surficial soils at the subject site. At each location, two (2) 83 mm holes, located approximately 1.5 m away each other, were excavated using a Riverside/Bucket auger to approximate depths ranging from 0.3 to 0.6 m below the existing ground surface. All soils from the auger flights were visually inspected and initially classified on-site. The permeameter reservoir was filled with water and inverted into the hole, ensuring that it was relatively vertical and rested on the bottom of the hole. As the water infiltrated into the soil, the water level of the reservoir was monitored at various time intervals until the rate of fall 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 results of testing are further discussed in Subsection 4.2.

3.6 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at each monitoring well location. The testing was completed to assist in confirming anticipated groundwater flow rates within the subsoils and within the bedrock at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden and bedrock aquifers. The assumption regarding screen length and well diameter is considered to be met based on a screen length generally ranging from 1.5 to 3 m and a diameter ranging from 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. 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 locations are presented in Appendix 1.

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

4.0 Observations

4.1 Surface Conditions

The subject site generally consists of undeveloped, vacant land which was previously tree-covered, with tree removal occurring between 2017 and 2019. The site is generally flat with a gradual downward slope from the northwest to the southeast, and is slightly below grade of adjacent Flewellyn and Shea Roads. The subject site is bordered to the southeast by Flewellyn Road, to the northeast by Shea Road, to the northwest by a transmission line easement and residential dwellings, and to the southwest by a proposed residential development and existing residential dwellings.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil overlying a loose to compact, brown silty sand to sandy silt deposit, followed by very loose to very dense glacial till, underlain by bedrock. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and trace clay. A thin veneer of stiff, brown silty clay with some sand was observed in boreholes BH23-21 and BH26-21. The silty clay veneer was observed to extend to a maximum depth of 1.1 m below the existing ground surface.

Bedrock

Bedrock was cored in three boreholes to a maximum depth of 4.2 m below the bedrock surface, with an RQD value ranging from 73 to 100%. This is indicative of good to excellent quality bedrock. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Based on available geological mapping, the bedrock in this area consists of Paleozoic limestone of the Bobcaygeon Formation and an overburden drift thickness of 3 to 10 m depth.

Grain Size Distribution Testing

Grain size distribution testing (sieve analysis) was also completed on three select soil samples. The results of the grain size analysis are summarized in Table 1 and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis				
Test Hole Number	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt &Clay (%)**
BH19-21	SS2 + SS3	0.1	13.8	86.1
BH24-21	SS2 + SS3	4.9	46.3	48.8
BH37-21	SS3	0.0	64.2	35.8

Permeameter Testing Results

Paterson completed site specific infiltration testing on September 7, 2022 using a Pask Permeameter in order to identify the infiltration potential of the underlying soils on site. A total of 10 permeameter tests were conducted at 5 locations to provide general coverage of the subject site. Preparation and testing of this investigation are in accordance with the Canadian Standards Association (CSA) B65-12-Annex E. Field saturated hydraulic conductivity (K_{fs}) values and estimated infiltration values are presented in Table 2 on the following page.

Field saturated hydraulic conductivity values were determined using the Engineering Technologies Canada (ETC) Ltd. Reference tables provided in the most recent ETC Past Permeameter User Guide dated July 2018. Infiltration rates have been determined based on approximate relationships provided by the Ontario Ministry of Municipal Affairs and Housing – Supplementary Guidelines to the Ontario Building Code, 1997 - SG-6 – Percolation Time and Soil Descriptions.

Table 2 – Summary of Field Saturated Hydraulic Conductivity Values and Infiltration Rates						
Permeameter Test Location	**Ground Surface Elevation (m)**	**Depth of Permeamete Testing (m)**	**Elevation of Permeameter Testing (m)**	**K_{fs} (m/sec)**	**Unfactored Infiltration Rate (mm/hr)**	**Soil Type**
BH22-21	102.98	0.30	102.68	1.1×10^{-6}	47	Silty Sand
		0.60	102.38	1.6×10^{-6}	52	
BH23-21	102.38	0.30	102.08	5.3×10^{-7}	39	Silty Clay with Sand
		0.65	101.73	$\leq 8.1 \times 10^{-9}$	≤13	
BH26-21	103.04	0.30	102.74	1.1×10^{-7}	26	Silty Clay with Sand
		0.60	102.44	1.1×10^{-7}	26	
BH31-21	103.43	0.30	103.13	1.1×10^{-6}	47	Silty Sand to Sandy Silt
		0.60	102.83	1.4×10^{-7}	27	
BH37-21	103.54	0.30	103.24	5.3×10^{-6}	72	Silty Sand to Sandy Silt
		0.60	102.94	5.9×10^{-6}	74	
Note: Infiltration rates above do not include a safety correction factor.						

The measured field saturated hydraulic conductivity (K_{fs}) values within the test holes are consistent with similar material Paterson has encountered on other sites and typical published values for silty sand, sandy silt and silty clay which typically range from $1x10^{-4}$ to $1x10^{-6}$, $1x10^{-6}$ to $1x10^{-8}$, $1x10^{-7}$ to $1x10^{-9}$ m/sec, respectively. The range in K_{fs} values is generally due to the variability in composition and consistency of the material encountered. It is important to note that the infiltration rates derived from the K_{fs} values in the table above are unfactored, and that a factor of safety will need to be applied prior to being considered for design purposes.

Hydraulic Conductivity Values

Hydraulic conductivity (slug testing) values were recorded at each monitoring well location. The results are presented in Table 3 below.

Table 3 - Summary of Hydraulic Conductivity Values

Test Hole ID	Ground Surface Elevation (m)	Screened Interval (m)	K (m/sec)	Test Type	Soil Type/Bedrock
BH22A-21	102.98	7.2 - 10.2	$4.3x10^{-7}$	Falling Head	Bedrock
BH24-21	103.07	4.9 - 7.9	$6.0x10^{-5}$	Falling Head	Bedrock
			$7.3x10^{-5}$	Falling Head	
			$5.8x10^{-5}$	Rising Head	
			$5.7x10^{-5}$	Rising Head	
BH33-21	104.70	3.3 - 6.3	$1.6x10^{-4}$	Rising Head	Bedrock

The slug testing completed at the monitoring wells screened in bedrock identified hydraulic conductivity values ranging from approximately $4.3x10^{-7}$ to $1.6x10^{-4}$ m/sec. These values are generally consistent with similar material Paterson has encountered on other sites and typical published values for limestone bedrock, which typically range from $1x10^{-5}$ to $1x10^{-10}$ m/sec and is dependent on the quality of the bedrock at a given location.

