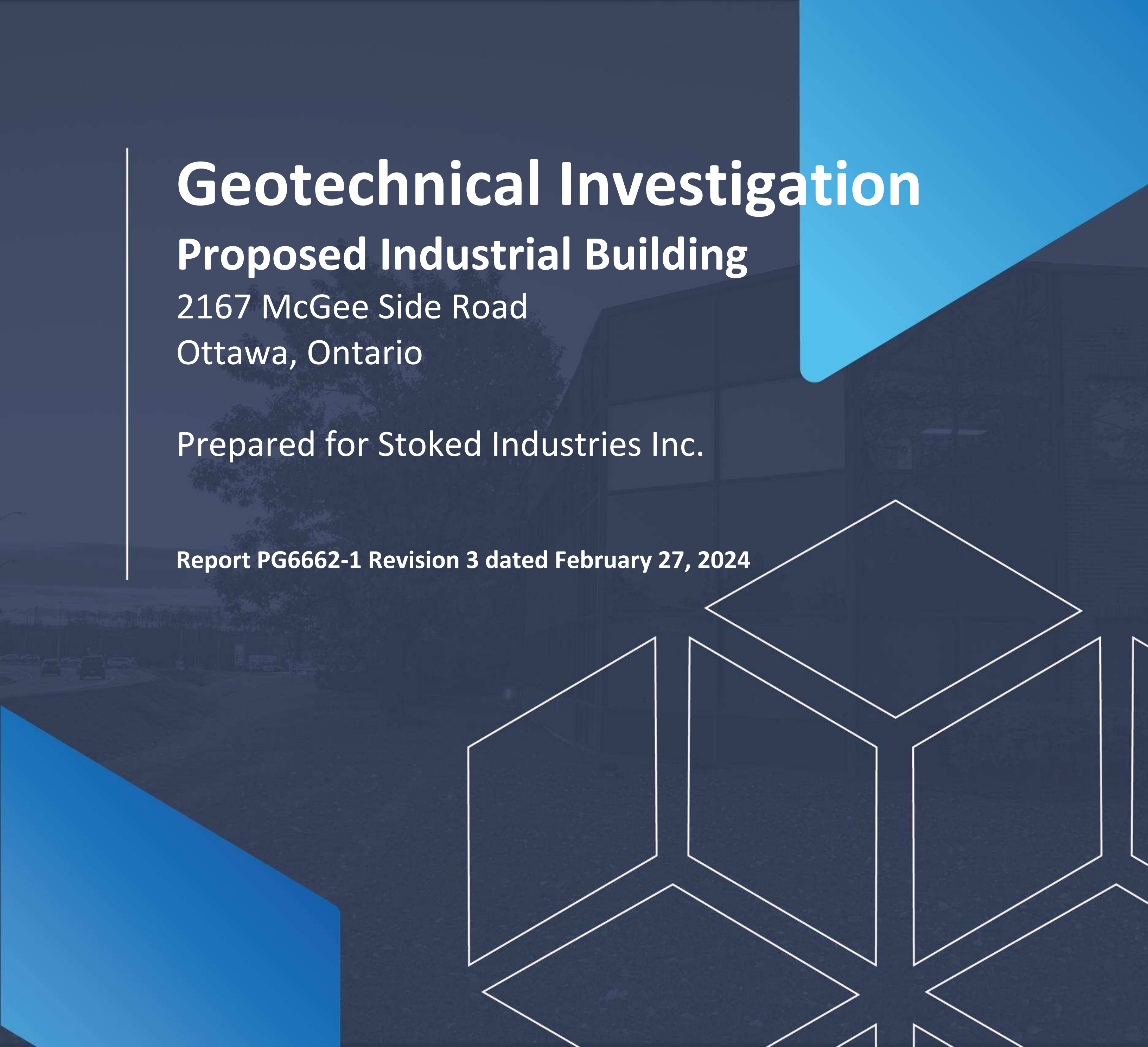
PATERSON GROUP



Geotechnical Investigation

Proposed Industrial Building

2167 McGee Side Road
Ottawa, Ontario

Prepared for Stoked Industries Inc.

Report PG6662-1 Revision 3 dated February 27, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Stoked Industries Inc. to prepare a geotechnical investigation report for the proposed industrial building to be located at 2167 McGee Side Road, Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

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

☐ Provide geotechnical recommendations for the design of the proposed building including construction considerations which may affect its 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.

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed building will consist of a low-rise industrial building of slab-on-grade construction with an approximate footprint of 1635 m2. It is also understood that there will be a mezzanine. Associated asphalt-paved access lanes, parking areas with landscaped margins, and garbage enclosure are also proposed surrounding the building. It is further understood that the proposed building will be privately serviced and contain an infiltration trench within the eastern corner of the property as part of the stormwater management design.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on November 20, 2020. At that time, seven (7) boreholes were advanced to a maximum depth of 4.8m. The borehole locations were distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations of the boreholes are shown on Drawing PG6662-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected locations, sampling and testing the overburden.

A supplemental hydrogeological field program was carried out on November 23, 2023, to determine the infiltration potential of the soils, as well as on-going monitoring of seasonal groundwater fluctuations in support of the infiltration trench. At that time, two (2) test pits and one hand auger were advanced to a maximum depth of 1.8 and 2.1 m bgs, respectively.

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, split-spoon and grab 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.

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

Infiltration Testing

In-situ infiltration testing was conducted using a Pask Constant Head Permeameter on November 23, 2023. Two test pits were excavated to a maximum depth of 1.8 m below ground surface (bgs). Each test pit was excavated in approximately 0.3 m increments to allow for safe entry into the pits as well as permeameter testing to be conducted at different elevations. Infiltration tests were conducted at an approximate elevation of 116 to 116.2 m asl and 115.2 to 115.7 m asl within each test pit. At approximately 0.2 m above the desired testing elevation, an 83 mm diameter hole was excavated using a Riverside/Bucket auger to the desired testing depth. All soils from the auger flights were visually inspected and initially classified on site. An aggregated soil sample was gathered at each test location. The test was conducted by filling the permeameter reservoir with water and inverting it into the hole, ensuring it was relatively vertical and rested at the bottom of the hole. The water level of the reservoir was monitored at 0.25 to 0.5 minute intervals until the rate of fall out of the permeameter reached equilibrium, known as a quasi "steady state" flow rate. Quasi steady state flow was considered to be obtained after measuring 3 to 5 consecutive rate of fall readings with identical values. The values for the steady state rate of fall were recorded for each location. The steady state rate of fall was converted to a field saturated hydraulic conductivity value (K_{fs}) using the Engineering Technology Canada Ltd. conversion tables. Unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation's Low Impact Development Stormwater Management Planning and Design Guide.

Groundwater

Flexible polyethylene standpipes were installed in all boreholes during the geotechnical field investigation to permit monitoring of the groundwater levels subsequent to the completion of the field program. The groundwater observations are discussed in subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

In addition, hand auger hole HA 1-23 has been equipped with a monitoring well installation. An on-going groundwater monitoring program has commenced following the most recent hydrogeological field program.

