Geotechnical Investigation

Proposed Warehouse Building

96 Bill Leathem Drive,
Ottawa, Ontario

Prepared for Prestige Construction

Report PG6668 – 1 dated June 1, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Prestige Construction to conduct a geotechnical investigation for the proposed residential development to be located at 96 Bill Leathem Drive in Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

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

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on our review of available information, it is anticipated the proposed development will be two-storey warehouse building over an area of 1346 m^2, with a light-duty and a heavy-duty parking area and general landscaping surrounding the building.

It is anticipated that the site will be municipally serviced by future water, storm and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on May 17, 2023, and consisted of a total of three (3) boreholes sampled to a maximum depth of 6.5 m below ground surface throughout the subject site. Furthermore, one (1) borehole BH 2, was previously completed within the site footprint as a part of the geotechnical investigation completed on May 9, 2008, for a different project.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG6668-1 - 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 drilling procedure consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sample. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets.

The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at one borehole location. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

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.

Groundwater

Flexible standpipe piezometers were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6668-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging.

All samples were submitted for moisture content testing. The test results are included on the Soil Profile and Test Data sheets presented in Appendix 1.
All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. 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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant with grassed terrain and some granulars at grade. A granular access path had been installed on the east portion of the site. A pile of granular fill was noted to have been left along the south portion of the site. The subject site is surrounded by a vacant parcel of land along the east, a stormwater management pond along the south, a two-storey commercial building along the west and Bill Leathem Drive along the north. The ground surface across the site is relatively flat at an approximate geodetic elevation of 90.10 to 90.20 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of topsoil overlying a deep silty clay deposit. Practical refusal to DCPT was encountered at depth of 27.6 m below the existing ground surface at BH 3-23. DCPT refusal was encountered at depth of 19.61 m within BH 2, advanced as a part of the previous geotechnical investigation completed on May 9, 2008.

Fill and Topsoil

Borehole BH3-23 was completed in the access path near the south of the parcel. The path was noted to be composed of a thin layer of granular crushed stone placed over top a 200 to 300 mm layer of topsoil.

Silty Clay

The topsoil was underlain by brown silty clay crust with traces of sand, to a depth of 4.5 to 5 m. In situ shear vane field testing carried out in the brown silty clay crust yielded undrained shear strength values ranging between 250 kPa to 77 kPa. These values are indicative of a hard to stiff consistency.

Grey silty clay was observed underlying the clay crust. In situ shear vane field testing carried out in the grey silty clay yielded undrained shear strength values ranging between 24 kPa to 68 kPa, which are indicative of a firm to stiff consistency.

Specific details of the soil profile at each test hole location are presented Appendix 1.

Bedrock

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded sandstone and dolostone of the March formation with an overburden drift thickness ranging from 15 to 25 m.

4.3 Groundwater

Groundwater level readings were measured on May 23, 2023, and are presented in Table 1 below, and on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	90.19	4.12	86.07	May 23, 2023
BH 2-23	90.13	2.57	87.56	May 23, 2023
BH 3-23	90.13	5.32	84.81	May 23, 2023

Note:
- The ground surface elevations are referenced to a geodetic datum.

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is anticipated to be at a depth ranging between 4.0 to 5.0 m throughout the subject site. However, groundwater levels are subject to seasonal fluctuations and could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed commercial development. It is expected that the proposed building can be constructed over conventional shallow footings placed on an undisturbed very stiff brown silty clay bearing surface.

Due to the presence of silty clay deposit, the site will be subjected to a permissible grade raise restriction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

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 areas should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. 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 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geo-composite drainage membrane connected to a perimeter drainage system.

5.3 Foundation Design

Conventional Shallow Footings

Footings placed on an undisturbed, very stiff brown silty clay bearing surface can be designed using a factored bearing resistance value at SLS of **150 kPa** and a bearing resistance value at ULS of **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing resistance values are provided on the assumption that the footings are placed on undisturbed soil bearing surfaces. 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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a soil bearing medium 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.

Permissible Grade Raise

Due to the presence of the silty clay deposit throughout the subject site, a permissible grade raise restriction is recommended for grading at the subject site. A permissible grade raise restriction of **2 m** is recommended for the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2020. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 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 materials, within the footprint of the proposed building, the native soil or approved fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 200 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

Loading Dock

The foundation wall at the loading dock, if the loading dock grade is depressed, will act as a retaining wall. Therefore, it should be designed to resist the lateral earth pressure of the fill material on the inside of the foundation wall. The wall should be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K_a \gamma H$ where:

K_a = 0.35 - active earth pressure coefficient if some movement can be tolerated and 0.5 if no movement can be tolerated.
γ = 22 kN/m^3, unit weight of the fill
H = height of the retaining wall, m

It should be noted that the fill on the inside of the wall should consist of free draining material such as OPSS Granular Type I or II.

