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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Curling Club
2740 Queensview Drive
Ottawa, Ontario

Prepared For

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May 19, 2021

Report: PG4353-2

Table of Contents

PAGE

1.0 Introduction **1**
2.0 Proposed Development **1**
3.0 Method of Investigation **2**
3.1 Field Investigation 2
3.2 Field Survey 3
3.3 Laboratory Testing 3
3.4 Analytical Testing 3
4.0 Observations **4**
4.1 Surface Conditions 4
4.2 Subsurface Profile 4
4.3 Groundwater 4
5.0 Discussion **6**
5.1 Geotechnical Assessment 6
5.2 Site Grading and Preparation 6
5.3 Foundation Design 7
5.4 Design for Earthquakes 7
5.5 Slab-on-Grade Construction 8
5.6 Pavement Design 8
6.0 Design and Construction Precautions **10**
6.1 Foundation Drainage and Backfill 10
6.2 Protection of Footings Against Frost Action 10
6.3 Excavation Side Slopes 10
6.4 Pipe Bedding and Backfill 11
6.5 Groundwater Control 12
6.6 Winter Construction 13
6.7 Corrosion Potential and Sulphate 13
7.0 Recommendations **14**
8.0 Statement of Limitations **15**

Appendices

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

Appendix 2 Figure 1 - Key Plan
Drawing PG4353-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Morley Hoppner to conduct a geotechnical investigation for the proposed curling club to be located at 2740 Queensview Drive in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a low-rise recreational building with a slab-on-grade, which will be located on the eastern portion of the site. Associated access lanes, parking areas, walkways, and landscaped areas are also proposed surrounding the building. It is also expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on December 8, 2017. At that time, a total of three (3) boreholes were advanced to a maximum depth of 7.0 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the proposed development with consideration of existing site features and underground utilities. The locations of the boreholes are shown on Drawing PG4353-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a rubber track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered directly from the auger flights or using a 50 mm diameter split spoon sampler. Rock cores were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard rock core boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The Standard Penetration Test (SPT) was carried out In conjunction with the recovery of the split spoon samples. The SPT results are recorded as “N” values on the Soil Profile & 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 recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

Flexible polytubing piezometers were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of existing site features and underground utilities. The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top of a catch basin located in the existing parking lot. An assumed elevation of 100.00 m was assigned to the TBM. The locations of the test holes and TBM are presented on Drawing PG4353-1 - 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 field logs. The results are presented on the Soil Profile and Test Data sheets in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are further discussed in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a one-storey commercial slab-on-grade building located within the west portion of the subject site. The remainder of the subject site consists of an asphalt surfaced parking lot and various landscaped areas. The ground surface was noted to generally slope downward gradually from south to north towards Queensview Drive. The subject site is bordered by a commercial property to the east, Highway 417 off-ramp to Pinecrest Road to the south, a church to the west and Queensview Drive to the north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a layer of asphalt overlying granular fill, which extends to between 0.6 and 1.8 m depth, with the exception of borehole BH 1 where a silty clay layer was encountered underlying the topsoil and extending to an approximate depth of 0.8 m.

Underlying the fill or silty clay, a glacial till deposit was encountered which consisted of a loose to very dense, grey silty clay to silty sand with gravel, cobbles, and boulders. Practical refusal to augering was encountered at the borehole locations at depths ranging from 2.8 to 3.4 m below existing ground surface.

Bedrock

At boreholes BH 2 and BH 3, a weathered shale bedrock was encountered underlying the glacial till. Further, at borehole BH 3, limestone bedrock was encountered underlying the shale at an approximate depth of 3.3 m. The shale bedrock was generally weathered and of poor quality, while the limestone bedrock was of good to excellent quality, based on the RQD values.

Based on available geological mapping, the bedrock in this area consists of shale from the Rockcliffe formation, with overburden drift thicknesses ranging from 2 to 5 m.