The monitoring well has been equipped with a Van Essen Instrument Mini-Diver Water Level Logger and Baro-Diver to monitor seasonal groundwater fluctuations and changes in atmospheric pressure. The Mini-Divers have been programmed to continuously measure and record groundwater levels at a rate of 1 reading every 24 hours. The continuous groundwater level readings at the monitoring well location are discussed in subsection 4.3 and presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision GPS and referenced to a geodetic datum. The location of the test holes is presented on Drawing PG6662-1 - 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.

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 others. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is occupied by a gravel-surface parking area. A culvert system, which runs parallel with McGee Side Road and John Cavanaugh Drive, is located along the eastern and southern limits of the site, respectively. The subject site is bordered by undeveloped, densely treed, land to the north, John Cavanaugh Drive to the east, McGee Side Road to the south and a commercial property to the west. The existing ground surface is relatively flat within the central portion of the site at approximate geodetic elevations of 117 to 118 m. At the eastern and southern boundaries of the site, the ground surface slopes downwards from approximate geodetic elevations of 119 to 116.5 m, where the culvert system is present.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile observed at the test hole locations consists of a an approximate 0.3 to 1.2 m thick layer of fill which is composed of a brown silty sand with crushed stone. A glacial till deposit was encountered underlying the fill and was observed to consist of a compact to very dense, brown silty sand with gravel, cobbles and boulders.

Practical refusal to augering was encountered in all boreholes at depths ranging from 1.6 to 4.8 m.

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

Bedrock

Based on the available geological mapping and on the findings of the investigation, the site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam formation with a drift thickness of 3 to 5 m.

4.3 Groundwater

Groundwater levels were measured in the installed piezometers and monitoring well, as well as the open excavations. The groundwater readings obtained are summarised in Table 1 below and are also presented on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1	117.98	3.96	114.02	December 2, 2020
BH 2	117.96	Blocked and Dry to 115.92		December 2, 2020
BH 3	117.39	2.26	115.13	December 2, 2020
BH 4	117.07	Blocked and Dry to 116.13		December 2, 2020
BH 5	117.98	Blocked and Dry to 116.65		December 2, 2020
BH 6	117.23	2.09	115.14	December 2, 2020
BH 7	118.08	Blocked and Dry to 116.70		December 2, 2020
TP 1-23	117.06	Dry	-	November 23, 2023
TP 2-23	116.94	Dry	-	November 23, 2023
HA 1-23	117.06	Dry	-	November 23, 2023

Note: The ground surface elevation at each borehole location was surveyed using a high precision GPS and referenced to a geodetic datum.

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole. Long-term groundwater levels can also be estimated based on recovered soils samples moisture levels and recovered soil sample coloring and consistency. Based on the existing groundwater information and our knowledge of the groundwater within the area, the long-term groundwater level is estimated to be at **2** to **3 m** depth below the existing grade.

Seasonal Groundwater Monitoring Results

The data presented in the monitoring well water elevation plot included in Appendix 1 illustrates the collected groundwater elevations between November 24, 2023, and February 16, 2024. The groundwater readings measured within the monitoring well varied from an elevation of <115.1 m asl to a maximum elevation of 115.4 m asl. However, it should be noted that a significant response in the measured groundwater level at the monitoring well location generally occurred immediately following a substantial rain event on December 18, 2023. This sudden groundwater response is not indicative of the seasonal groundwater table at that time, but rather a surficial connection to the monitoring well screen. It should also be noted the adjacent roadside ditch with a maximum invert of approximately 115.3 m asl has been observed to be dry at the time of the hydrogeological investigation.

Based on the Site Servicing Study and Stormwater Management Report prepared by D.B. Gray Engineering Inc. (Report No. 23024, dated June 27, 2023), it is understood the invert of the proposed infiltration trench has been designed to an elevation of 116.3 m asl. Given the measured groundwater table for the current monitoring period is generally below 115.1 m asl, a minimum of 1 m separation has been maintained from the invert of the proposed infiltration trench to date. The on-going groundwater monitoring program at the subject site will continue to measure groundwater fluctuations through the upcoming spring freshet.

4.4 Infiltration Rates

In-situ infiltration testing was conducted using a Pask permeameter at two locations within the footprint of the proposed infiltration trench and below the proposed invert of the system. The testing locations are shown on Paterson Drawing PG6662-1 - Test Hole Location Plan.

The unfactored infiltration rates of the unsaturated surficial soils at the subject site varied between 47 to 68 mm/hr for the glacial till, and 104 mm/hr for the fill material. The field saturated hydraulic conductivity a values and estimated infiltration rates have been summarized in Table 2 below.

Table 2 – Summary of Field Saturated Hydraulic Conductivity Values and Estimated Infiltration Rates

Test Completed within Test Pit ID	Ground Surface (m asl)	Testing Elevation (m asl)	Material	K_{fs} (m/s)*	Unfactored Infiltration Rate (mm/hr)
TP 1-23	117.1	116	Glacial Till	2.1×10^{-6}	56
		115.5	Glacial Till	1.1×10^{-6}	47
TP 2-23	116.9	116.2	Fill Material	2.1×10^{-5}	104
		115.7	Glacial Till	4.3×10^{-6}	68
		115.2	Glacial Till	2.1×10^{-6}	56

*Field Saturated Hydraulic Conductivity

Based on the Site Servicing Study and Stormwater Management Report prepared by D.B. Gray Engineering Inc. (Report No. 23024, dated June 27, 2023), an unfactored infiltration rate of 30 to 75 mm/hr has been estimated for the underlying soils. In-situ infiltration testing at the subject site is generally in accordance with the estimated infiltration rates provided in the study.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building. The proposed industrial building can be founded using conventional shallow footings placed on over an undisturbed, compact to very dense glacial till bearing surface.

Dependent on the founding depths of the site servicing, it is anticipated that localized bedrock removal may be required. However, it is expected that bedrock removal, if required, would be in relatively small quantities and could be completed using hoe ramming. This is discussed further in Section 5.2.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Bedrock Removal

Bedrock removal could be carried out by hoe-ramming where only small quantities of bedrock need to be removed. It is anticipated that bedrock may be encountered within the depth of excavation required for the installation of site services. It is expected that bedrock removal will be possible using hoe ramming in conjunction with conventional excavation techniques.

Fill Placement

Fill used 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, 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 building areas should be compacted to at least 98% of its 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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective 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 geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Footings)

Strip footings, up to 2 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied 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, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the encountered overburden material above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered fill of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the 2012 Ontario Building Code (OBS 2012).

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

5.5 Slab on Grade Construction

With the removal of any topsoil and fill, such as those containing significant amounts of deleterious materials, the existing fill or native soil subgrade approved by Paterson at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction. 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, and which is approved by Paterson personnel. Any poor performing areas should be removed and replaced with an engineered fill, such as Granular B Type II.

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

5.6 Pavement Design

The pavement structures presented in the following tables can be used for the design of car only parking, access lanes and heavy truck parking areas.

