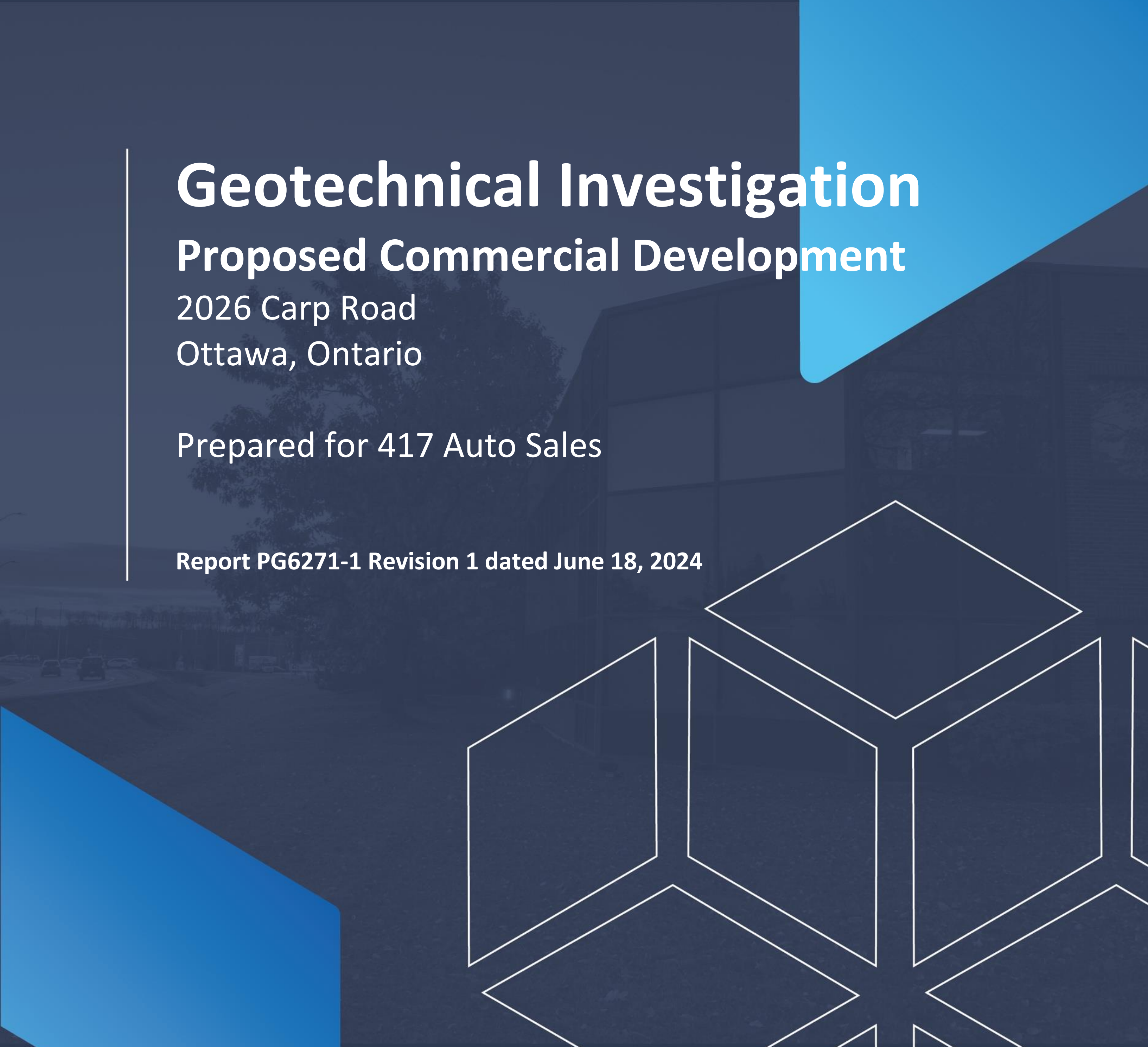

Geotechnical Investigation

Proposed Commercial Development

2026 Carp Road
Ottawa, Ontario

Prepared for 417 Auto Sales

Report PG6271-1 Revision 1 dated June 18, 2024

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 417 Auto Sales to conduct a geotechnical investigation for the proposed commercial development to be located at 2026 Carp Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

- ☐ 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 proposed development as they are understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist primarily of gravel- and asphalt-surfaced parking areas with landscaped margins located around the perimeter of the subject site. It is further understood that the existing residential dwelling is to be re-purposed for commercial use.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the supplemental geotechnical investigation was carried out on January 9, 2024 and consisted of advancing one (1) borehole to a maximum depth of 6.1 m. A previous geotechnical investigation was carried out on July 13, 2022, and consisted of advancing a total of four (4) boreholes to a maximum depth of 5.8 m.

The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features.

The borehole locations are shown on Drawing PG6271-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted 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 borehole procedure consisted of augering to the required depth at the selected borehole locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the borehole are shown as AU and SS, 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.

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.

In-Situ Infiltration Testing

In-situ infiltration testing was conducted at three (3) locations on November 30, 2023 to estimate infiltration rates of the unsaturated surficial soils at the subject site. At each testing location two (2) in-situ infiltration tests were conducted at depths ranging from 0.3 to 0.6 m below ground surface (bgs). The testing locations are shown on Drawing PG6271-1 - Test Hole Location Plan in Appendix 2.