4.3 Groundwater

Groundwater levels were measured in the piezometers installed in each borehole on December 15, 2017. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Summary of Groundwater Levels				
Borehole Number	**Ground Elevation (m)**	**Groundwater Levels (m)**		**Recording Date**
		Depth	**Elevation**	
BH 1	101.478	2.42	99.06	December 15, 2017
BH 2	100.528	2.59	97.94	December 15, 2017
BH 3	100.114	2.17	97.94	December 15, 2017

Based on field observations of the recovered soil samples, such as moisture levels, colouring and consistency, the long-term groundwater level is anticipated between a 2 to 3 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Details of the groundwater levels are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings bearing on the undisturbed, compact glacial till.

Where fill is encountered at the underside of footing, it should be sub-excavated to the surface of the undisturbed, compact glacial till and replaced with engineered fill 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.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill within the future building footprint, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times **under dry conditions and above freezing temperatures** and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 99% of the standard Proctor maximum dry density (SPMDD).

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. Site excavated soils are not suitable for use as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

5.3 Foundation Design

Footings bearing on an undisturbed, compact glacial till, or on engineered fill which is placed directly over the undisturbed, compact 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**. The bearing resistance values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a glacial till or engineered fill bearing surface when a plane extending down and out from the bottom edges of the footings, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. A higher site class, such as Class A or B, may be applicable for this site. However, the higher site class must be confirmed by a site specific shear wave velocity test. The soils underlying the subject site are not susceptible to liquefaction. Refer to the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill, silty clay, and/or glacial till subgrades, approved by the geotechnical consultant at the time of excavation, will be considered acceptable subgrades on which to commence backfilling for slab-on-grade construction. Where the slab-on-grade 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 OPSS Granular B Type II.

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

5.6 Pavement Design

Car only parking areas and heavy traffic access areas are proposed at this site. The subgrade material will consist of existing fill, silty clay, and/or glacial till. The proposed pavement structures are presented in Tables 2 and 3 below.

Table 2 – 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 3 – 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	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

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

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 compaction equipment.

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 A or Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

A milled step joint should be provided in existing asphalt to provide proper tie-in where new and existing pavements abut. The step joint should be 300 mm wide by 50 mm deep, and provided with a light tack coat consisting of SS-1 emulsified asphalt to ensure proper bonding of the new and existing asphalt pavement.

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 structure. The system should consist of a 100 to 150 mm diameter perforated and corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone and is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, 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. A minimum of 2.1 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided for all exterior unheated footings.

6.3 Excavation Side Slopes

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

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 and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

The site excavated material may be placed above cover material if the excavation operations are completed in dry weather conditions and the site excavated material is approved by the geotechnical consultant. All cobbles greater than 200 mm in the longest dimension should be removed prior to the materials being reused. The shale bedrock is not recommended for placement as backfill material.

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 reduce 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 SPMDD.

Within the frost zone, non-frost susceptible materials should be used when backfilling trenches below the original bedrock level.

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Infiltration levels are anticipated to be low to moderate through the excavation face, depending on the local groundwater table. The groundwater infiltration is anticipated to be controllable with open sumps and pumps.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Structures

The excavation for the proposed building is not anticipated to extend below the groundwater level, and dewatering is only anticipated to be required following storm events. Therefore, there will be no adverse effects to the surrounding buildings or properties.

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. Ice could form within the soil mass in presence of water and freezing conditions. 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.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

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

8.0 Statement of Limitations

The recommendations provided herein are in accordance with our 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 Morley Hoppner or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Owen Canton, E.I.T.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2740 Queensview Drive
Ottawa, Ontario

DATUM TBM - Top of catch basin located near the northwest corner of existing building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Track-mount Drill