Table 3 - 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 soils or OPSS Granular B Type I or II material placed over in situ soil, or fill	

Table 4 - Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 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 sump pump pit or storm sewer.

Foundation Backfilling

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta 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.

Concrete Sidewalks and Walkways

Backfill material below sidewalks and walkway subgrade areas throughout the subject site, including along the building, should be provided with a minimum 300 mm thick layer of OPSS Granular A or OPSS Granular B Type II crushed stone. This material should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the materials SPMDD. The subgrade for walkway structures against the building should be shaped to promote drainage towards the buildings perimeter drainage system.

6.2 Protection 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.

6.3 Excavation Side Slopes

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

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be 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 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Service Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of its SPMDD. The bedding should extend at least to the spring line of the pipe.

The cover material, which should consists of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be 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.

However, based on the groundwater level readings at the borehole locations, as well as the observed colour, moisture content and consistency of the recovered soil samples, it is anticipated that the long-term groundwater levels are below the founding elevation of the proposed building. Therefore, an EASR or PTTW will not be required for dewatering during the construction of the proposed development.

6.6 Winter Construction

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

The subsoil conditions at this site mostly 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.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.01%. 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 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 ensure that the specified level of compaction has been 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 Stoked Industries Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

MONITORING WELL WATER EVELATION PLOT

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

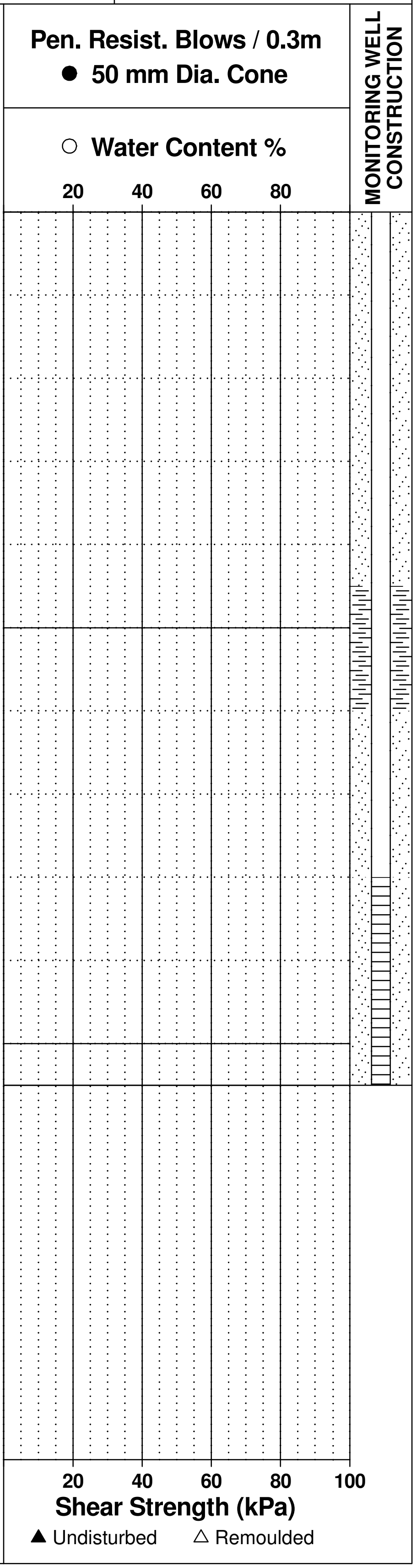
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

EASTING: **NORTHING:** **ELEVATION:** 117.06
DATUM: Geodetic
REMARKS:
BORINGS BY: Hand Auger **DATE:** November 23, 2023

FILE NO. **PG6662**
HOLE NO. **HA 1-23**

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	117.06
FILL: Brown silty sand with gravel, cobbles and organics	G	1				
0.30						
FILL: Brown silty sand with crushed stone and cobbles	G	2				
					1	116.06
	G	3				
1.10						
GLACIAL TILL: Brown silty sand, trace gravel	G	4				
					2	115.06
2.10						
End of Hand Auger Hole Practical refusal to hand augering on inferred boulders at 2.10m depth (Dry upon completion)						



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

EASTING: **NORTHING:** **ELEVATION:** 117.06

DATUM: Geodetic

REMARKS:

BORINGS BY: Excavator **DATE:** November 23, 2023

FILE NO. PG6662

HOLE NO. TP 1-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	117.06		
FILL: Brown silty sand with gravel, cobbles and organics 0.30	▨	G	1						
FILL: Brown silty sand with crushed stone and cobbles	▨	G	2						
	▨	G	3			1	116.06		
1.10 **GLACIAL TILL**: Brown silty sand, trace gravel 1.56	^^^								
End of Test Pit (Test pit dry upon completion on Nov. 23, 2023)									

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 Building - 2167 McGee Side Road
Ottawa, Ontario

EASTING: **NORTHING:** **ELEVATION:** 116.94

DATUM: Geodetic

REMARKS:

BORINGS BY: Excavator **DATE:** November 23, 2023

FILE NO. PG6662

HOLE NO. TP 2-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	116.94		
FILL: Brown silty sand with gravel, cobbles, boulders and organics 0.20	▨	G	1						
FILL: Brown silty sand with gravel, cobbles, boulders and crushed stone	▨	G	2						
	▨					1	115.94		
1.20	▨	G	3						
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 1.75	^^^	G	4						
End of Test Pit Practical refusal to excavation on inferred boulders at 1.75m depth (Test pit dry upon completion on Nov. 23, 2023)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