The testing was conducted using a Pask (Constant Head Well) Permeameter. An 83 mm diameter hole was excavated using a Riverside/Bucket auger to remove topsoil and other subsoil material until the desired testing elevation. Two holes were advanced at each testing location to approximately 0.3 and 0.6 m bgs. All soil from the auger flights was visually inspected and initially classified on site. An aggregated soil sample was gathered at each test hole location. The permeameter reservoir was filled with water and inverted into the hole, ensuring it was relatively vertical and rested on the bottom of the hole. The water level of the reservoir was monitored at 0.5 to 1 minute intervals until the rate of fall out of the permeameter reached equilibrium, known as quasi “steady state” flow rate. Quasi steady state flow can be considered to have been obtained after measuring 3 to 5 consecutive rate of fall readings with identical values. The values for the steady state rate of fall were recorded for each test.

The steady state rate of fall was converted to a field saturated hydraulic conductivity value (Kfs) using the Engineering Technologies Canada (ETC) Ltd. reference tables provided in the most recent ETC Pask Permeameter User Guide. Unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation’s Low Impact Development Stormwater Management Planning and Design Guide.

Groundwater

A groundwater monitoring well was installed in BH1-24 during the supplemental investigation and piezometers were installed in BH2-22, BH3-22 and BH4-22 to permit the monitoring of groundwater levels subsequent to their respective geotechnical investigations.

Monitoring Well Installation

Typical monitoring well construction details are described below:

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

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

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

Groundwater Monitoring

A submersible datalogger (TD-Diver, VanEssen Instruments) was installed in BH1-24 on January 12, 2024, to continuously monitor fluctuations in groundwater levels, primarily over the spring months and early summer. The datalogger was installed at a depth of 5.5 m bgs. The datalogger can measure the equivalent hydrostatic pressure of the water above the sensor diaphragm to calculate the total water depth. The datalogger was programmed to continuously measure and record groundwater levels at a fixed rate of one (1) reading every twenty-four (24) hours. In addition to the continuous datalogger measurements, manual water level measurements were periodically taken throughout the groundwater monitoring program using an electronic water level meter.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration existing site features and underground utilities. The borehole location and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum.

The locations of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG6271-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing residential dwelling with a gravel-surfaced access lane located within the southwest corner of the subject site. The remainder of the site is generally crass covered with mature trees located near the site boundaries.

The subject site is bordered to the north, east and south by residential properties and to the west by Carp Road. The existing ground surface across the subject site slopes gently downward from west to east, away from Carp Road, from approximate geodetic elevations of 126.5 to 124.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the site consists of a thin topsoil layer and/or an approximate 0.2 to 1.3 m thick fill layer underlain by glacial till. The fill material was generally observed to consist of brown silty sand with gravel, crushed stone, cobbles, boulders, and organics.

Glacial till was encountered underlying the fill material at borehole BH1-24 and boreholes BH 2-22 to BH 4-22 at approximate depths of 0.5 to 1.3 m. The glacial till was generally observed to consist of a compact to very dense, brown silty sand with traces of gravel, cobbles, and boulders.

Practical refusal to augering was encountered at approximate depths of 1.1 to 5.8 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of limestone of the Bobcaygeon Formation with an overburden drift thickness of 5 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the piezometers on July 19, 2022 and measured periodically in the monitoring well installed in BH1-24. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 2-22	125.85	Dry	-	July 19, 2022
BH 3-22	124.36	Dry	-	July 19, 2022
BH 4-22	126.49	4.23	122.26	July 19, 2022
BH1-24	124.67	Dry	-	January 12, 2024
		Dry	-	March 5, 2024
		Dry	-	June 3, 2024

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

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 well was 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 for approximately five (5) months.

The monitoring program was undertaken from January to June 2024. The monitoring data was compared with Environment and Natural Resources Canada precipitation data from the Ottawa International Airport over the same timeframe as part of the monitoring program. The monitoring data is presented in Figure 2 in Appendix 2.

Upon review of the datalogger readings and manual measurements, the groundwater table did not rise above the installed depth of 5.5 m below the existing ground surface. This is supported by the manual groundwater level readings carried out at the well location during the program.

Based on these observations, the long-term groundwater level is expected to be located at a depth below the installed depth of the datalogger. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

4.4 Low Impact Development Feasibility (LID)

It is understood that Low Impact Development (LID) measures are being considered to be implemented for the proposed development. In order to assess the feasibility of LID measures, in-situ infiltration testing was completed at the subject site.

In-Situ Infiltration Testing Results

In-situ infiltration tests were conducted at three (3) locations to provide general coverage of the proposed LID measures at the subject site. Field saturated hydraulic conductivity (K_{fs}) values and estimated infiltration values are presented in Table 2 below.

Field saturated hydraulic conductivity values were determined using the Engineering Technologies Canada (ETC) Ltd. reference tables provided in the most recent ETC Pask Permeameter User Guide. Unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation's Low Impact Development Stormwater Management Planning and Design Guide.