The excavation side slope of the footing/foundation wall excavation should be tapered at 3H:1V or flatter on the pavement side of the loading dock and backfilled with OPSS Granular B Type I or II to minimize frost heaving. The fill material should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's standard Proctor dry density. The depressed area should be properly drained to minimize total and differential frost heaving.

The loading dock ramps may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

5.6 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of the car parking areas and local roadways.

Table 2 - Recommended Pavement Structure – Car Parking Only 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 Loading 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, bedrock 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.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

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 any proposed buildings. The system, where considered, should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 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 the storm sewer.

Foundation Backfill

For proposed buildings with below-grade space, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) connected to a drainage system is provided.

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. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

Exterior unheated footings, such as 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 the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient space will be available for the great part of the excavations 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 ground water level. The subsoil at this site appeared to be mainly a Type 2 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.

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. However, the bedding thickness should be increased to 300 mm and placed over a woven geotextile for areas where the services are bearing on firm grey silty clay. 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 300 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the moist (not wet) site-generated silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated silty clay fill will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in the longest direction should be segregated from re-use as trench backfill.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 300 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. A perched groundwater condition may be encountered within the silty sand deposit which may produce significant temporary groundwater infiltration levels. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will 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 package and issuance of the permit by the MECP.

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

6.6 Winter Construction

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

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

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

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 the subject site, whereas the resistivity is indicative of an aggressive corrosive environment.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

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

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Observation of the placement of the foundation insulation, if applicable.
- 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 and follow-up field density tests to determine the level of compaction achieved.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided herein 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 Prestige Construction, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 1, 2023
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PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Proposed Development
96 Bill Leathem Drive
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

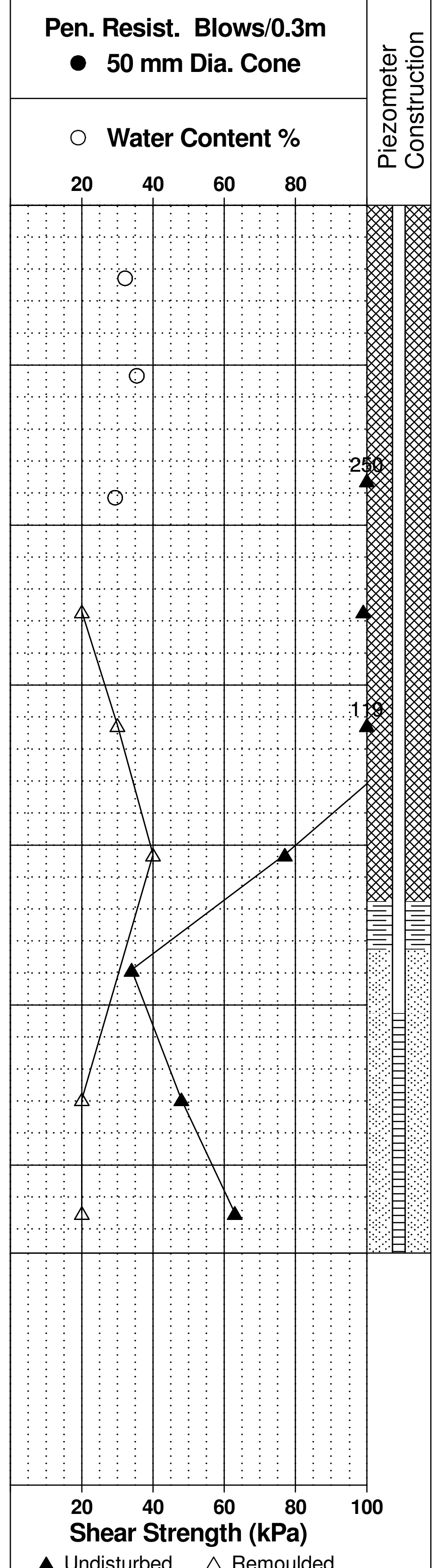
BORINGS BY CME 55 Power Auger

DATE May 17, 2023

FILE NO. PG6668

HOLE NO. BH 1-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	90.19
TOPSOIL with organics, brown silty clay, some gravel and cobbles 0.40		AU	1				
Hard to very stiff, brown **SILTY CLAY** trace sand		SS	2	42	15	1	89.19
		SS	3	75	P	2	88.19
						3	87.19
4.42						4	86.19
Firm to stiff, grey **SILTY CLAY**						5	85.19
6.55						6	84.19
End of borehole (GWL @ 4.12 m depth on May 23, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Proposed Development
96 Bill Leathem Drive
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 17, 2023