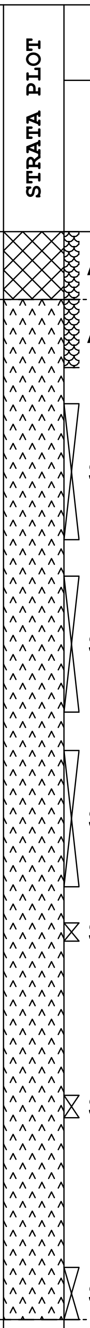
BORINGS BY CME-55 Low Clearance Drill

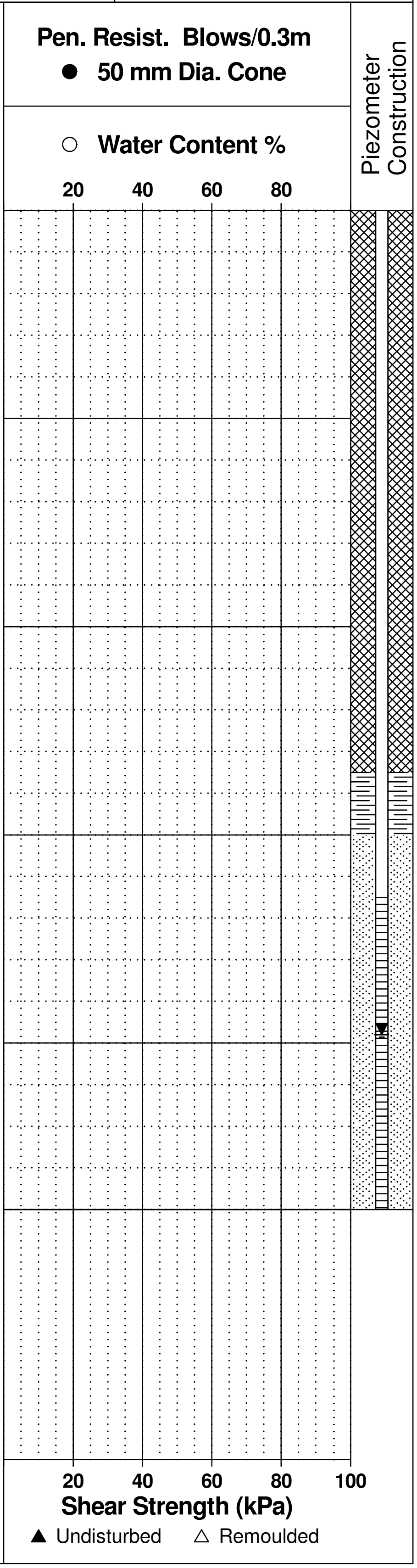
DATE November 20, 2020

FILE NO. PG6662

HOLE NO. BH 1-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	117.98
FILL: Brown silty sand with crushed stone (to 0.30)	AU	1				
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders (0.30 to 4.80)	AU	2				
	SS	2	25	48	1	116.98
	SS	3	67	44	2	115.98
	SS	4	79	66		
	SS	5	0	50+	3	114.98
	SS	6	50	50+	4	113.98
	SS	7	67	50+		
End of Borehole (4.80)						





End of Borehole

Practical refusal to augering at 4.80m depth

(GWL @ 3.96m - Dec. 2, 2020)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

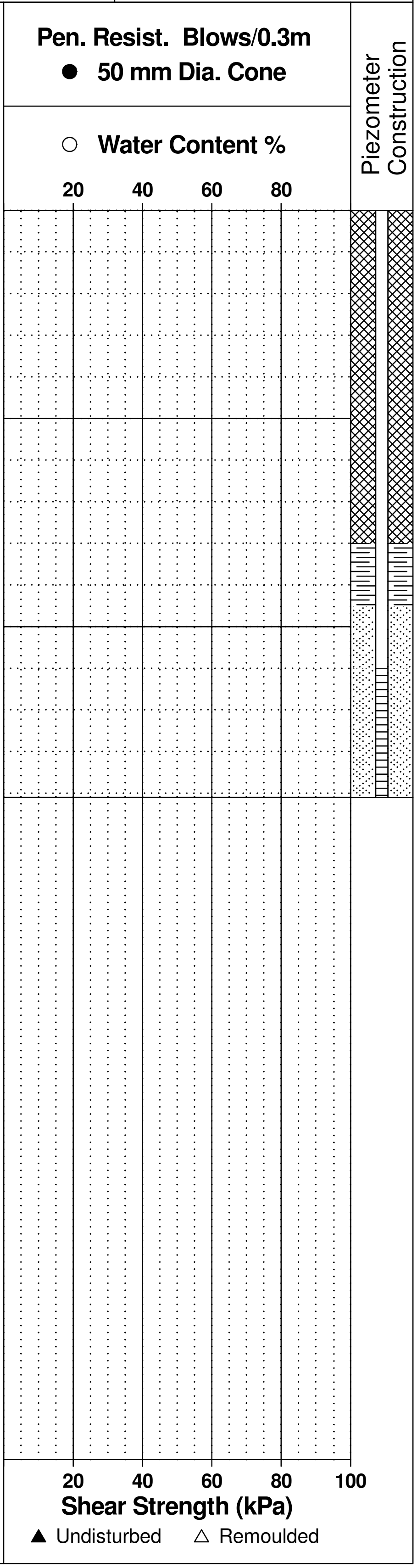
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

HOLE NO. BH 2-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	117.96
FILL: Crushed stone with silty sand and gravel	AU	1				
0.30						
	AU	2				
	SS	3	75	28	1	116.96
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders						
	SS	4	60	50+		
					2	115.96
	SS	5	27	50+		
	SS	6		50+		
2.82						
End of Borehole Practical refusal to augering at 2.82m depth (Piezometer blocked and dry at 2.04m depth - Dec. 2, 2020)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

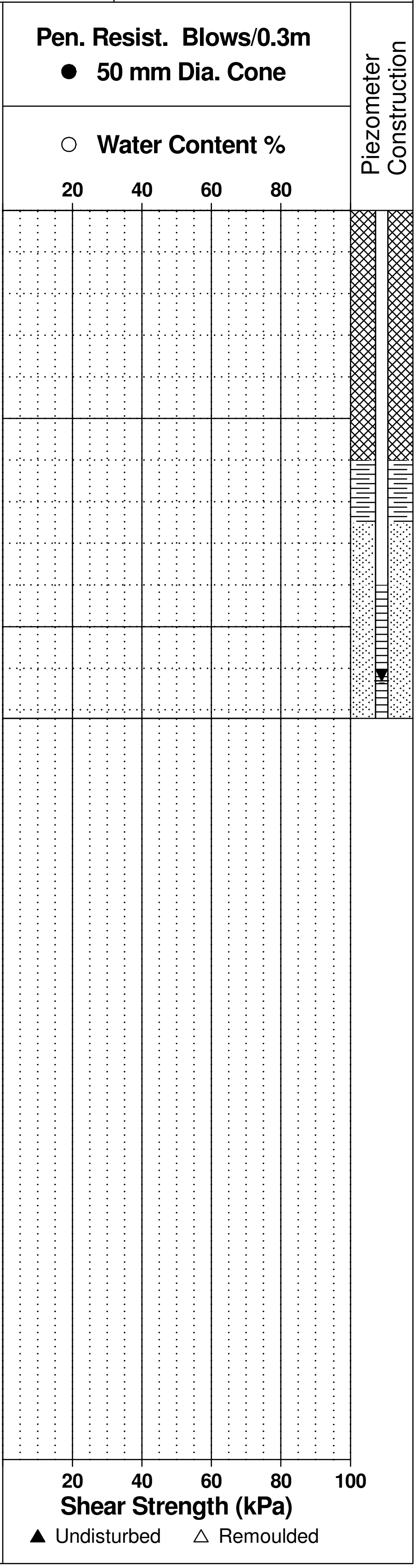
BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	117.39
FILL: Brown silty sand, some gravel	▩	AU	1				
0.60							
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	2	60	50+	1	116.39
		SS	3	22	50+		
						2	115.39
2.44		SS	4	100	50+		
End of Borehole							
Practical refusal to augering at 2.44m depth							
(GWL @ 2.26m - Dec. 2, 2020)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