Table 2 – Summary of Field Saturated Hydraulic Conductivity Testing Results and Estimated Unfactored Infiltration Rates

Testing Location ID	Ground Surface Elevation (m)	Infiltration Testing Elevation (m)	K_{fs} (m/sec)	Unfactored Infiltration Rate (mm/hr)	Material
PT1-23	123.39	123.09	8.0×10^{-6}	80.62	Glacial Till
		122.79	1.5×10^{-5}	95.39	Glacial Till
PT2-23	123.30	123.00	1.2×10^{-5}	89.86	Glacial Till
		122.70	1.9×10^{-5}	101.62	Glacial Till
PT3-23	123.32	123.02	1.1×10^{-5}	87.80	Glacial Till
		122.82	1.2×10^{-5}	89.86	Glacial Till

The measured K_{fs} values are consistent with expected field saturated hydraulic conductivity values for a glacial till with a silty sand matrix. 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 estimated infiltration rates derived from the Kfs values are unfactored. Prior to use for design purposes, a safety correction factor will need to be applied to the above infiltration rates. It should also be noted that for most LID measures, the invert of the system should be planned to be in accordance with the latest and pertinent City of Ottawa design guidelines, which are anticipated to require a minimum separation of 1 m above the seasonally high groundwater table and bedrock surface.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. If required, foundation support for proposed buildings could consist of conventional spread footings bearing on undisturbed, compact to very dense glacial till.

Cobbles and boulders were encountered in the fill and glacial till while drilling the boreholes. Therefore, the contractor should be prepared for cobbles and boulders removal during the site excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas and other settlement sensitive structures. It is anticipated that the existing fill within the footprint of the proposed parking areas, free of deleterious material and significant amounts of organics, can be left in place.

It is recommended that the existing fill layer be proof-rolled with several passes of a vibratory drum roller, under dry conditions and above freezing temperatures, and which is 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.

Fill Placement

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

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

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Conventional Shallow Footings

If buildings are included as part of the site development, footings placed on an undisturbed, compact to very dense glacial till bearing surface, or on engineered fill which is placed over a compact to very dense glacial till bearing surface, can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

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

Footings places on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and different settlements of 25 to 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 a glacial till or engineered fill bearing surface 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 of the same or higher capacity as that of the bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade Construction

Should a slab-on-grade be required for proposed buildings, all topsoil and deleterious fill should be removed, and the existing fill or native soil subgrade, approved by the geotechnical consultant at the time of excavation, will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. Where the subgrade consists of existing fill, it is recommended that the slab-on-grade subgrade be proof-rolled with a suitably sized roller making several passes under dry conditions prior to fill placement. Any poor performing areas should be removed and replaced with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials required to raise grade within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Design

The pavement structures for car only parking areas and access lanes are presented in Tables 3 and 4 below.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Pavement Structure - Access Lanes

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

Where gravel parking areas are proposed, the OPSS Granular A base should be increased to account for the missing asphalt thickness (i.e. a total base thickness of 200 mm of OPSS Granular A for car only parking areas and 240 mm for access lanes).

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Should buildings with below-grade space be proposed at the subject site, it is recommended that a perimeter foundation drainage system be provided for these structures. The system should consist of a 150 mm diameter, perforated and corrugated plastic or PVC pipe which is surrounded by 150 mm of 19 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

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against foundation walls, where required, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Backfill against the exterior sides of foundation walls should consist of free-draining, non-frost susceptible granular materials. 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 recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation should be provided in this regard.

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

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient space will be available to slope the excavations.

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.

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 is used 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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be 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 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.

Impacts on Neighbouring Properties

Based on the available drawings, excavation below the existing groundwater table is not anticipated at the subject site. Therefore, groundwater lowering is not anticipated during or after construction and accordingly, the proposed development will not negatively impact the neighbouring structures.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

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

☐ 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 soils 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 417 Auto Sales 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.

Zavian Buchanan

Zavian Buchanan, EIT.

LICENSED PROFESSIONAL ENGINEER
Jun. 18, 2024
K. A. PICKARD
100531344
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PROVINCE OF ONTARIO

Kevin A. Pickard, P.Eng

Report Distribution:

☐ 417 Auto Sales (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2026 Carp Road
Ottawa, Ontario

