Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Residential Development
Amberwood Village - 54 Springbrook Drive
Ottawa, Ontario

Prepared For

Amberwood Village Recreation Association

June 24, 2020

Report PG5408-1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

Table of Contents

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Analytical Testing Results

Appendix 2 Figure 1 - Key Plan
Drawing PG5408-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by the Amberwood Village Recreation Association to conduct a geotechnical investigation for the proposed residential development to be located in Amberwood Village at 54 Springbrook Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

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

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

Based on the available drawings, it is understood that the subject site is located north of the intersection of Trailway Circle and Snowy Owl Trail, and has an approximate footprint of 2,800 m^2. A 750 mm storm sewer, which has an approximate north-south alignment, is located to the northeast of the subject site.

Although drawings were not available during the preparation of this report, it is anticipated that the proposed residential development will consist of single-family residential dwellings and/or townhomes which will generally conform to the overall architectural character of the surrounding residential homes in Amberwood Village. It is also anticipated that the proposed residential development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on March 20, 2020 and consisted of 3 boreholes which were advanced to a maximum depth of 2.1 m below existing ground surface. All 3 boreholes were terminated due to practical refusal to augering. The boreholes were distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The locations of the boreholes are shown on Drawing PG5408-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using 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. The test hole procedures consisted of advancing the boreholes to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

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

Sample Storage

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

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of grate of the catch basin located on the north side of Trailway Circle, which was assigned an elevation of 100.00 m. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG5408-1 Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the boreholes and visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site, which is undeveloped and generally forested, is bordered by single family homes to the north and south, the Amberwood Village Golf and Country Club to the west, and Trailway Circle to the east. Further, a 750 mm storm sewer is located to the northeast of the subject site, behind the homes on Eagle Rock Way, with an approximate north-south alignment. The existing ground surface across the site is relatively level with a slight downslope towards the northwest.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 0.7 to 0.8 m thickness of topsoil underlain by fill which extends to approximate depths of 1.5 to 1.7 m below the existing ground surface. The fill was generally observed to consist of a loose, brown silty sand to sandy silt with trace to some gravel and trace organics.

A glacial till deposit was encountered underlying the fill and was observed to consist of a compact to dense, brown silty sand with some gravel and trace clay.

Practical refusal to augering was encountered in all boreholes at depths ranging from approximately 1.8 to 2.1 m below the existing ground surface.

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 available geological mapping, the bedrock at the subject site consists of limestone of the Bobcaygeon Formation.

4.3 Groundwater

Based on groundwater level measurements, field observations during the boreholes, knowledge of the groundwater within the local area of the subject site, and the recovered soil samples' moisture levels, consistency and colouring, the long-term groundwater table can be expected between an approximate 2 to 3 m depth below ground surface.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed buildings. It is recommended that the proposed buildings be constructed with conventional shallow footings bearing on the undisturbed, compact to dense glacial till or clean, surface sounded bedrock.

Where fill is encountered at the underside of footing, it should be sub-excavated to the surface of the undisturbed, compact to dense glacial till or clean, surface sounded bedrock and replaced with engineered fill or lean concrete to the proposed founding elevation. The lateral limits of the engineered fill or lean concrete placement should be in accordance with our lateral support recommendations provided herein.

Bedrock removal may be required for the building excavation and installation of site services, dependent on the depths of the proposed structures and utilities. 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 or fill, containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

Bedrock Removal

Should bedrock removal be required, hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be conducted prior to commencing construction.

The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. It is also recommended that vibration monitoring be conducted at adjacent structures, including the storm sewer, should blasting be employed at the subject site.

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

Vibration Considerations

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

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

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill used for grading beneath the proposed 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 building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, 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.

Lean Concrete Filled Trenches

As an alternative to placing engineered fill, where required, consideration should be given to excavating vertical trenches to the undisturbed, compact to dense glacial till or clean, surface sounded bedrock, and backfilling with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying glacial till or bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact to dense glacial till bearing surface, or on engineered fill or lean concrete placed directly over the undisturbed, compact to dense glacial till, can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

Footings placed on a soil bearing surface and designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

Footings placed on clean, surface sounded bedrock, or on lean concrete placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

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

Adequate lateral support is provided to glacial till 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 of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

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 buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

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

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the existing fill, glacial till, or bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

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

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

5.6 Pavement Structure

Where required at the subject site, the recommended pavement structures for car only parking areas, access lanes and heavy truck parking areas are shown in Tables 1 and 2.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 2 - 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
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or 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 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 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 which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to a catch basin.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or Miradrain G100N. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should be placed for this purpose. A waterproofing system should be provided to the elevator pit (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

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

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

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

Where the bedrock is considered frost susceptible, foundation insulation will need to be provided by Paterson, upon request. Alternatively, the frost susceptible bedrock can to be removed and replaced with lean concrete (minimum 17 MPa 28-day strength).