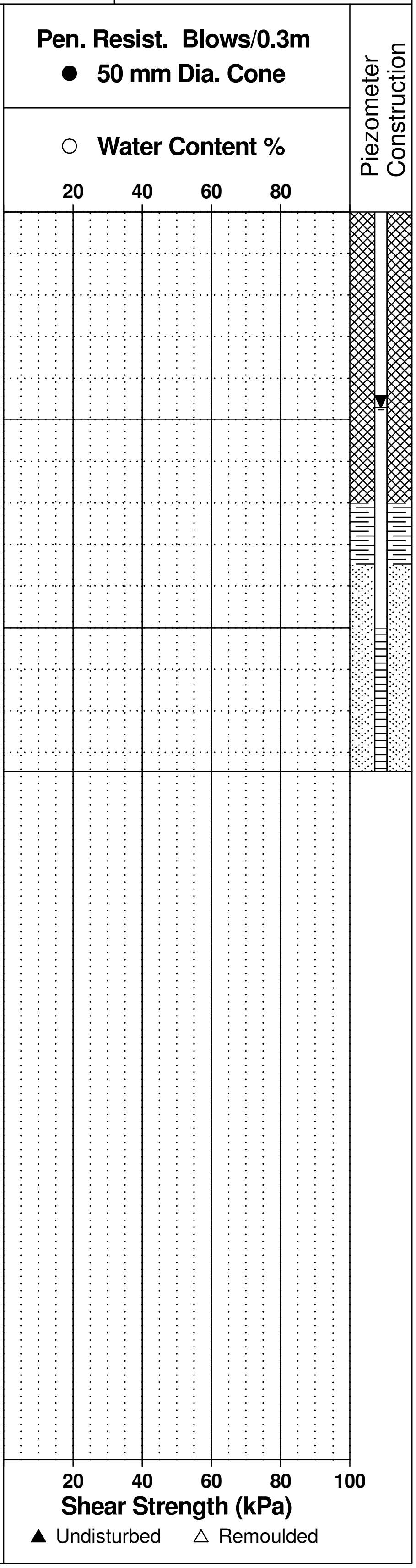
HOLE NO. BH 4-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	117.07
FILL: Brown silty sand with crushed stone — 0.30	AU	1				
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders	AU	2				
	SS	3	50	28	1	116.07
	SS	4	75	34	2	115.07
— 2.69	SS	5	50	50+		

End of Borehole

Practical refusal to augering at 2.69m depth

(GWL @ 0.94m - Dec. 2, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

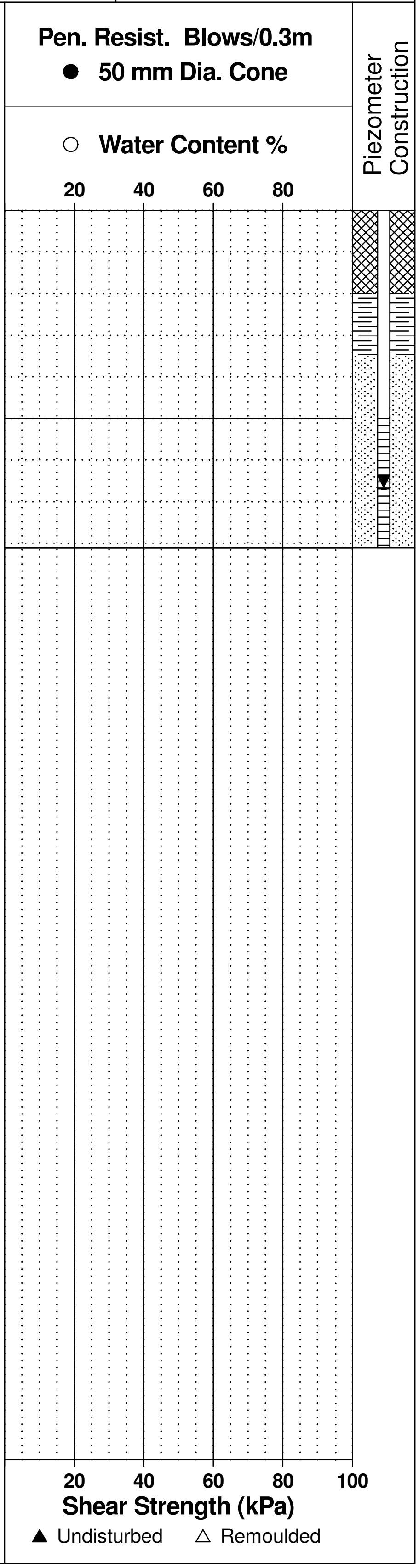
HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	117.98
FILL: Brown silty sand, some gravel, trace organics		AU	1				
0.60							
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders		SS	2	83	17	1	116.98
1.62		SS	3	100	50+		

End fo Borehole

Practical refusal to augering at 1.62m depth

(GWL @ 1.33m - Dec. 2, 2020)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

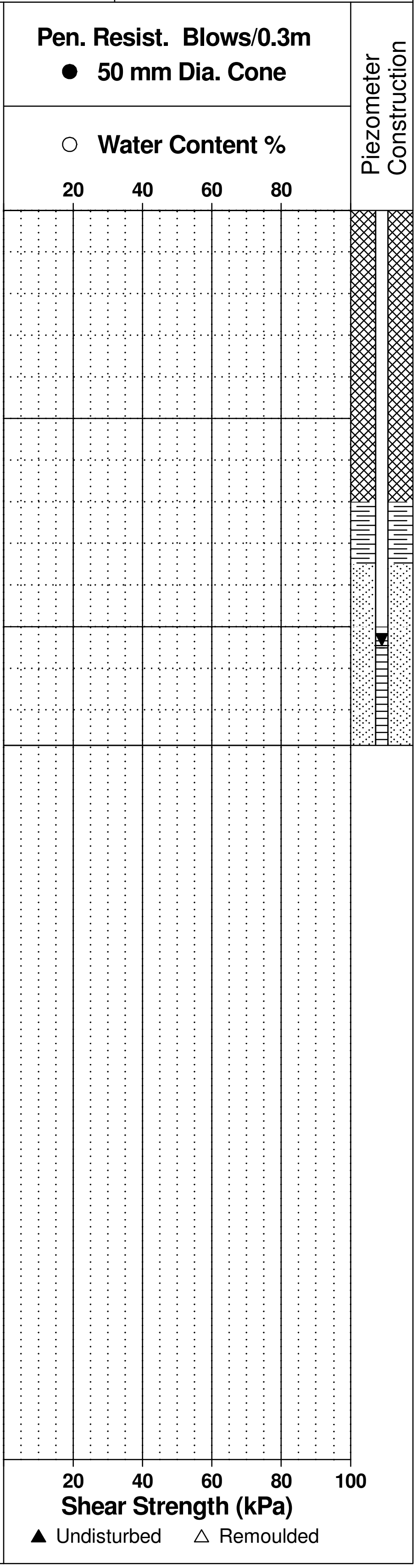
HOLE NO. BH 6-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	117.23
FILL: Crushed stone with silty sand, trace organics (to 0.30)	AU	1				
GLACIAL TILL: Compact to very dense, light brown silty sand with gravel, cobbles and boulders	SS	2	67	26	1	116.23
	SS	3	75	57	2	115.23
(to 2.57)	SS	4	73	50+		