EASTING: 348162.319 **NORTHING:** 5015040.015 **ELEVATION:** 124.67

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** January 9, 2024

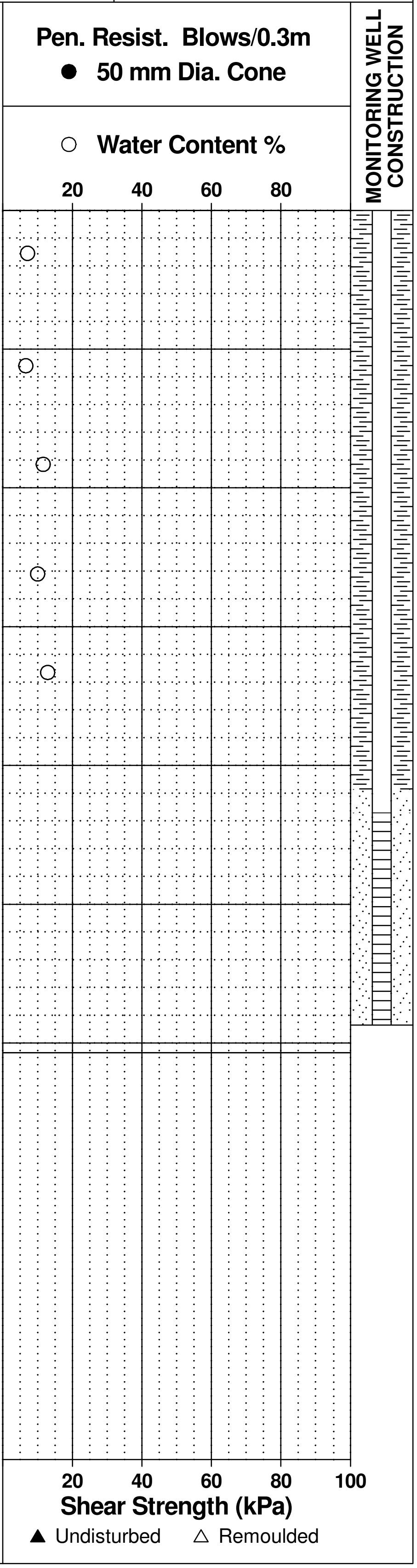
FILE NO. PG6271

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL with organics	0.15
FILL: Brown silty sand	0.76
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	6.07
End of Borehole (Dry Borehole - Jan. 12, 2024)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	50	16
SS	2	42	25
SS	3	42	46
SS	4	54	+50
SS	5	40	+50
RC	1	56	
RC	2	59	

DEPTH (m)	ELEV. (m)
0	124.67
1	123.67
2	122.67
3	121.67
4	120.67
5	119.67
6	118.67



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - 2026 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 13, 2022

FILE NO. PG6271

HOLE NO. BH 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE									
25mm Crushed stone						0	126.29		
FILL: Brown silty sand with gravel and crushed stone		AU	1						
- some cobbles and boulders by 0.7m depth		SS	2	33	50+	1	125.29		
1.14									
End of Borehole Practical refusal to augering at 1.14m 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
Proposed Commercial Building - 2026 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 13, 2022

FILE NO. PG6271

HOLE NO. BH 1A-22

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	126.29		
25mm Crushed stone **FILL:** Brown silty sand with gravel and crushed stone - some cobbles and boulders by 0.7m depth	▨					1	125.29		
1.32									
End of Borehole Practical refusal to augering at 1.32m 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
Proposed Commercial Building - 2026 Carp Road
Ottawa, Ontario

DATUM Geodetic

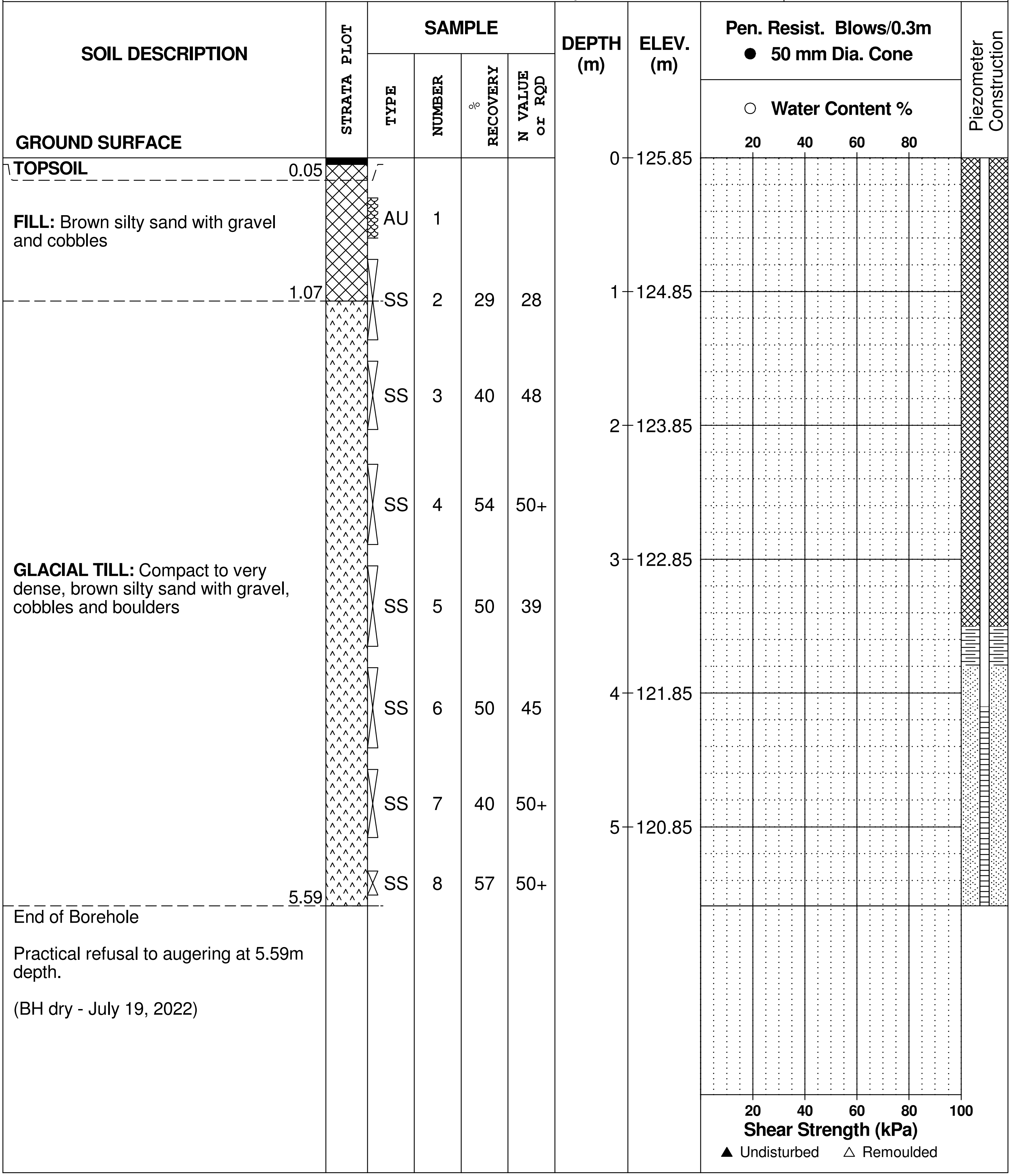
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 13, 2022