4.3 Groundwater

The groundwater levels were manually recorded within the monitoring wells and piezometers installed at each borehole. Data loggers were installed in all monitoring wells to record seasonal fluctuations and precipitation collected within the upper portion of the subsurface profile across the site. The recorded groundwater levels are presented in Table 4 and are further noted on the Soil Profile and Test Data sheets in Appendix 1. The groundwater data recorded at the subject site to date is presented on Figures 5 to 7: Monitoring Well Water Elevations in Appendix 2.

Table 4 – Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH19-21	101.85	1.04	100.81	January 11, 2022
BH20-21	102.25	1.71	100.54	January 11, 2022
BH21-21	102.92	Blocked	N/A	January 11, 2022
BH22A-21*	102.98	2.49	100.49	January 11, 2022
		2.61	100.37	October 11, 2022
		1.77	101.21	April 4, 2023
		2.72	100.26	May 31, 2023
BH23-21	102.38	Blocked	N/A	January 11, 2022
BH24-21*	103.07	0.67	102.40	January 11, 2022
		0.60	102.47	October 11, 2022
		0.46	102.61	October 28, 2022
		-0.03	103.10	April 4, 2023
		0.74	102.34	May 31, 2023
BH25-21	102.73	0.71	102.02	January 11, 2022
BH26-21	103.04	0.78	102.26	January 11, 2022
BH27-21	102.71	0.84	101.87	January 11, 2022
BH28-21	101.85	1.79	100.06	January 11, 2022
BH30-21	102.44	1.62	100.82	January 11, 2022
BH31-21	103.43	1.27	102.16	January 11, 2022
BH33-21*	104.70	1.84	102.86	January 11, 2022
		2.12	102.58	October 11, 2022
		1.98	102.72	October 28, 2022
		1.20	103.51	April 4, 2023
		2.22	102.49	May 31, 2023
BH36-21	102.79	0.62	102.17	January 11, 2022
BH37-21	103.54	1.52	102.02	January 11, 2022

Notes:
-The ground surface elevation at each test hole location was surveyed using a handheld GPS and referenced to a geodetic datum
-* Denotes groundwater monitoring well

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered samples.

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 in the monitoring well water levels based upon seasonal fluctuations. The monitoring wells were equipped with a submersible datalogger (TD-Diver, VanEssen Instruments) to accurately monitor fluctuations in the water levels. The datalogger was programmed to continuously measure and record water levels at a fixed rate of one (1) reading every 24 hours.

The monitoring program was undertaken from October 2022 to May 2023. 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 monitoring data is presented in Figures 5 to 7 in Appendix 2.

Upon review of the datalogger readings and manual measurements, the groundwater readings measured within the monitoring wells and the piezometers across the subject site varied from an elevation of 100.06 m to a maximum elevation of 103.80 m, generally decreasing with the topography of the site. Based on our analysis of the measured groundwater levels and the data logger groundwater readings, seasonal groundwater in piezometers and the monitoring wells varied between 0.0 to 2.8 m below ground surface and 0.0 to 2.7 m, respectively.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is expected that the proposed residential buildings will be founded on conventional style footings placed on an undisturbed compact silty sand to sandy silt, stiff brown silty clay, compact to very dense glacial till, bedrock bearing surface, or engineered fill placed over an undisturbed compact silty sand to sandy silt, stiff silty clay, compact to very dense glacial till and/or bedrock bearing surface.

It is anticipated that bedrock removal may be required in localized areas across the site for building construction and service installation. All contractors should be prepared for bedrock removal within the subject site.

Due to the presence of a silty clay deposit within a portion of the site, settlement sensitive structures placed over the silty clay deposit will be subjected to a permissible grade raise restriction.

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 significant organic materials, or construction debris/remnants should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundations and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction debris should be excavated to a minimum of 1 m below final grade.

Proof-Rolling

Where loose or disturbed native soil is encountered at subgrade level, a proof-rolling program should be implemented, consisting of compacting the loose material with several passes of a vibratory drum roller under dry conditions and above freezing temperatures, and under the observation of Paterson. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill. Provisions should be made for a proof rolling program at the subject site.

Bedrock Removal

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

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

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 structures.

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

Excavation side slopes in sound bedrock can be excavated almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipment. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency.

For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines.

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

Fill Placement

Fill placed for grading beneath the building areas 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 imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

To in-fill existing channels/ditches below building areas, roadways or other settlement sensitive structures, it is recommended to place Granular A, Granular B Type I or II, well graded blast rock (maximum 200 mm diameter) or select subgrade material. The backfill material should be placed under dry conditions, in above freezing temperatures and approved by the geotechnical consultant. The backfill should be placed in maximum 300 mm loose lifts and compacted to 98% of its 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 with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. 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. This material should be used structurally only to build up the subgrade for pavements.

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.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Bearing resistance values are provided in Table 5 for footings placed on an undisturbed stiff, brown silty clay, compact silty sand to sandy silt, compact to very dense glacial till or clean bedrock bearing surface.

Table 5 – Bearing Resistance Values

Bearing Surface	Factored Bearing Resistance Value at ULS (kPa)	Bearing Resistance Value at SLS or Allowable Bearing Pressure (kPa)
Stiff, Brown Silty Clay	150	100
Compact Silty Sand to Sandy Silt	250	150
Compact to Very Dense Glacial Till	250	150
Engineered Fill (Granular A or Granular B Type II)	250	150
Clean Surface Sounded Bedrock	1000	-

Notes:
- The bearing resistance value provided for stiff, brown silty clay is limited to strip footings up to 3 m wide and pad footings up to 5 m wide.
- A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

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

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

Footings designed using the bearing resistance values at SLS provided in Table 5 will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. Footings placed on clean, surface sounded bedrock will be subjected to negligible 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 an undisturbed soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V, passing through in situ soil of the same or higher capacity as the bearing medium soil.