FILE NO. PG6668

HOLE NO. BH 2-23

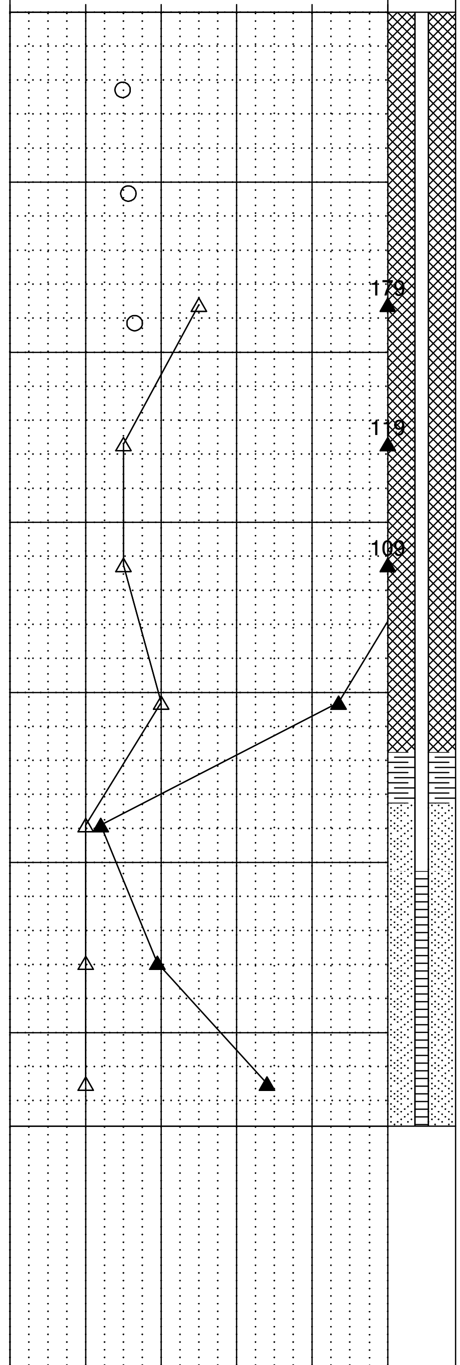
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	90.13
TOPSOIL with organics, brown silty clay, some gravel and cobbles (0.41)		AU	1				
		SS	2	100	10	1	89.13
Very stiff, brown **SILTY CLAY** trace sand		SS	3	63	P	2	88.13
						3	87.13
(4.42)						4	86.13
Soft to firm to stiff, grey **SILTY CLAY**						5	85.13
(6.55)						6	84.13
End of borehole (GWL @ 2.57m depth on May 23, 2023)							

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

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Proposed Development
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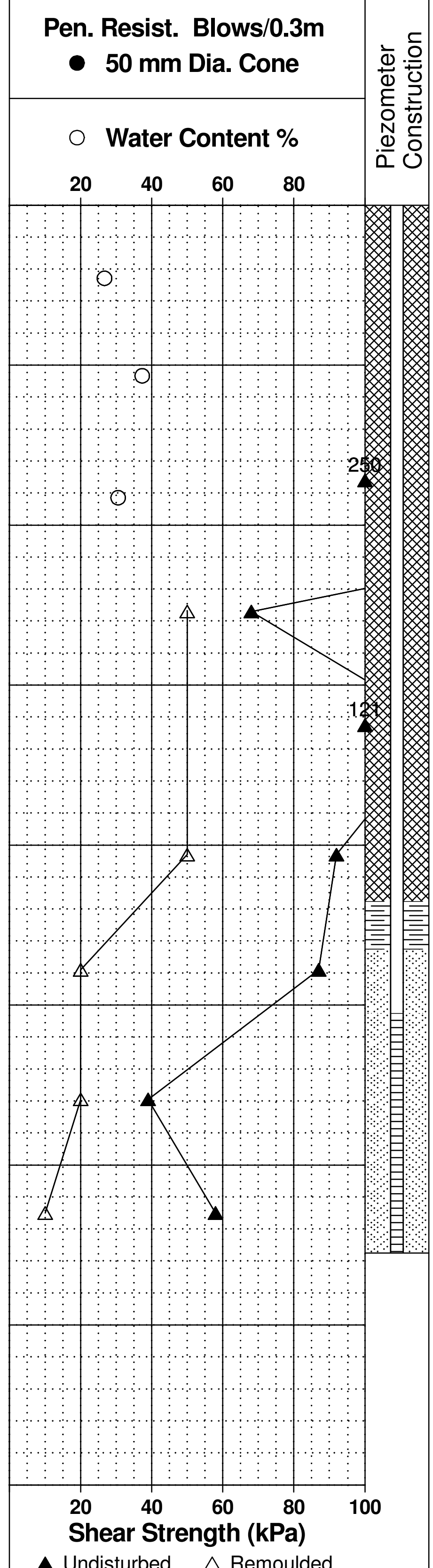
FILE NO. PG6668