FILE NO. PG6271

HOLE NO. BH 2-22



SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.05				
FILL: Brown silty sand with gravel and cobbles	1.07	AU	1		
		SS	2	29	28
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders	5.59	SS	3	40	48
		SS	4	54	50+
		SS	5	50	39
		SS	6	50	45
		SS	7	40	50+
		SS	8	57	50+

End of Borehole

Practical refusal to augering at 5.59m depth.

(BH dry - July 19, 2022)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - 2026 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 13, 2022

FILE NO. PG6271

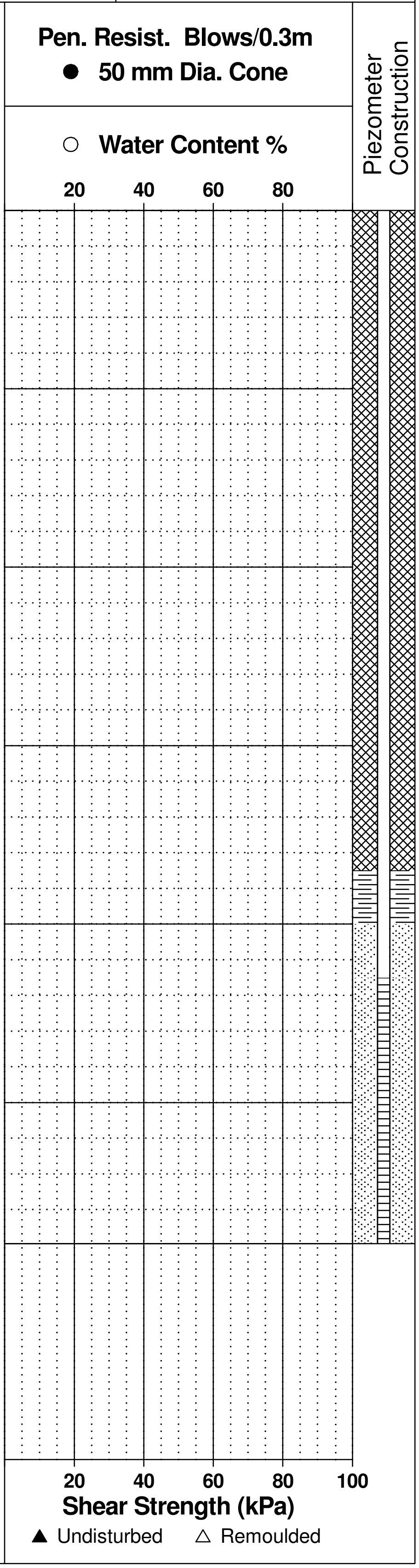
HOLE NO. BH 3-22

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	124.36
TOPSOIL 0.05						
FILL: Brown silty sand, trace gravel	AU	1				
0.91	SS	2	50	50+		
					1	123.36
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	3	38	50+		
					2	122.36
	SS	4	57	50+		
					3	121.36
	SS	5	75	50+		
	SS	6	86	50+		
					4	120.36
	SS	7	57	50+		
					5	119.36
	SS	8		50+		
5.79						

End of Borehole

Practical refusal to augering at 5.79m depth.

(BH dry - July 19, 2022)



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Building - 2026 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