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 bearing medium 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. 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.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class X_C** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024 for foundations considered at this site.

A higher seismic class may be applicable, such as Class X_A or X_B, provided the footings are within 3 m of the bedrock surface. However, this would need to be confirmed by performing a site-specific seismic shear wave velocity test at the subject site. The soils underlying the site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction. Provision should be made for proof rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m3.However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using 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.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case. Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 site area is 0.30 g 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

Car only parking areas, access and heavy traffic access areas are expected at this site. The subgrade material is anticipated to consist of silty sand to sandy silt, glacial till, compacted engineered fill or bedrock. The proposed pavement structures are presented in Tables 6, 7 and 8.

Table 6 – 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, or bedrock.	

Table 7 – Recommended Pavement Structure – Local and Collector Roadways Without Bus Traffic	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
450	**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, or bedrock.	

Table 8 – 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 Asphaltic Concrete
50	**Lower Binder Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
600	**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, or bedrock.	

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

For residential driveways and car only parking areas, an Ontario Traffic Category A will be used. For local and collector roadways, an Ontario Traffic Category B should be used for design purposes. For roadways with bus traffic, an Ontario Traffic Category D should be used for design purposes.

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 compaction equipment.

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, 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. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

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

Frost Susceptibility of Bedrock

When bedrock is encountered above the proposed founding depth and soil frost cover is less than 1.5 m, the frost susceptibility of the bedrock should be determined.

This can be accomplished as follows:

- ❒ Drill supplemental coreholes within the bedrock in the vicinity of the foundations and assess the frost susceptibility.

- ❒ Examine service trench profiles extending in the bedrock in the vicinity of the foundations to determine if weathering is extensive.

If the bedrock is considered to be **non-frost susceptible**, the footings can be poured directly on the bedrock without any further frost protective measures.

If the bedrock is considered to be **frost susceptible**, the following measures should be implemented for frost protection:

- ❒ Option A – Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

- ❒ Option B – Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to potential undulating of the bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weathered bedrock.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient room will be available for the greater part of the excavation to be undertaken by open- cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

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

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

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the department of public works and services, infrastructure services branch of the City of Ottawa.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material for areas over a soil subgrade. However, the bedding thickness should be increased to 300 mm for areas over a bedrock subgrade, if encountered. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe. Reference should be made to drawings - OPSD 802.030, OPSD 802.031 & OPSD 802.033 for Rigid Pipe Bedding, Cover and Backfill on Type 1, 2 or 3 soil and bedrock excavation.

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

Generally, it should be possible to re-use the moist (not wet) silty sand to sandy silt and glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet sub-excavated soil should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. All stones greater than 300 mm in their greatest dimension should be removed prior to reuse of site-generated glacial till.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of the soils exposed at the trench walls to minimize differential frost heaving.

The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package 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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

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

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

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

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

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and/or detailed designs of the proposed development have been prepared:

- ❑ Review detailed grading, servicing and landscaping plan(s) from a geotechnical perspective.

- ❑ Review of bedrock excavation activities (if applicable).

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

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

- ❒ Observation of all bearing surfaces prior to the placement of concrete.

- ❒ Sampling and testing of the concrete and fill materials used.

- ❒ 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.

All excess soils generated by construction activities should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

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.

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 Caivan (Stittsville South) Inc. & Caivan (Stittsville West) 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.

LICENSED PROFESSIONAL ENGINEER
June 3, 2025
K. A. PICKARD
100531344
PROVINCE OF ONTARIO

Owen R. Canton, B.Eng.

Kevin A. Pickard, P.Eng.

Michael Killam, P.Eng.

Report Distribution:

- ❑ Caivan (Stittsville South) Inc. & Caivan (Stittsville West) Ltd. (Digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 16, 2021

FILE NO. PG5570

HOLE NO. BH19-21

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Compact, brown **SILTY SAND to SANDY SILT**	1.73	SS	2	50	12
Loose, brown **SILTY SAND,** some clay	2.59	SS	3	75	8
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders	2.84	SS	4	100	12

DEPTH (m)	ELEV. (m)
0	101.85
1	100.85
2	99.85

End of Borehole

Practical refusal to augering at 2.84m depth

(GWL @ 1.04m - Jan. 11, 2022)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 17, 2021

FILE NO. PG5570

HOLE NO. BH20-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	102.25		
TOPSOIL 0.23									
Compact to loose, brown **SILTY SAND to SANDY SILT**		AU	1						
		SS	2	58	11	1	101.25		
2.44		SS	3	42	7	2	100.25		
Interlayered grey **SANDY SILT** and grey **SILTY CLAY** 3.20		SS	4	75	3	3	99.25		
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders		SS	5	67	23				
4.19		SS	6		50+	4	98.25		

End of Borehole

Practical refusal to augering at 4.19m depth

(GWL @ 1.71m - Jan. 11, 2022)

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 17, 2021

FILE NO. PG5570

HOLE NO. BH21-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	102.92		
TOPSOIL 0.25		AU	1						
Loose, brown **SILTY SAND to SANDY SILT** 1.07		SS	2	42	36	1	101.92		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders 2.23		SS	3	50	71	2	100.92		

End of Borehole

Practical refusal to augering at 2.23m depth

(Piezometer damaged - Jan. 11, 2022)

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2021

FILE NO. PG5570

HOLE NO. BH22-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	102.98		
TOPSOIL 0.20									
Loose, brown **SILTY SAND,** trace gravel 0.69		AU	1						
		SS	2	100	22	1	101.98		
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders		SS	3	92	29	2	100.98		
		SS	4	83	46	3	99.98		
3.48		SS	5	50	50+				
End of Borehole Practical refusal to augering at 3.48m depth.									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 10, 2022