HOLE NO. BH 3-23

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL Crushed stone, some sand and gravel, trace organics	0.05				
TOPSOIL with brown silty clay, trace organics and crushed stone	0.41	AU	1		
Hard to stiff brown **SILTY CLAY** trace sand		SS	2	83	8
		SS	3	42	P
	5.18				
Firm to stiff, grey **SILTY CLAY**	6.55				
Dynamic Cone Penetration Test commenced at 6.55m depth Cone pushed to 14.53m					

DEPTH (m)	ELEV. (m)
0	90.13
1	89.13
2	88.13
3	87.13
4	86.13
5	85.13
6	84.13
7	83.13
8	82.13



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SOIL PROFILE AND TEST DATA

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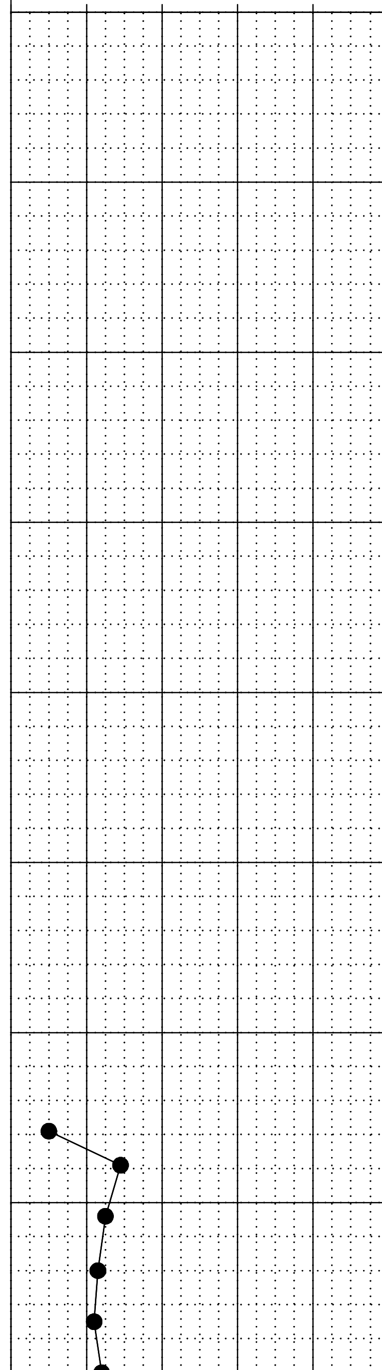
FILE NO. PG6668

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
Ground Surface									
						8	82.13		
						9	81.13		
						10	80.13		
						11	79.13		
						12	78.13		
						13	77.13		
						14	76.13		
						15	75.13		
						16	74.13		



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Proposed Development
96 Bill Leathem Drive
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