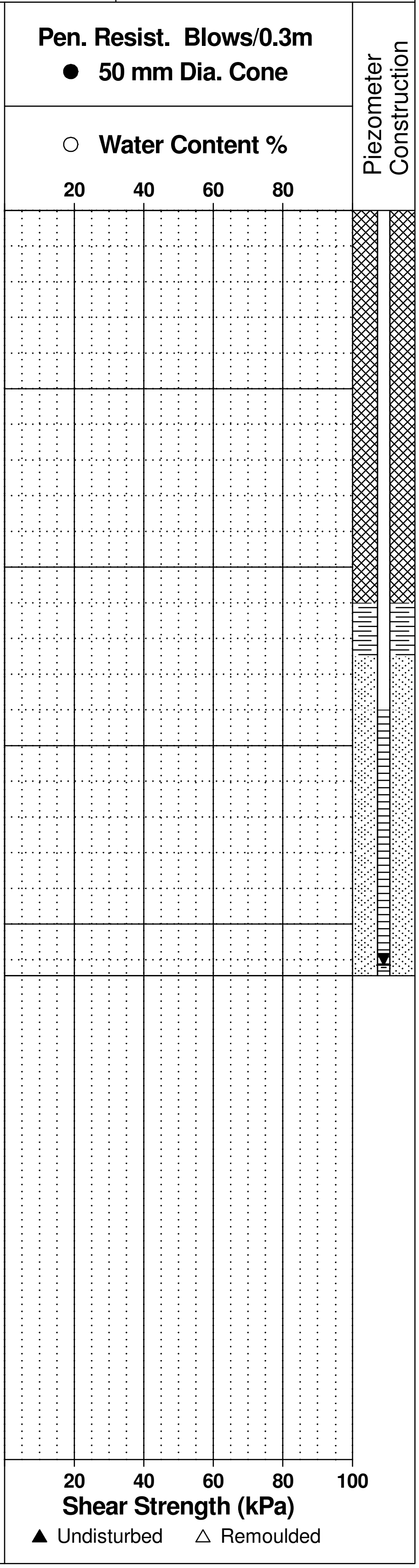
DATE July 13, 2022

FILE NO. PG6271

HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.25		AU	1		
FILL: Brown silty sand with organics, trace wood 0.46					
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders		SS	2	46	25
		SS	3	77	50+
		SS	4	89	50+
		SS	5	57	50+
		SS	6	57	50+
4.29					
End of Borehole Practical refusal to augering at 4.29m depth. (GWL @ 4.23m - July 19, 2022)					

DEPTH (m)	ELEV. (m)
0	126.49
1	125.49
2	124.49
3	123.49
4	122.49



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

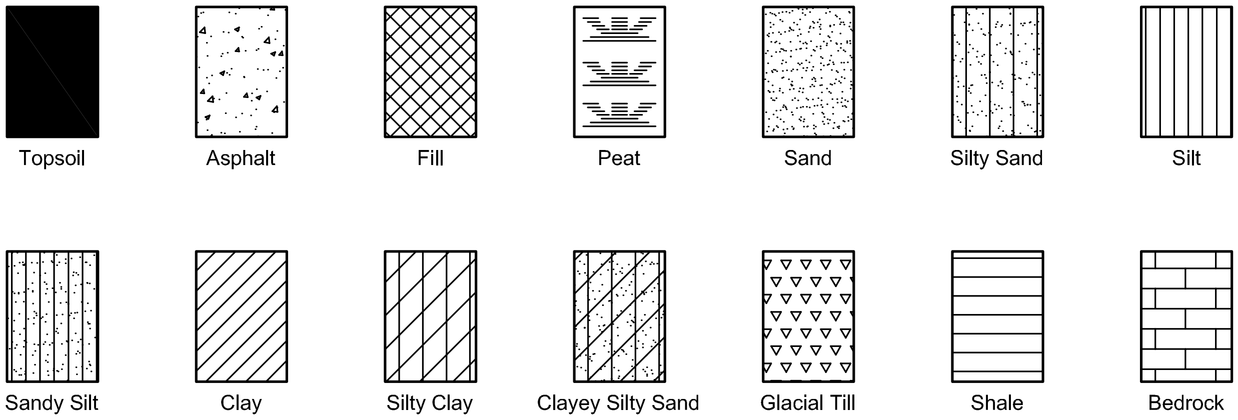
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

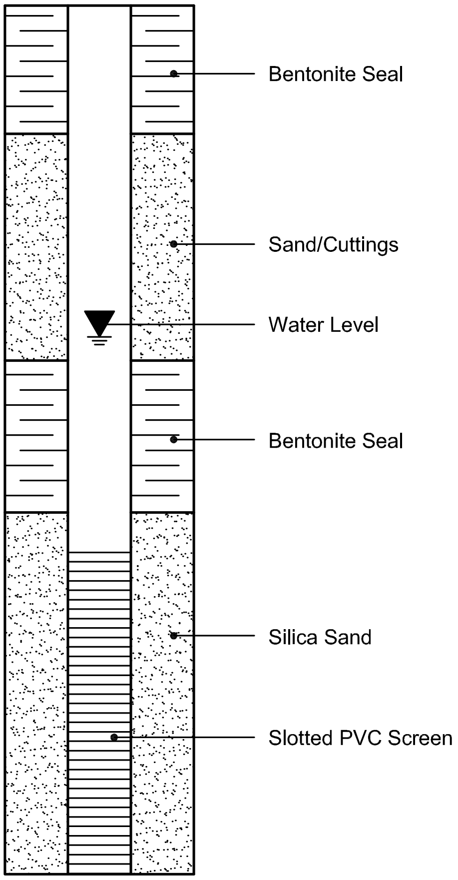
SYMBOLS AND TERMS (continued)