FILE NO. PG5570

HOLE NO. BH22A-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20				
Loose, brown **SILTY SAND,** trace gravel	0.69	AU	1		
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	5.97	SS	2	100	22
		SS	3	92	29
		SS	4	83	46
		SS	5	50	50+
		RC	1	77	
		RC	2	14	
BEDROCK: Excellent quality, grey dolostone interbedded with grey limestone	10.21	RC	3	100	94
		RC	4	100	100
		RC	5	100	100
End of Borehole					
(GWL @ 2.49m - Jan. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	102.98
1	101.98
2	100.98
3	99.98
4	98.98
5	97.98
6	96.98
7	95.98
8	94.98
9	93.98
10	92.98

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2021

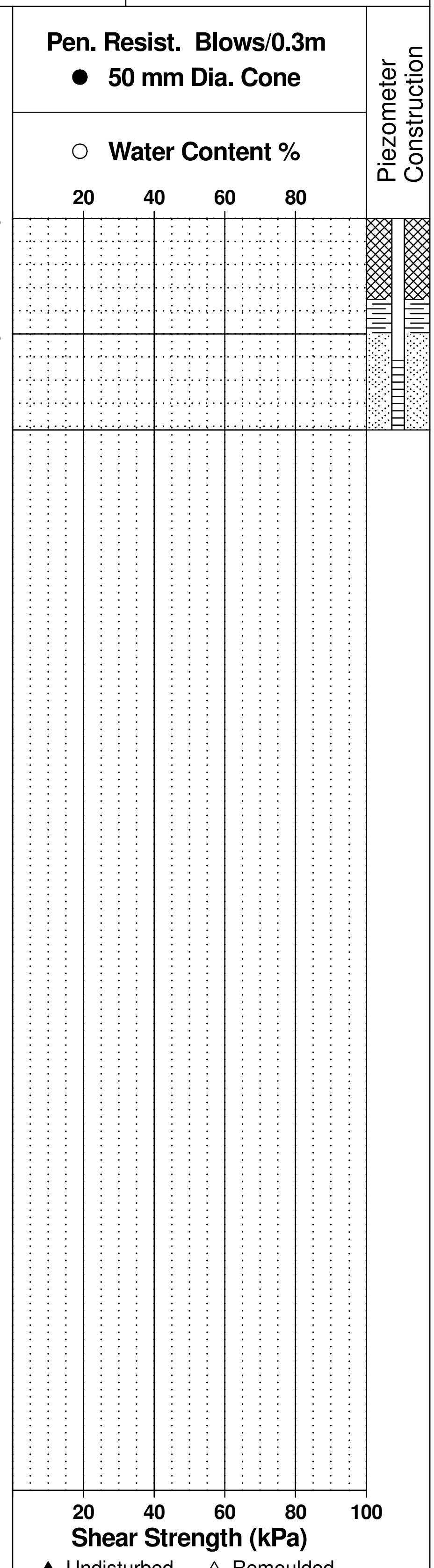
FILE NO. PG5570

HOLE NO. BH23-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.38
TOPSOIL 0.28							
Stiff, brown **SILTY CLAY,** some sand 1.12		AU	1				
		SS	2	25	32	1	101.38
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay 1.83		SS	3	55			
End of Borehole							

Practical refusal to augering at 1.83m depth

(Piezometer damaged - Jan. 11, 2022)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2021

FILE NO. PG5570

HOLE NO. BH24-21

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30				
Loose to dense, brown **SILTY SAND to SANDY SILT**	1.83	AU	1		
		SS	2	58	8
		SS	3	75	32
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders - boulders cored from 2.46 to 4.42m depth	4.42	SS	4	50	50+
		RC	1	100	
		RC	2	19	
BEDROCK: Good to excellent quality, grey limestone interbedded with dolostone - 15mm thick mud seam at 5.25m depth	7.92	RC	3	100	81
		RC	4	100	100
		RC	5	100	100
End of Borehole (GWL @ 0.67m - Jan. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	103.07
1	102.07
2	101.07
3	100.07
4	99.07
5	98.07
6	97.07
7	96.07

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH25-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	102.73		
TOPSOIL 0.25									
Loose, brown **SILTY SAND,** trace clay and gravel		AU	1						
1.17		SS	2	71	50+	1	101.73		
End of Borehole									
Practical refusal to augering at 1.17m depth									
(GWL @ 0.71m - Jan. 11, 2022)									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH26-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.04
TOPSOIL 0.25							
Stiff, brown **SILTY CLAY,** some sand 0.69		AU	1				
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	2	40	50+	1	102.04
		SS	3	61	33	2	101.04
2.16 End of Borehole							

End of Borehole

Practical refusal to augering at 2.16m depth

(GWL @ 0.78m - Jan. 11, 2022)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH27-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.71
TOPSOIL 0.28							
Compact to loose, brown **SILTY SAND to SANDY SILT,** trace clay		AU	1				
		SS	2	67	19	1	101.71
		SS	3	83	21	2	100.71
- grey by 2.4m depth		SS	4	50	9		
3.12						3	99.71
GLACIAL TILL: Very loose, grey silty sand with gravel, cobbles and boulders 3.43		SS	5	86	3		

End of Borehole

Practical refusal to augering at 3.43m depth

(GWL @ 0.84m - Jan. 11, 2022)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH28-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.85
TOPSOIL 0.30		AU	1				
Loose, brown **SILTY SAND to SANDY SILT,** trace clay		SS	2	42	7	1	100.85
2.21		SS	3	58	8	2	99.85
Interbedded layers of grey **SILTY SAND** and grey **SILTY CLAY** 2.97		SS	4	100	2	3	98.85
GLACIAL TILL: Very loose, grey silty sand with clay, gravel and cobbles		SS	5	100	3		
3.89		SS	6		50+		
End of Borehole Practical refusal to augering at 3.89m depth (GWL @ 1.79m - Jan. 11, 2022)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH30-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	102.44		
TOPSOIL 0.25									
Compact to very loose, brown **SILTY SAND to SANDY SILT,** trace clay - grey by 2.0m depth		AU	1						
		SS	2	50	12	1	101.44		
		SS	3	33	10				
						2	100.44		
		SS	4	92	1				
3.45		SS	5	83	2	3	99.44		
GLACIAL TILL: Very loose to compact, grey silty sand with gravel, cobbles and boulders, trace clay		SS	6	33	24	4	98.44		
4.82		SS	7	50	50+				
End of Borehole Practical refusal to augering at 4.82m depth (GWL @ 1.62m - Jan. 11, 2022)									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 21, 2021