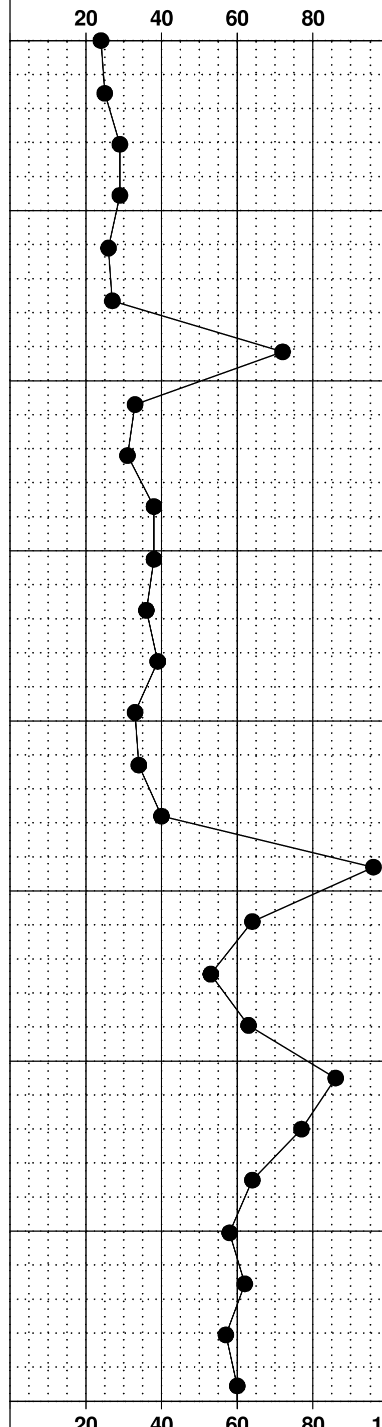
REMARKS

BORINGS BY CME 55 Power Auger **DATE** May 17, 2023

FILE NO. PG6668

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
Ground Surface						16	74.13		
						17	73.13		
						18	72.13		
						19	71.13		
						20	70.13		
						21	69.13		
						22	68.13		
						23	67.13		
						24	66.13		



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

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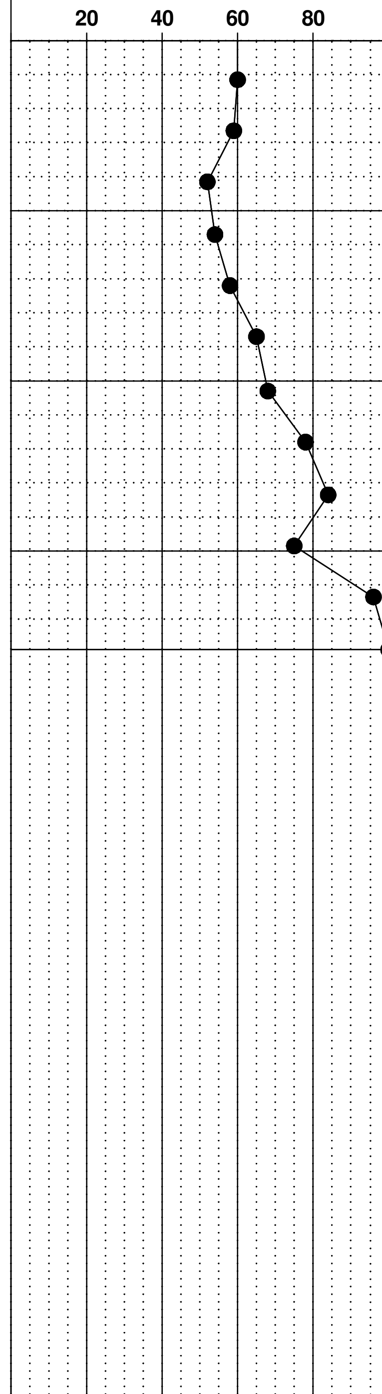
DATE May 17, 2023

FILE NO. PG6668

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
Ground Surface						24	66.13		
						25	65.13		
						26	64.13		
						27	63.13		
27.58									
Practical DCPT refusal to at 27.58m depth. (GWL @ 5.32m depth on May 23, 2023)									



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
South Merivale Business Park, Bill Leathem Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