End of Borehole

Practical refusal to augering at 2.57m depth

(GWL @ 2.09m - Dec. 2, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 2167 McGee Side Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 20, 2020

FILE NO. PG6662

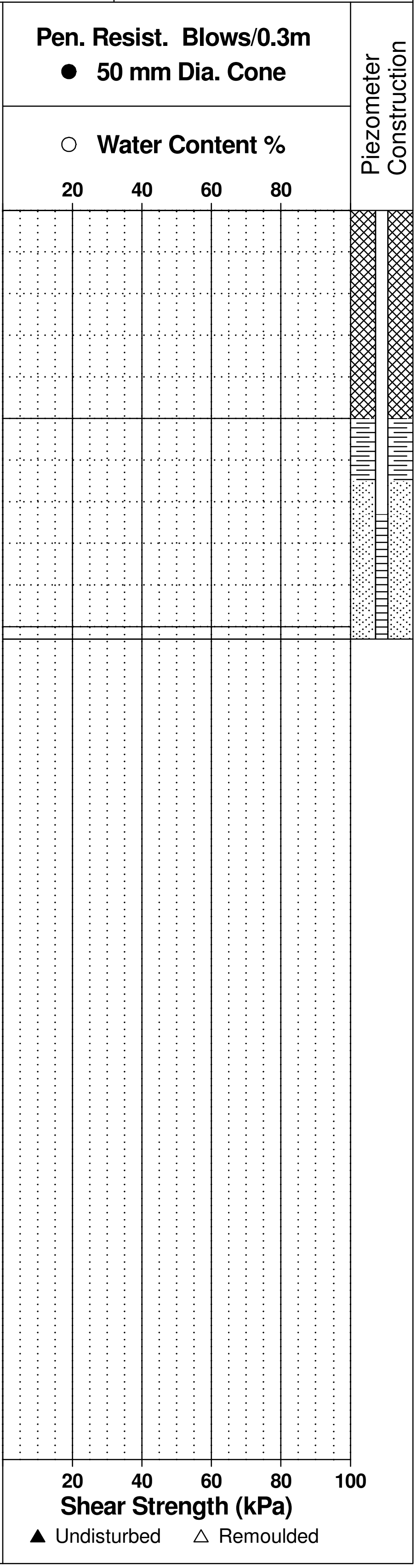
HOLE NO. BH 7-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	118.08
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boudlers	AU	1				
	AU	2				
	SS	3	92	34	1	117.08
	SS	4	75	50+	2	116.08
2.06						

End of Borehole

Practical refusal to augering at 2.06m depth

(Piezometer blocked and dry at 1.38m depth - Dec. 2, 2020)



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

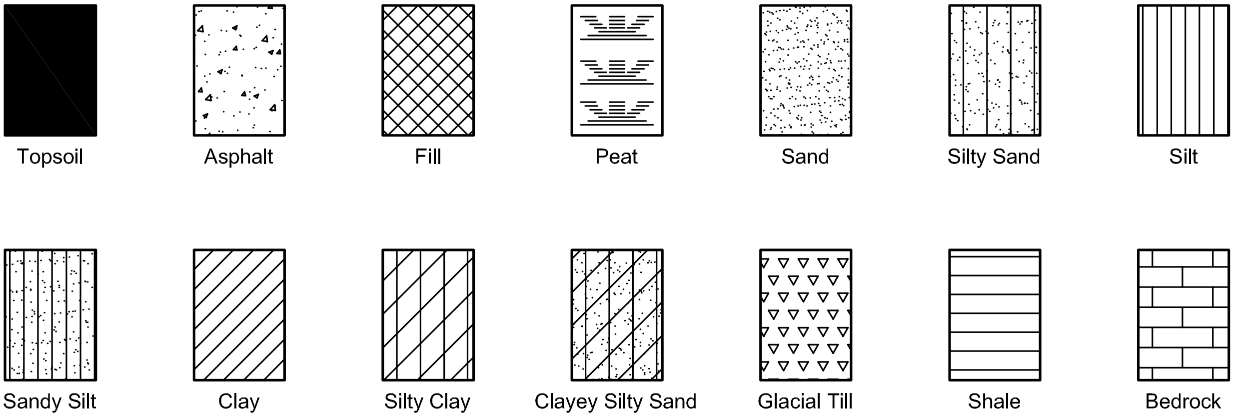
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

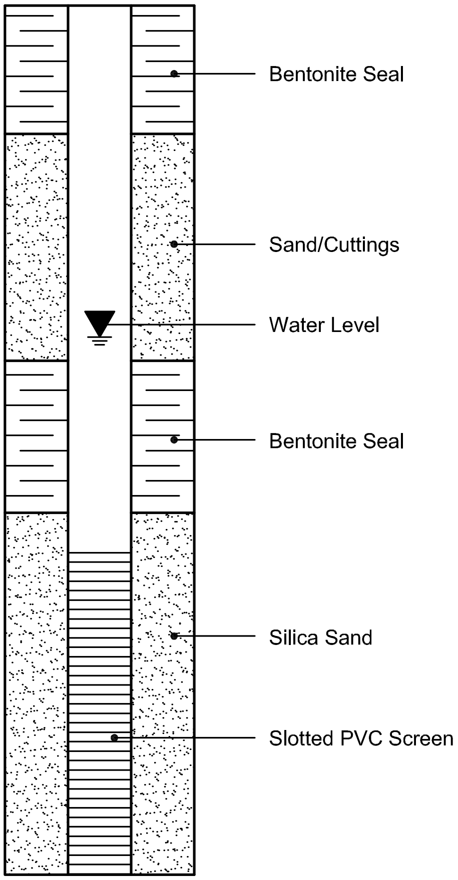
SYMBOLS AND TERMS (continued)