FILE NO. PG5570

HOLE NO. BH31-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.43
TOPSOIL (0.36)							
Compact to loose, brown **SILTY SAND to SANDY SILT,** trace clay		AU	1				
		SS	2	50	14	1	102.43
		SS	3	50	22	2	101.43
		SS	4	42	9	3	100.43
- grey by 3.2m depth		SS	5	58	5		
(4.72)		SS	6	42	12	4	99.43
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders		SS	7	58	37	5	98.43
(6.12)		SS	8		58	6	97.43
End of Borehole		SS	9	0	50+		

Practical refusal to augering at 6.12m depth

(GWL @ 1.27m - Jan. 11, 2022)

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

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 22, 2021

FILE NO. PG5570

HOLE NO. BH33-21

SOIL DESCRIPTION	DEPTH TO BASE (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Compact, brown **SILTY SAND,** trace clay and organics	1.45	AU	1		
		SS	2	50	13
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders	2.74	SS	3	8	11
		RC	1	30	
BEDROCK: Good to excellent quality, grey limestone - 25mm thick mud seam at 3.7m depth - 30mm thick mud seam at 3.8m depth	6.27	RC	2	100	73
		RC	3	95	85
		RC	4	100	100
End of Borehole (GWL @ 1.84m - Jan. 11, 2022)					

DEPTH (m)	ELEV. (m)
0	104.70
1	103.70
2	102.70
3	101.70
4	100.70
5	99.70
6	98.70

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 7, 2022

FILE NO. PG5570

HOLE NO. BH36-21

SOIL DESCRIPTION	Depth to base (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	102.79
TOPSOIL	0.30							
Compact, brown **SILTY SAND to SANDY SILT**	1.45		AU	1				
			SS	2	42	15	1	101.79
GLACIAL TILL: Very dense to compact, brown silty sand with gravel, cobbles and boulders	2.90		SS	3	60	50+		
							2	100.79
			SS	4	8	15		

End of Borehole

Practical refusal to augering at 2.90m depth.

(GWL @ 0.62m - Jan. 11, 2022)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6115 Flewellyn Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 7, 2022

FILE NO. PG5570

HOLE NO. BH37-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.54
TOPSOIL	0.36	AU	1				
Compact to dense, brown **SILTY SAND to SANDY SILT**		SS	2	42	22	1	102.54
	2.21	SS	3	58	34	2	101.54
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders	2.67	SS	4	50	50+		

End of Borehole

Practical refusal to augering at 2.67m depth.

(GWL @ 1.52m - Jan. 11, 2022)

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

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 5-7 – MONITORING WELL WATER ELEVATIONS