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill 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 expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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 Services Branch of the City of Ottawa.

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

The cover material, which should consist 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 a minimum of 95% of its SPMDD.

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

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.

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, 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.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Recommendations

Slopes at the subject site are expected to be stable under long term conditions between approximately 3H:1V to 5H:1V. The soils encountered during this investigation should be considered to be stable at the design slopes provided.

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 used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation of this nature is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 its 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 the Amberwood Village Recreation Association or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Yolanda Tang

LICENSED PROFESSIONAL ENGINEER
June 24, 2020
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

Report Distribution

- ❏ Amberwood Village Recreation Association (e-mail copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
54 Springbrook Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the north side of Trailway Circle. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 15 Power Auger

DATE March 20, 2020

FILE NO. PG5408

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	99.88		
TOPSOIL		AU	1						
0.76									
FILL: Brown sandy silt, trace organics		SS	2	21	7	1	98.88		
1.65									
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, trace clay		SS	3	50	50+				
1.83									
End of Borehole Practical refusal to augering at 1.83m depth.									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
54 Springbrook Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the north side of Trailway Circle. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 15 Power Auger

DATE March 20, 2020

FILE NO. PG5408

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.94
TOPSOIL (to 0.76)		AU	1				
FILL: Brown silty sand, some gravel, trace organics (0.76 to 1.65)		SS	2	33	8	1	98.94
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, trace clay (1.65 to 1.83)		SS	3	67	50+		
End of Borehole Practical refusal to augering at 1.83m depth.							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
54 Springbrook Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the north side of Trailway Circle. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

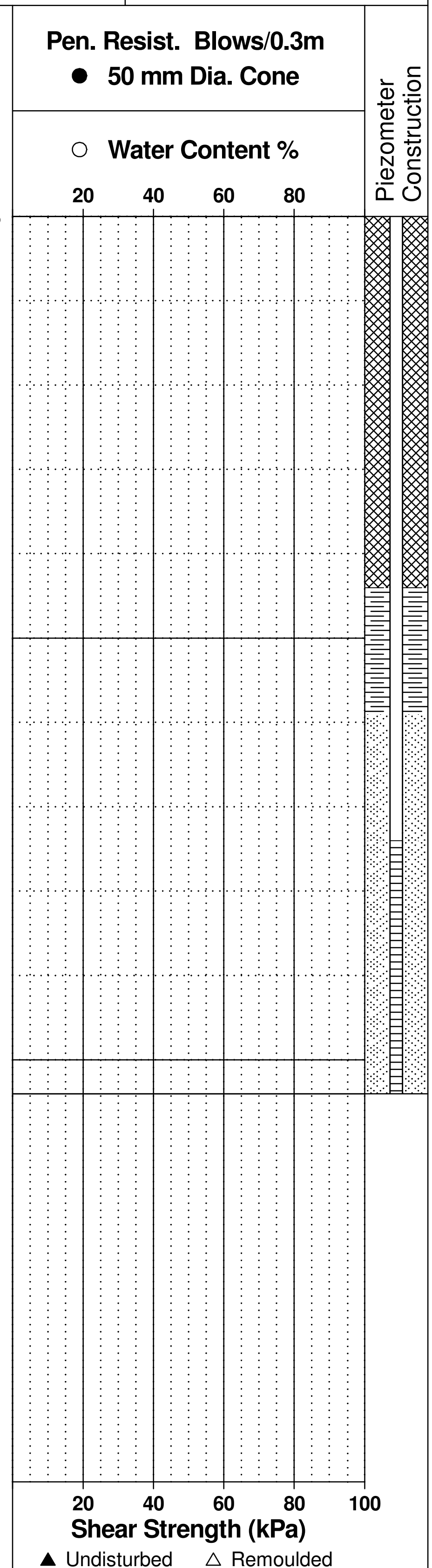
BORINGS BY CME 15 Power Auger

DATE March 20, 2020

FILE NO. PG5408

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.06
TOPSOIL		AU	1				
0.76							
FILL: Brown silty sand, trace gravel and organics		SS	2	21	4	1	99.06
1.52							
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, trace clay		SS	3	55	50+	2	98.06
2.08							
End of Borehole Practical refusal to augering at 2.08m depth.							