STRATA PLOT

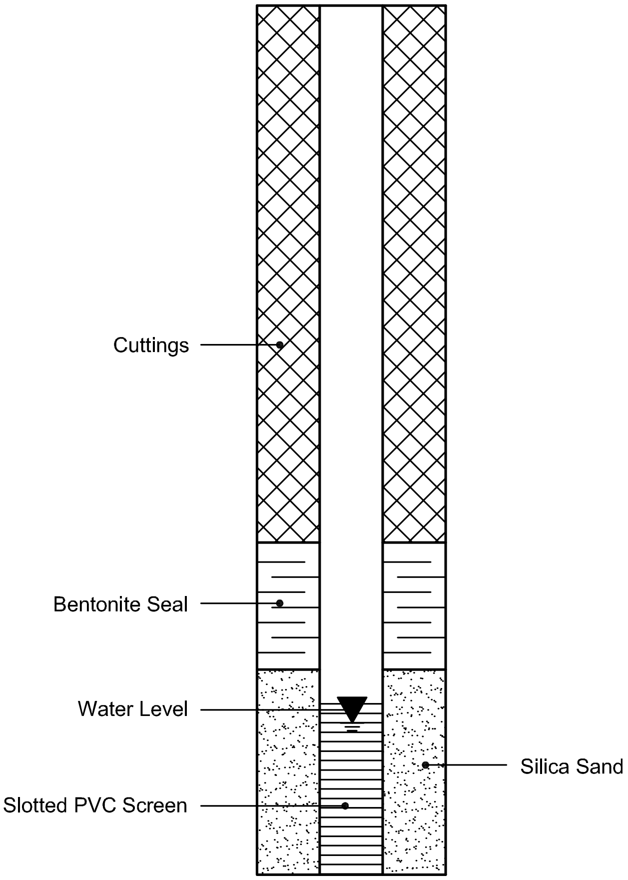


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

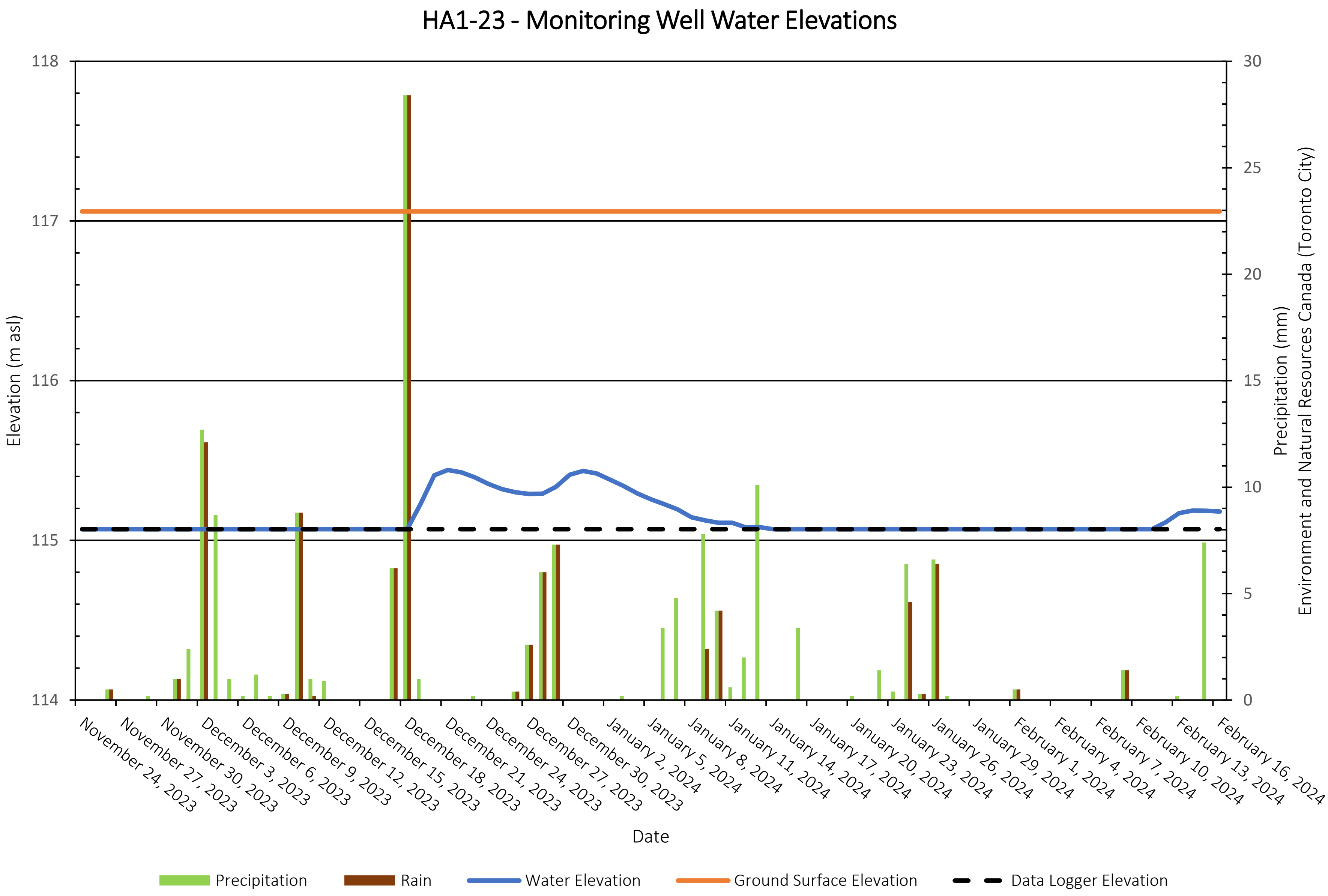
Client PO: 31283

Report Date: 27-Nov-2020

Order Date: 20-Nov-2020

Project Description: PG5602

Client ID:		BH3-20-SS2	-	-	-
Sample Date:		20-Nov-20 12:00	-	-	-
Sample ID:		2047669-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.75	-	-	-
Resistivity	0.10 Ohm.m	78.8	-	-	-
Anions					
Chloride	5 ug/g dry	9	-	-	-
Sulphate	5 ug/g dry	13	-	-	-


HA1-23 - Monitoring Well Water Elevations
Elevation (m asl)
118
117
116
115
114
Precipitation (mm)
Environment and Natural Resources Canada (Toronto City)
30
25
20
15
10
5
0
Date
November 24, 2023
November 27, 2023
November 30, 2023
December 3, 2023
December 6, 2023
December 9, 2023
December 12, 2023
December 15, 2023
December 18, 2023
December 21, 2023
December 24, 2023
December 27, 2023
December 30, 2023
January 2, 2024
January 5, 2024
January 8, 2024
January 11, 2024
January 14, 2024
January 17, 2024
January 20, 2024
January 23, 2024
January 26, 2024
January 29, 2024
February 1, 2024
February 4, 2024
February 7, 2024
February 10, 2024
February 13, 2024
February 16, 2024
Precipitation
Rain
Water Elevation
Ground Surface Elevation
Data Logger Elevation

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6662-1 – TEST HOLE LOCATION PLAN