DRAWING PG5570-4 – TEST HOLE LOCATION PLAN

DRAWING PG5570-5 – BEDROCK CONTOUR PLAN

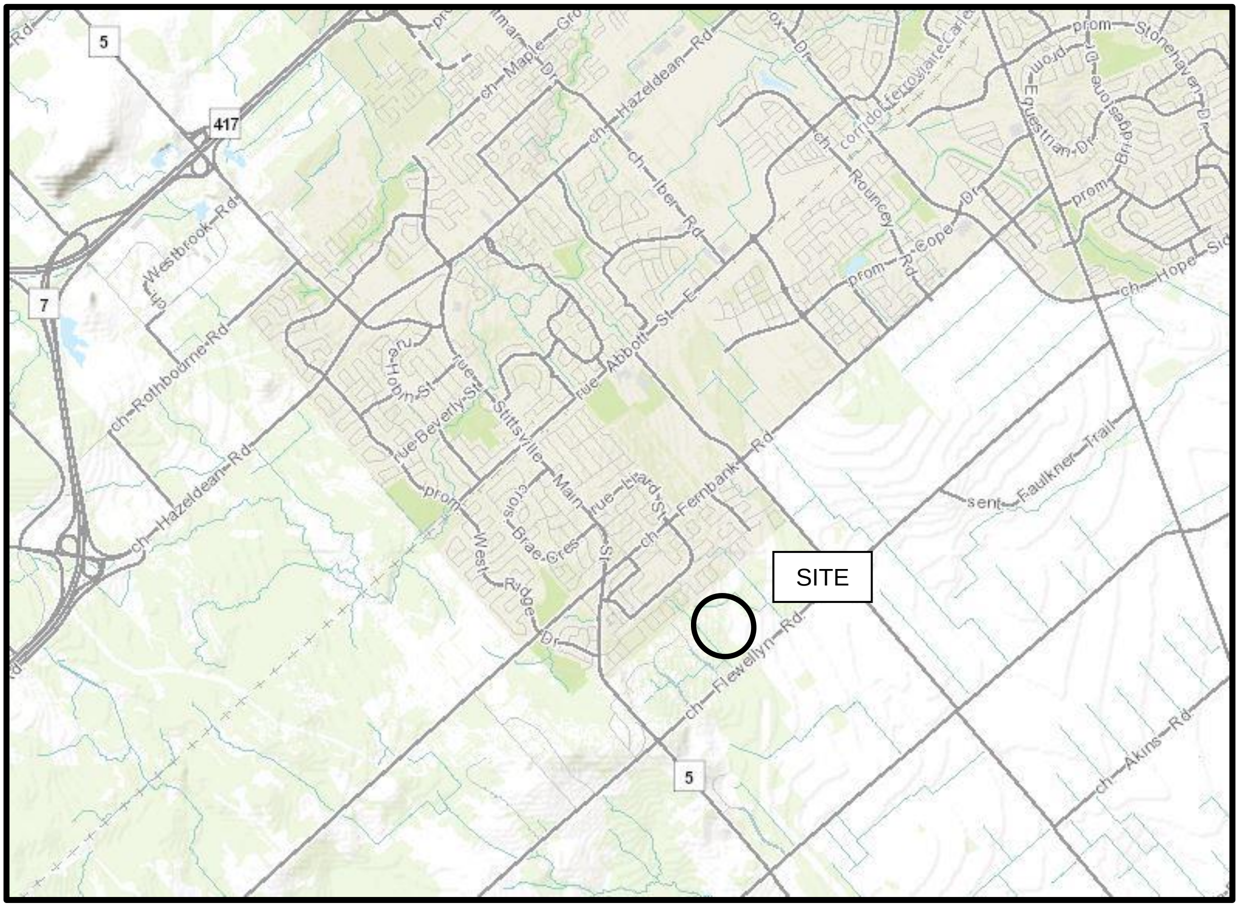


FIGURE 1

KEY PLAN

PATERSON
GROUP

Figure 5: BH22A-21 - Monitoring Well Water Elevations

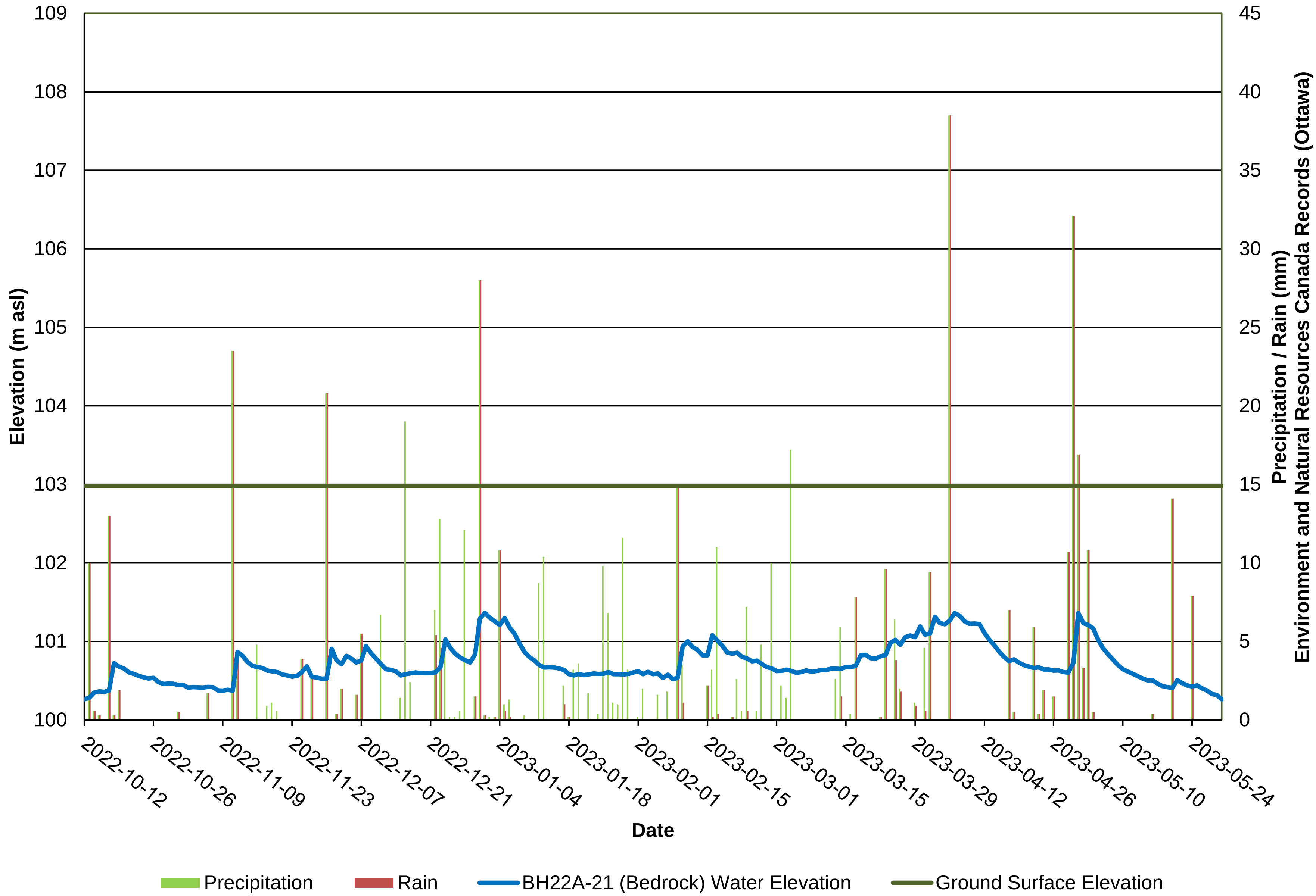
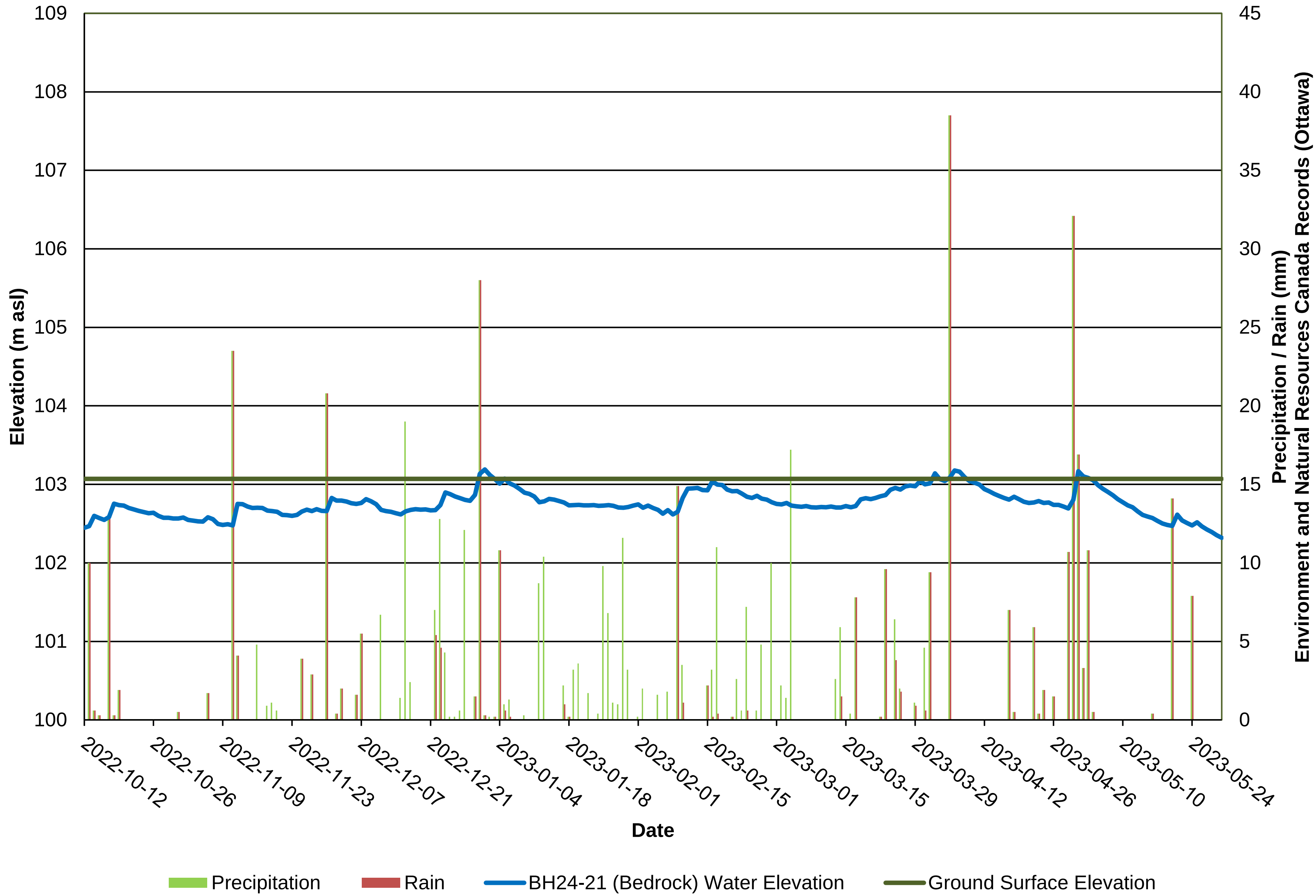
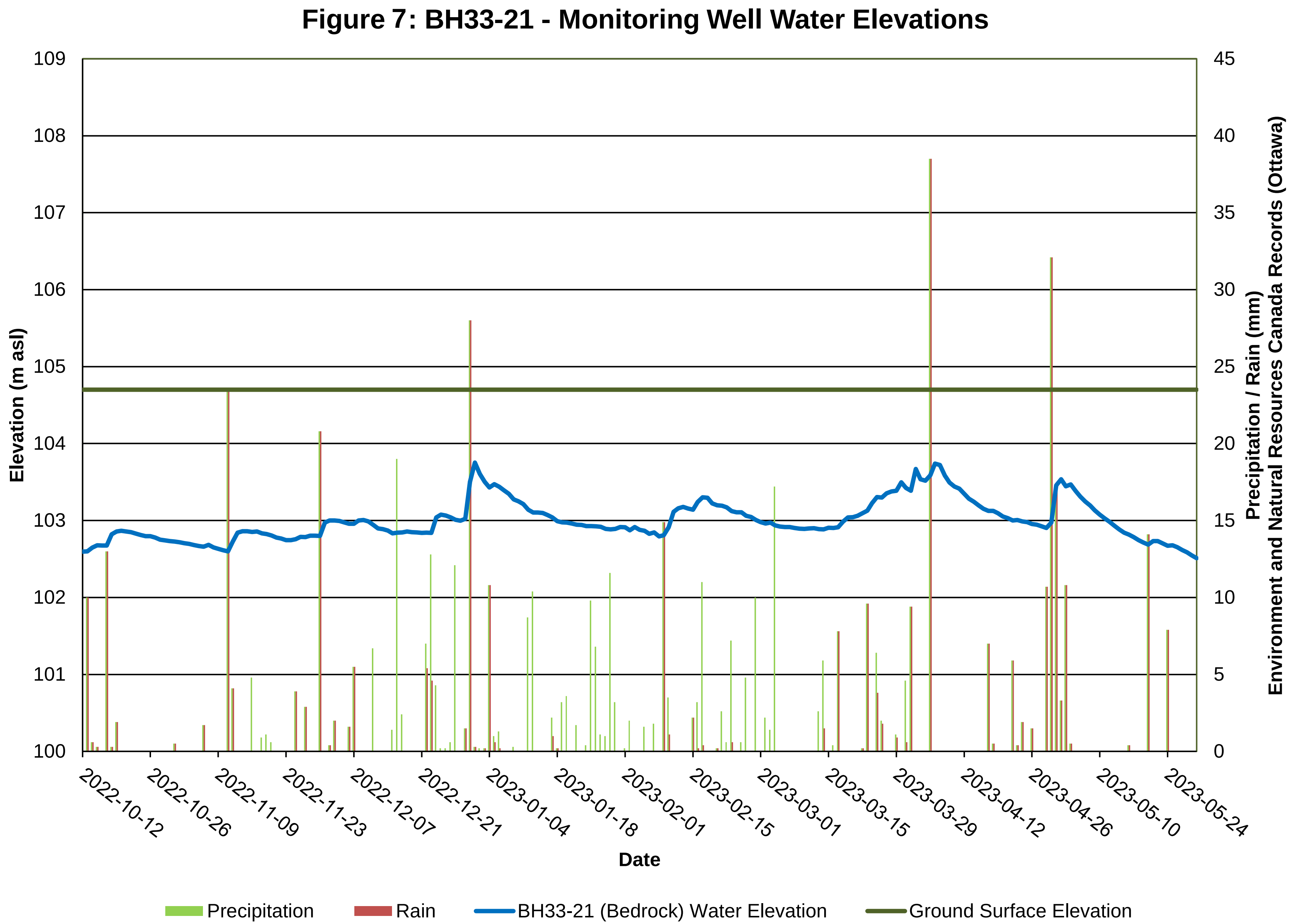


Figure 6: BH24-21 - Monitoring Well Water Elevations





PATERSON GROUP

LEGEND:

BOREHOLE WITH PIEZOMETER LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)

BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)

104.29 GROUND SURFACE ELEVATION (m)

[102.10] BEDROCK SURFACE ELEVATION (m)

(102.39) PRACTICAL REFUSAL TO AUGERING / EXCAVATION ELEVATION (m)

BASE PLAN PREPARED BY J.D. BARNES LIMITED.