DATE December 8, 2017

FILE NO. PG4353

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 %	Piezometer Construction
GROUND SURFACE						0	101.48		
TOPSOIL 0.30		AU	1						
Very stiff, brown **SILTY CLAY,** trace organics 0.76									
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	2	58	6	1	100.48		
		SS	3	83	21	2	99.48		
2.82		SS	4	100	50+				
End of Borehole									
Practical refusal to augering at 2.82m depth									
(GWL @ 2.42m - Dec. 12, 2017)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
2740 Queensview Drive
Ottawa, Ontario

DATUM TBM - Top of catch basin located near the northwest corner of existing building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Track-mount Drill

DATE December 8, 2017

FILE NO. PG4353

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.53
Asphaltic concrete	0.06						
FILL: Grey sand and gravel, some silt	0.60	AU	1				
GLACIAL TILL: Very dense, grey silty sand, some gravel, cobbles and boulders	1.83	SS	2	62	12	1	99.53
		SS	3	100	50+		
BEDROCK: Weathered shale	3.35					2	98.53
		SS	4	100	50+		
		SS	5	100	50+	3	97.53

End of Borehole

Practical refusal to augering at 3.35m depth

(GWL @ 2.59m - Dec. 12, 2017)

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
2740 Queensview Drive
Ottawa, Ontario

DATUM TBM - Top of catch basin located near the northwest corner of existing building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Track-mount Drill

DATE December 8, 2017

FILE NO. PG4353

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.11
Asphaltic concrete (0.06)						
FILL: Grey sand and gravel (0.76)	AU	1				
FILL: Brown silty sand (1.83)	SS	2	67	4	1	99.11
	SS	3	100	33		
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders (2.44)					2	98.11
BEDROCK: Weathered shale (3.25)	SS	4	100	50+		
	SS	5	75	50+	3	97.11
BEDROCK: Grey limestone (6.96)	RC	1	100	76	4	96.11
	RC	2	100	88	5	95.11
	RC	3	100	80	6	94.11
End of Borehole						
(GWL @ 2.17m - Dec. 18, 2017)						

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

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

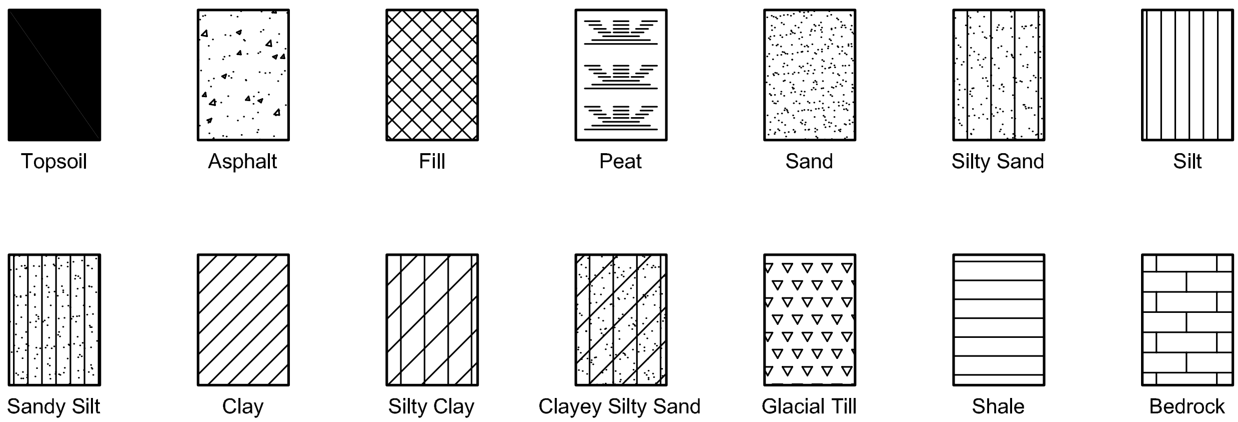
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