STRATA PLOT

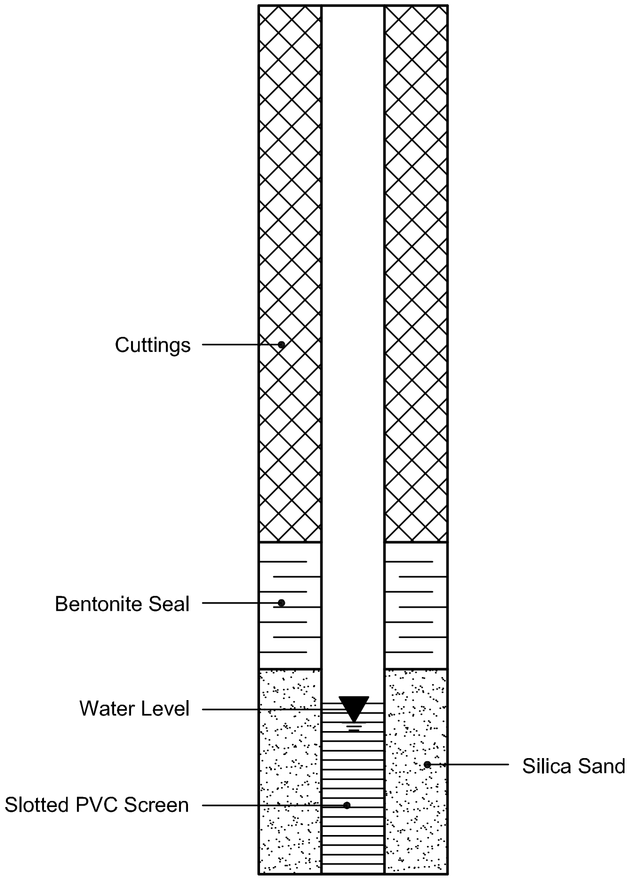


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 55274

Report Date: 21-Jul-2022

Order Date: 13-Jul-2022

Project Description: PG6271

	Client ID:	BH2-22-SS3	-	-	-
	Sample Date:	13-Jul-22 09:00	-	-	-
	Sample ID:	2229467-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	97.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	81.4	-	-	-
Anions					
Chloride	5 ug/g dry	16	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - MONITORING WELL WATER ELEVATIONS