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

SCALE: 1:2500

0 25 50 75 100 125 150 175m

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

NO.	REVISIONS	DATE	INITIAL

CAIVAN (STITTSVILLE SOUTH) INC. & CAIVAN (STITTSVILLE WEST) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
EAST PARCEL 1770 & 1820 SHEA ROAD

OTTAWA, ONTARIO

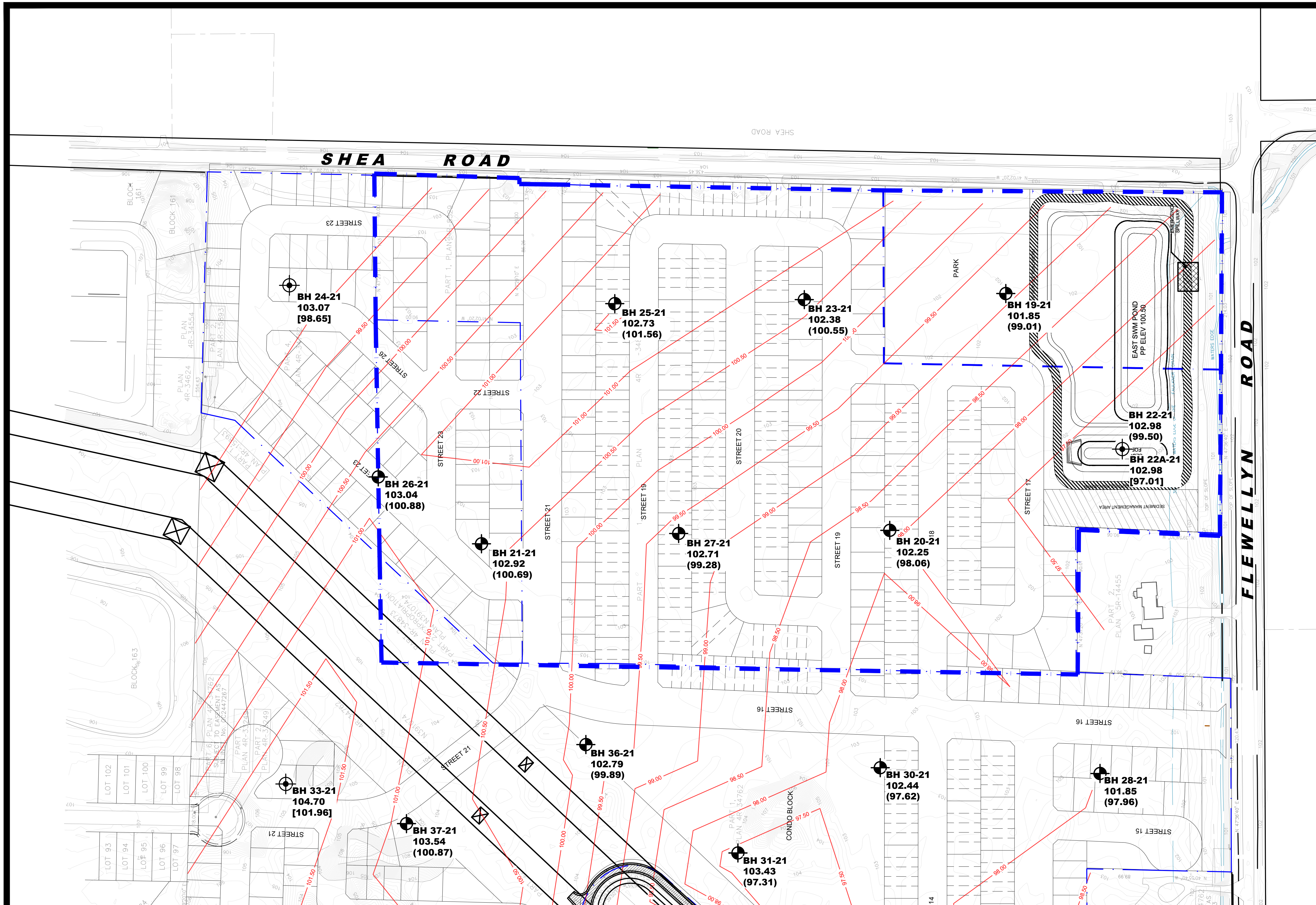
Title:

TEST HOLE LOCATION PLAN

Scale:	1:2500	Date:	06/2025
Drawn by:	ZS	Report No.:	PG5570-3
Checked by:	OC	Dwg. No.:	PG5570-4
Approved by:	KP	Revision No.:	

p:\autocad drawings\geotechnical\pg55xx\pg5570\pg5570-4-test hole location plan.dwg

11x17



LEGEND:

⌖	BOREHOLE WITH PIEZOMETER LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5570-2, JAN 20, 2022)
97.50	BEDROCK CONTOURS (m)
104.29	GROUND SURFACE ELEVATION (m)
[102.10]	BEDROCK SURFACE ELEVATION (m)
(102.39)	PRACTICAL REFUSAL TO AUGERING / EXCAVATION ELEVATION (m)

BASE PLAN PREPARED BY J.D. BARNES LIMITED.

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

SCALE: 1:2500

0 25 50 75 100 125 150 175m

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

NO.	REVISIONS	DATE	INITIAL

CAIVAN (STITTSVILLE SOUTH) INC. & CAIVAN (STITTSVILLE WEST) LTD.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
EAST PARCEL 1770 & 1820 SHEA ROAD

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:2500	Date: 06/2025
Drawn by: YA	Report No.: PG5570-3
Checked by: OC	Dwg. No.: PG5570-2
Approved by: KP	Revision No.:

11x17

p:\autocad drawings\geotechnical\pg55xx\pg5570\pg5570-4-test hole location plan.dwg