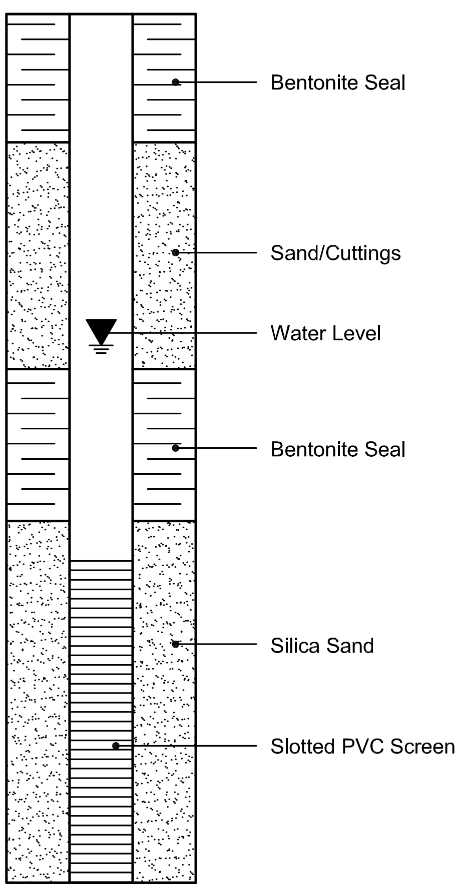
SYMBOLS AND TERMS (continued)

STRATA PLOT

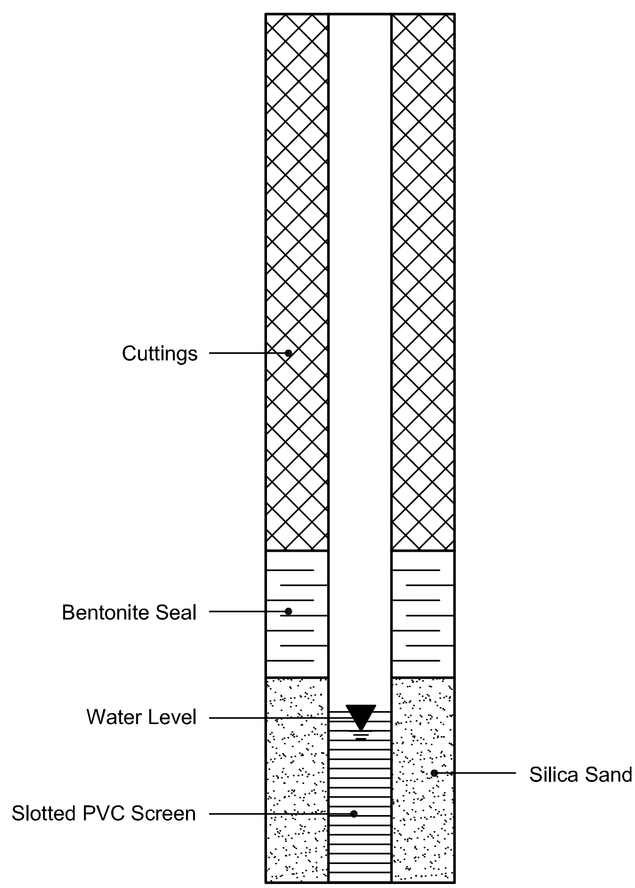


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 23304

Report Date: 21-Dec-2017
Order Date: 15-Dec-2017
Project Description: PG4353

	Client ID:	BH1 SS3	-	-	-
	Sample Date:	08-Dec-17	-	-	-
	Sample ID:	1750506-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.00	-	-	-
Resistivity	0.10 Ohm.m	49.4	-	-	-
Anions					
Chloride	5 ug/g dry	28	-	-	-
Sulphate	5 ug/g dry	12	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG4353-1 – TEST HOLE LOCATION PLAN