DRAWING PG6271-1 - TEST HOLE LOCATION PLAN

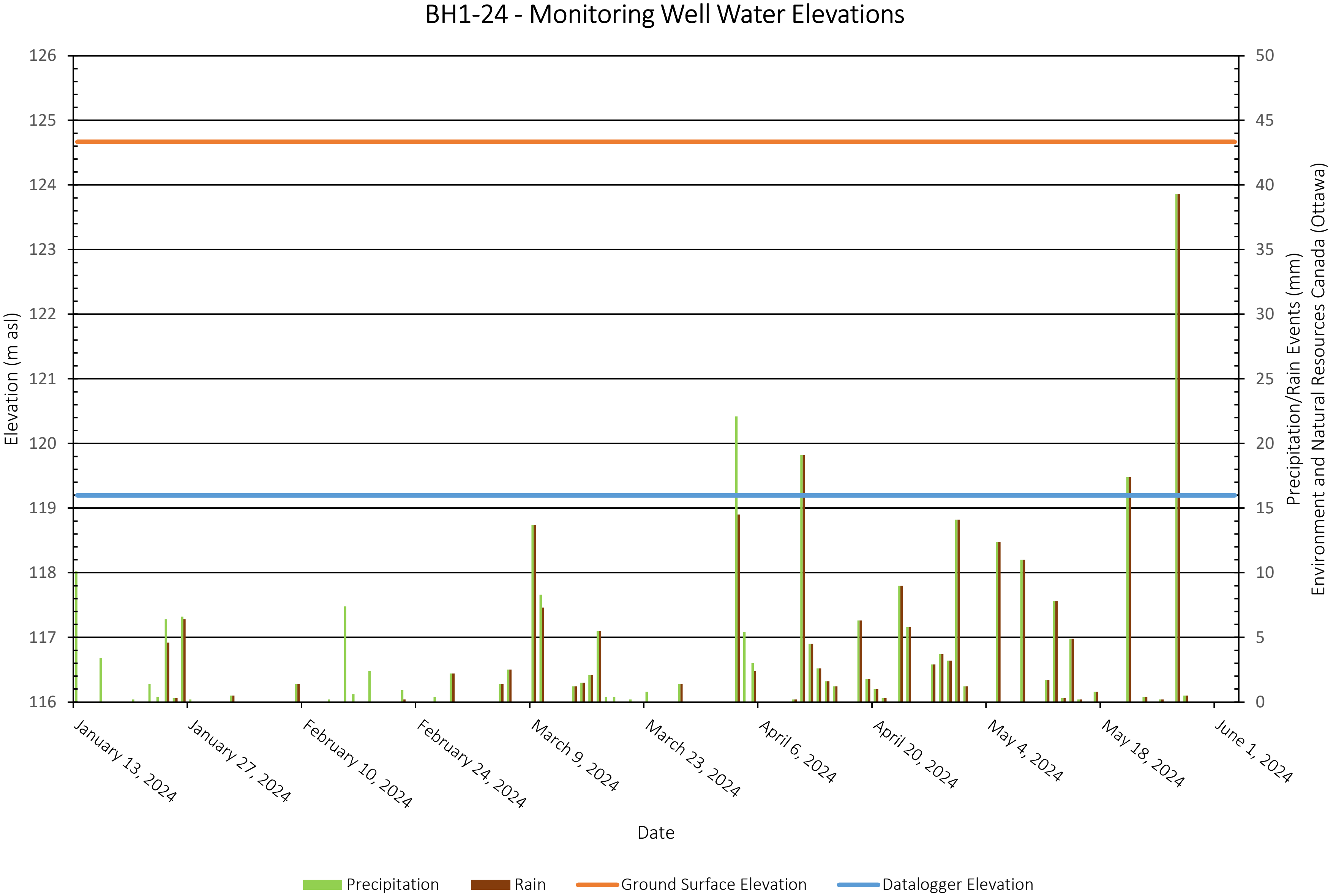
FIGURE 1

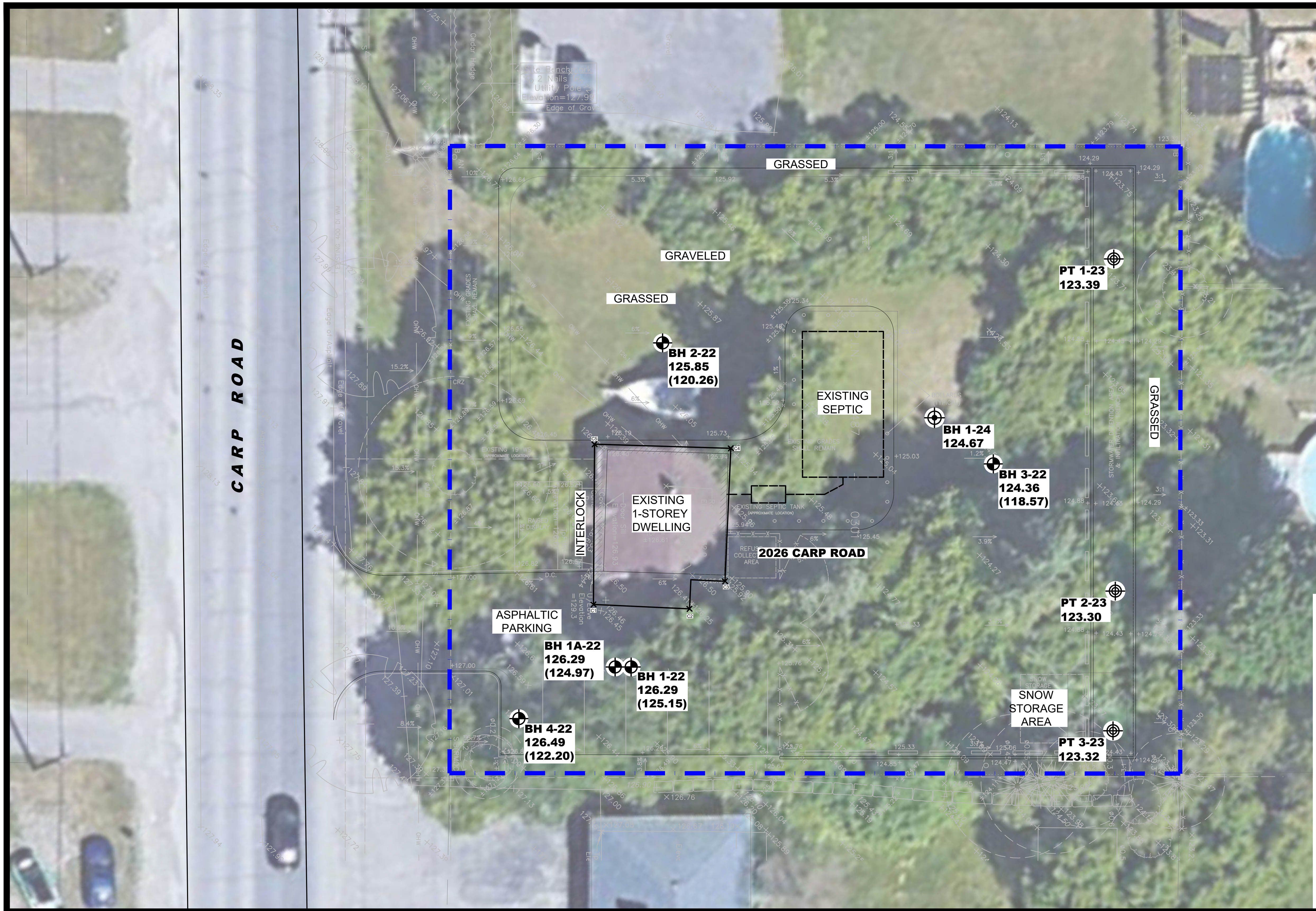
KEY PLAN

PATERSON
GROUP

File: PG6271

417 Auto Sales
2026 Carp Road
Monitoring Well Water Elevations





LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION
⌖	PERMEAMETER TESTING LOCATION
126.29	GROUND SURFACE ELEVATION (m)
(125.15)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

BASE PLAN PROVIDED BY D.B. GRAY ENGINEERING INC., DATED JULY 31, 2023.

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED BASEPLAN, BOREHOLE LOCATION BH1-24 AND PERMEAMETER TESTING LOCATIONS ADDED	23/01/2024	ZB

417 AUTO SALES

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING
2026 CARP ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250

Date: 07/2022

Drawn by: GK

Report No.: PG6271-1-REV.01

Checked by: ZB

Dwg. No.: PG6271-1

Approved by: KP

Revision No.: 1

11x17

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