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

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

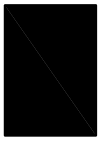
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

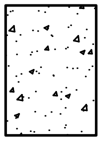
PERMEABILITY TEST

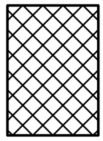
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

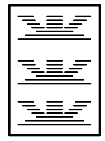
SYMBOLS AND TERMS (continued)

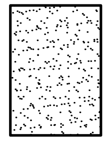
STRATA PLOT

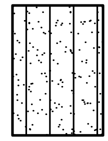

Topsoil

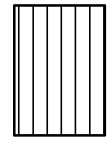

Asphalt

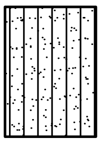

Fill

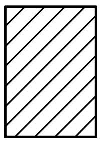

Peat

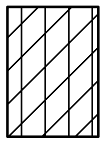

Sand

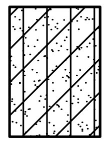

Silty Sand

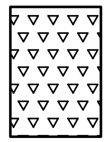

Silt

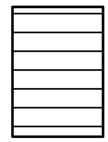

Sandy Silt

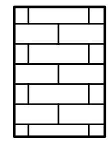

Clay


Silty Clay


Clayey Silty Sand

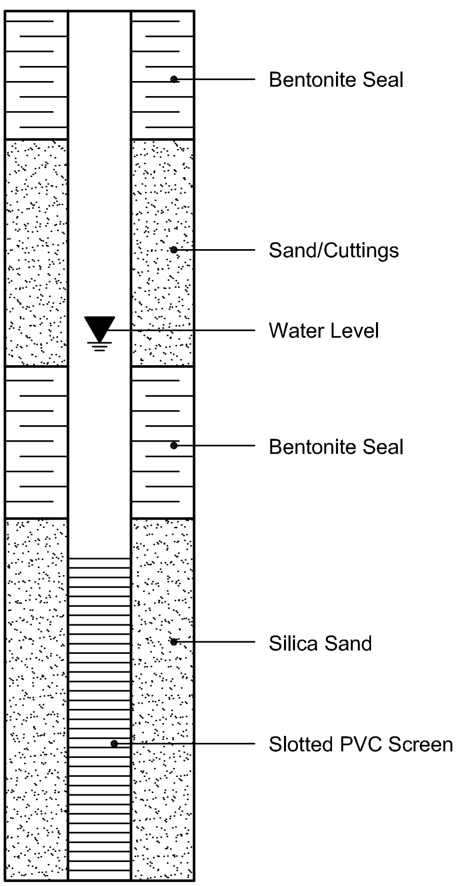

Glacial Till


Shale

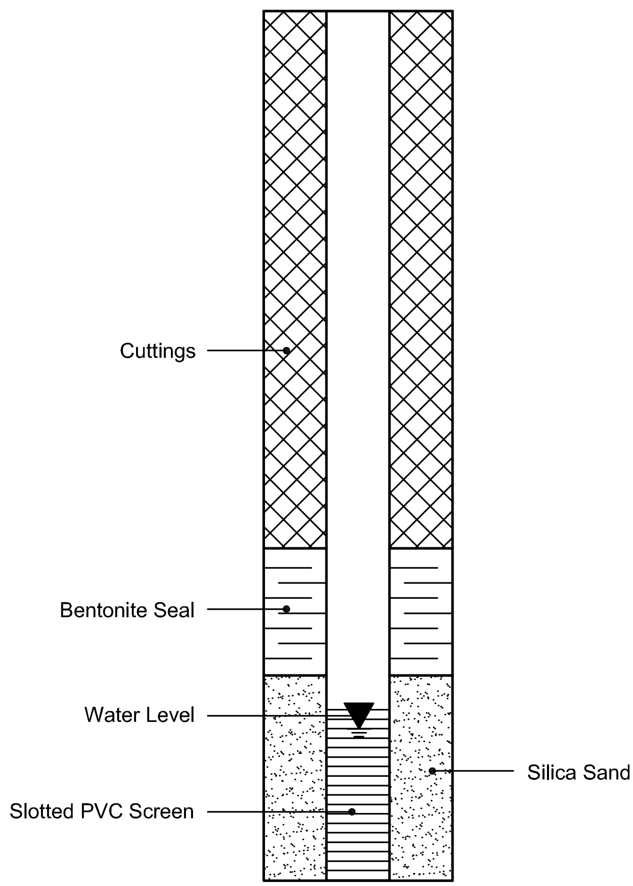

Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29705

Report Date: 30-Mar-2020

Order Date: 24-Mar-2020

Project Description: PE5296

	Client ID:	BH1+BH2 SS3 5'-7'	-	-	-
	Sample Date:	20-Mar-20 10:00	-	-	-
	Sample ID:	2013103-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.75	-	-	-
Resistivity	0.10 Ohm.m	42.1	-	-	-
Anions					
Chloride	5 ug/g dry	42	-	-	-
Sulphate	5 ug/g dry	74	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5408-1 - TEST HOLE LOCATION PLAN