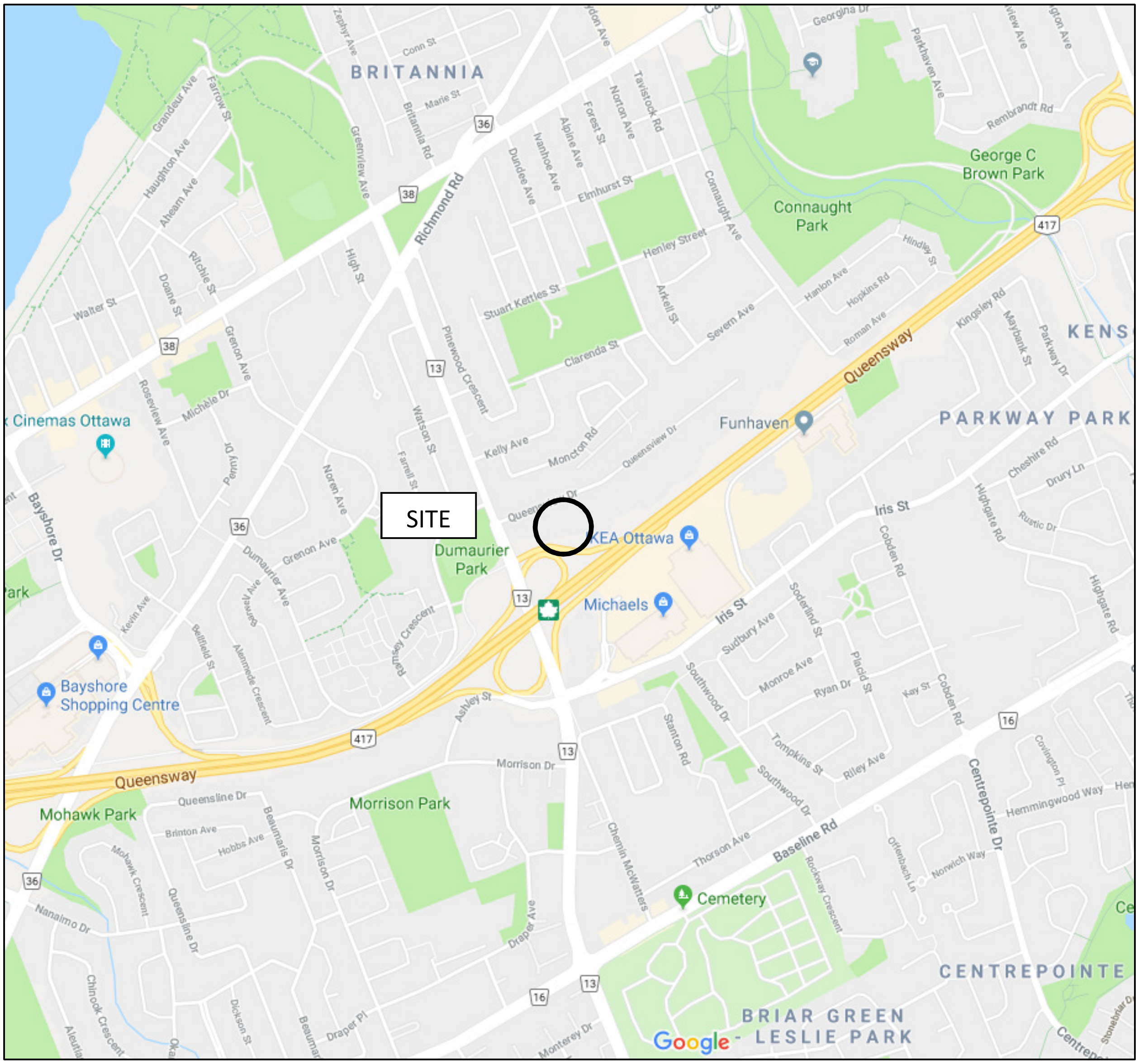
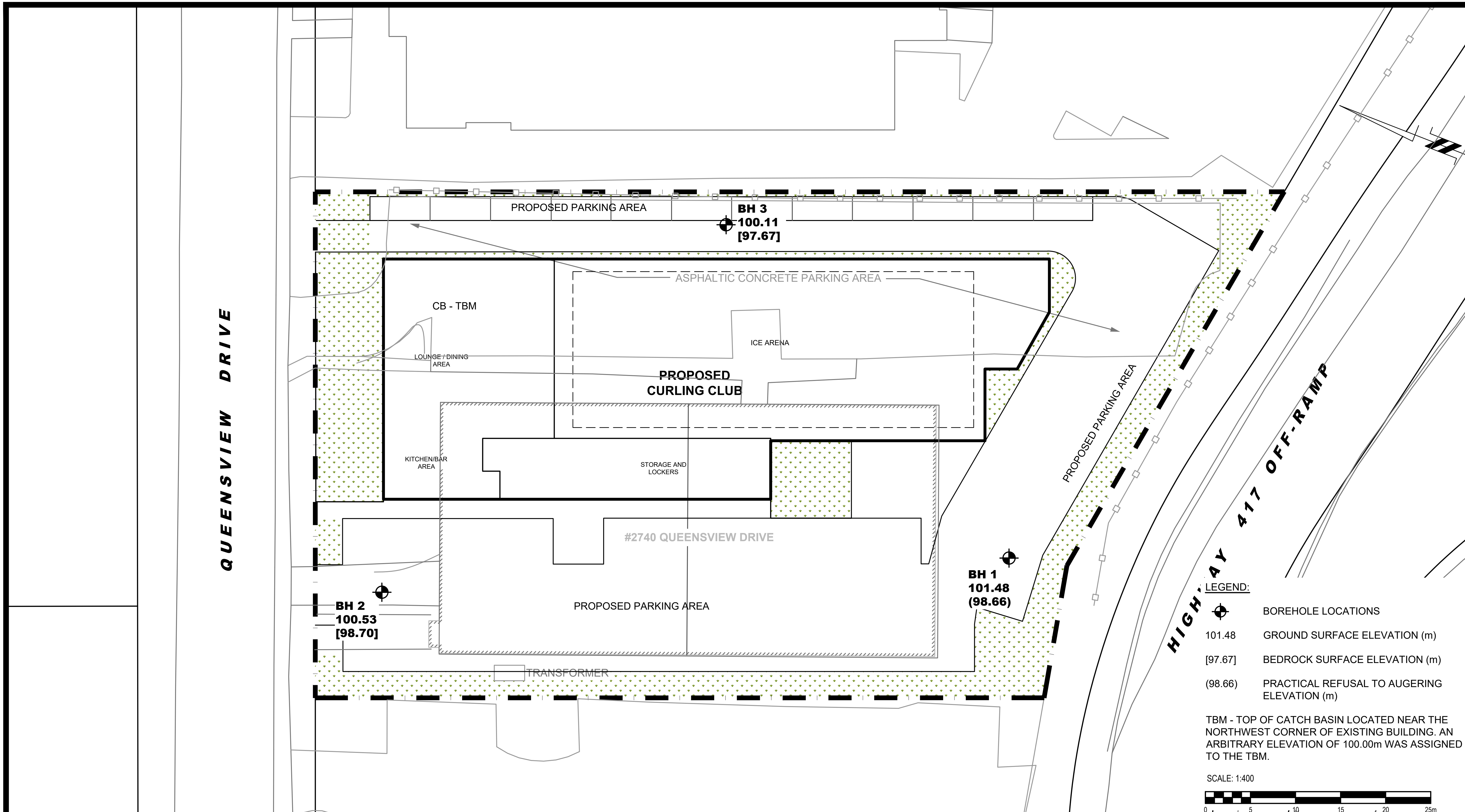


FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST CONCEPTUAL PLAN	03/05/2021	OC

MORLEY HOPPNER

GEOTECHNICAL INVESTIGATION
2740 QUEENSVIEW DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 12/2017
Drawn by: RCG	Report No.: PG4353-2
Checked by: OC	Dwg. No.: PG4353-1
Approved by: SD	Revision No.: 1

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