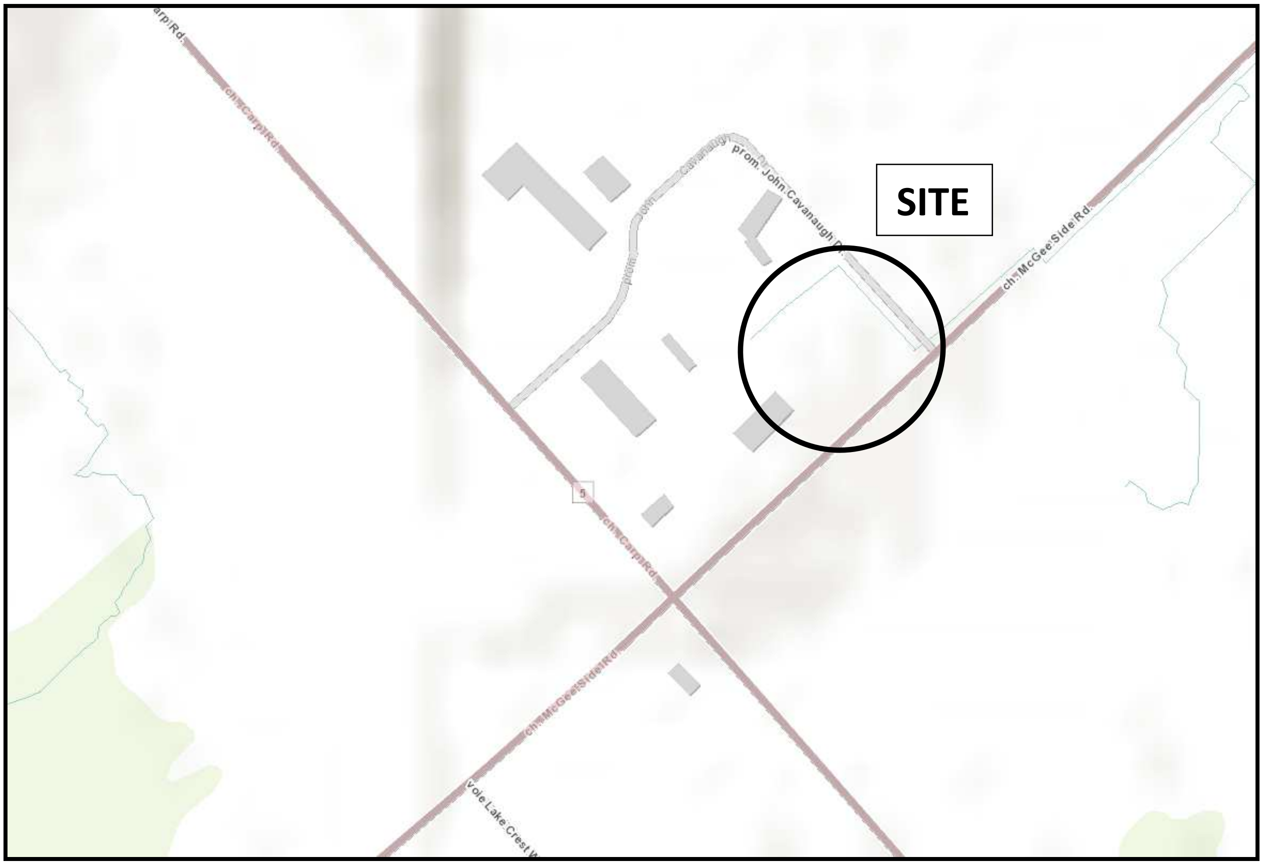
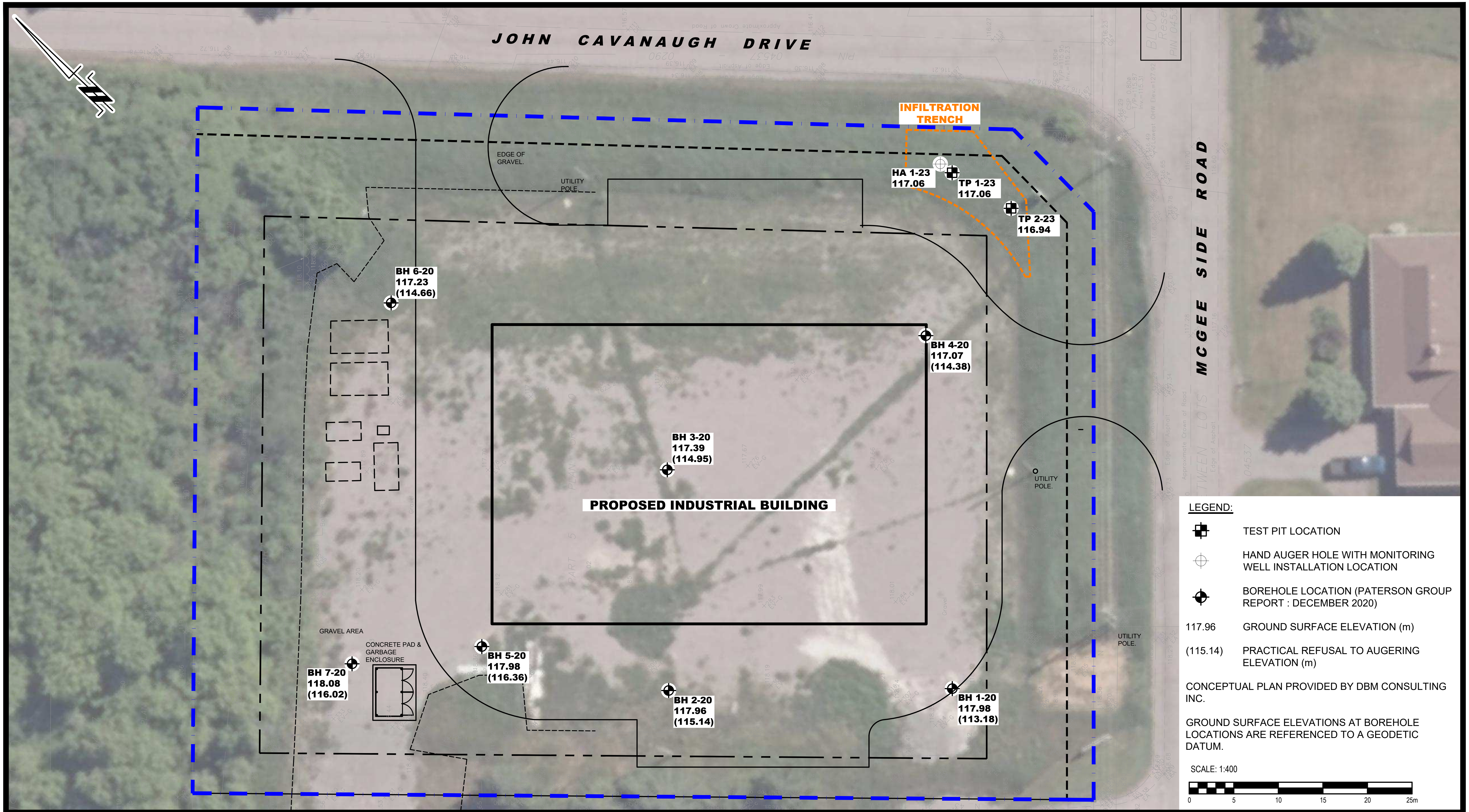


FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⯐	TEST PIT LOCATION
⌖	HAND AUGER HOLE WITH MONITORING WELL INSTALLATION LOCATION
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT : DECEMBER 2020)
117.96	GROUND SURFACE ELEVATION (m)
(115.14)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY DBM CONSULTING INC.

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

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	ADDED 2023 TEST PIT AND HAND AUGER HOLE LOCATIONS	08-02-2024	NZ

STOKED INDUSTRIES INC.
GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING
2167 MCGEE SIDE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 05/2023
Drawn by: YA
Report No.: PG6662-1
Checked by: ZA
Dwg. No.: PG6662-1
Approved by: NZ
Revision No.: 1

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11x17