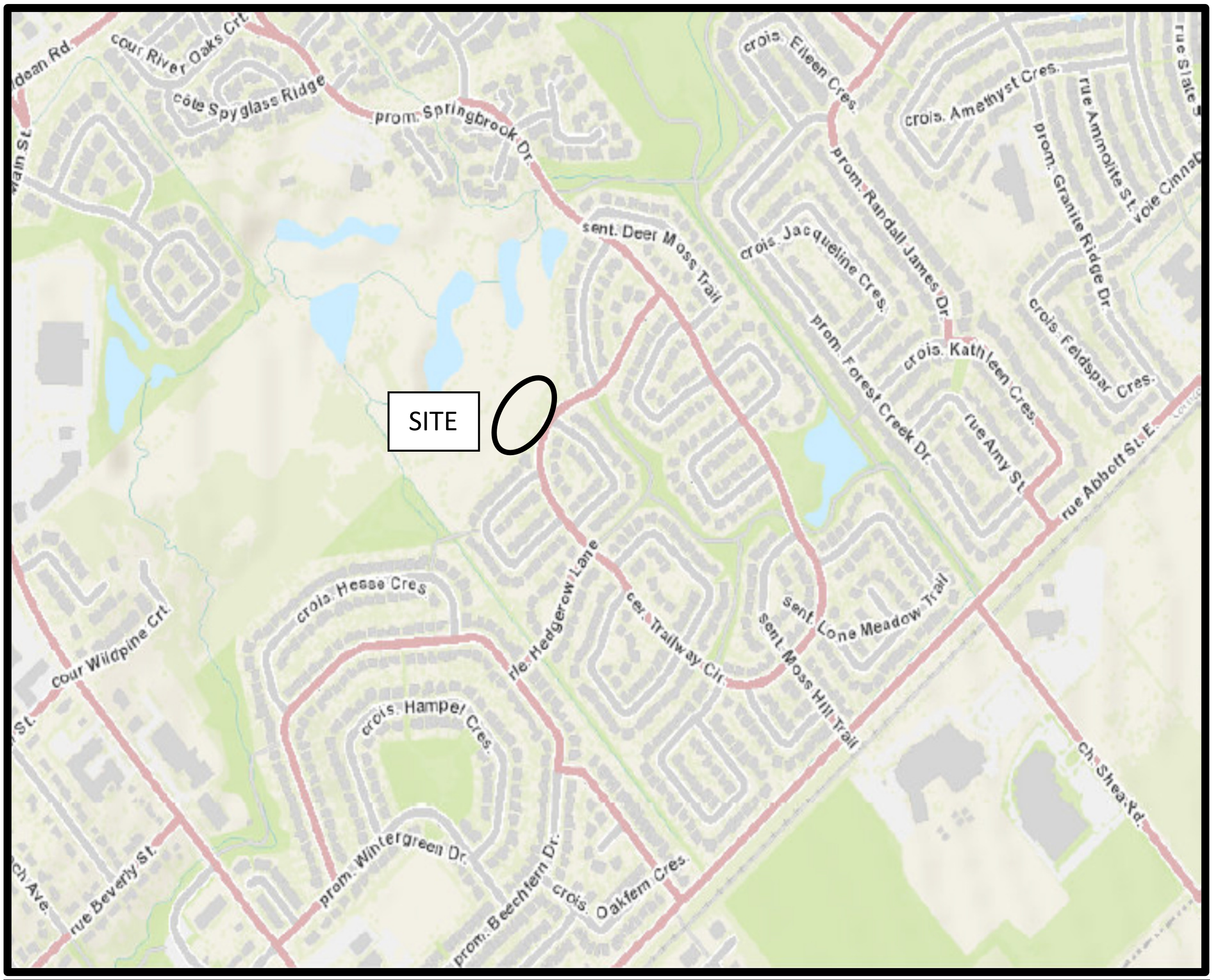
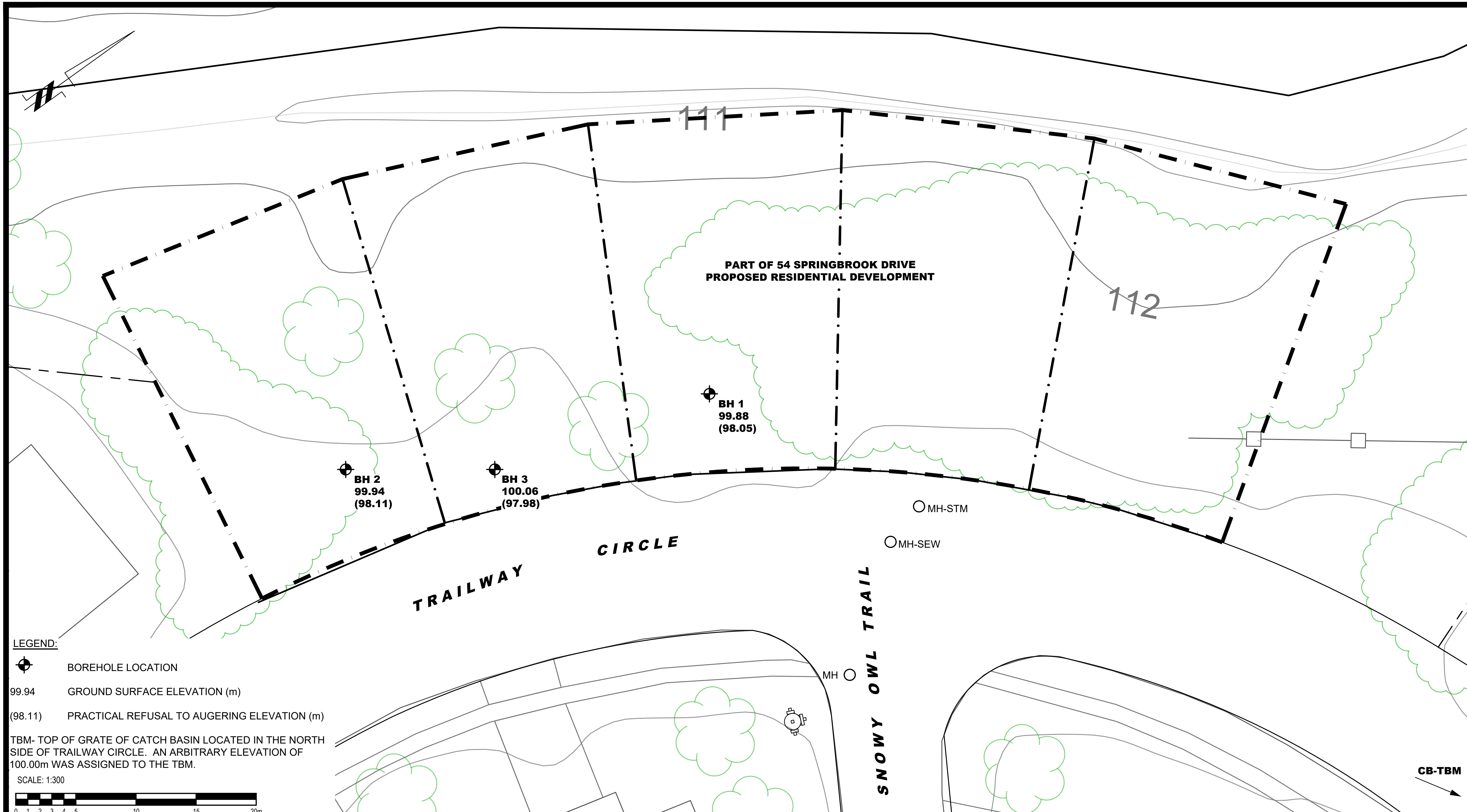


FIGURE 1

KEY PLAN



LEGEND:

BOREHOLE LOCATION

99.94 GROUND SURFACE ELEVATION (m)

(98.11) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM- TOP OF GRATE OF CATCH BASIN LOCATED IN THE NORTH SIDE OF TRAILWAY CIRCLE. AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

AMBERWOOD VILLAGE RECREATION ASSOCIATION

GEOTECHNICAL INVESTIGATION
54 SPRINGBROOK DRIVE - PROPOSED RESIDENTIAL DEVELOPMENT
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 06/2020
Drawn by: YA	Report No.: PG5408-1
Checked by: YT	Dwg. No.: **PG5408-1**
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg54xx\pg5408\pg5408-1-test hole location plan.dwg