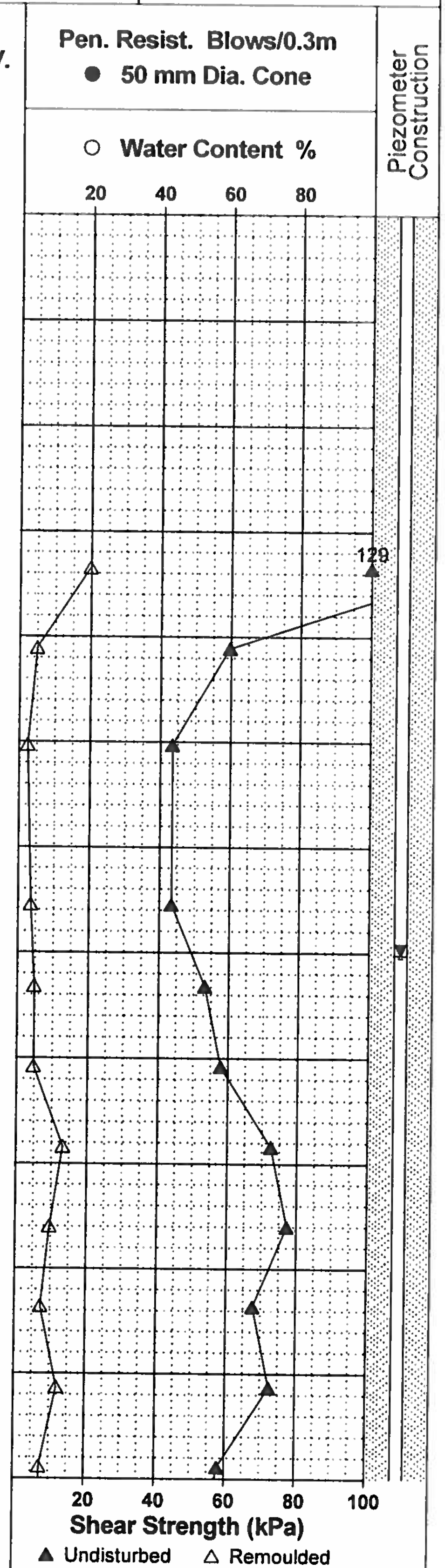
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 9 May 08

FILE NO. PG1658

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.98
TOPSOIL 0.13		AU	1				
		SS	2	96	17	1	88.98
		SS	3	83	11	2	87.98
		SS	4	100	8	3	86.98
Very stiff to stiff, brown SILTY CLAY, some sand seams						4	85.98
- stiff and grey by 4.3m depth						5	84.98
		TW	5	100		6	83.98
						7	82.98
						8	81.98
						9	80.98
						10	79.98
						11	78.98
						12	77.98



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
South Merivale Business Park, Bill Leathem Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

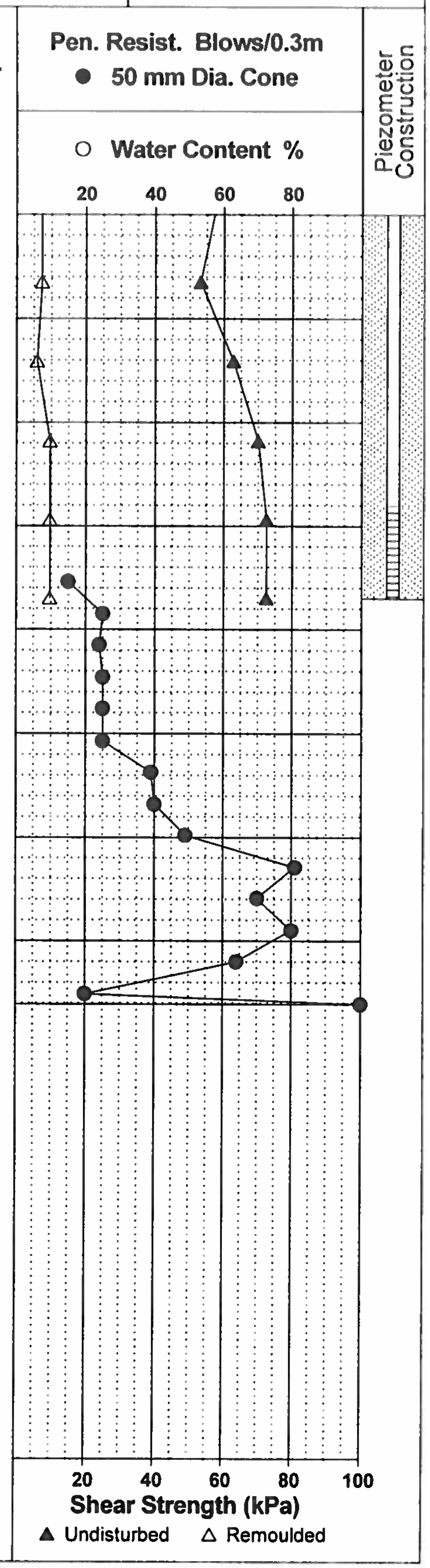
BORINGS BY CME 55 Power Auger

DATE 9 May 08

FILE NO. **PG1658**

HOLE NO. **BH 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						12	77.98
						13	76.98
Stiff, grey **SILTY CLAY**							
						14	75.98
						15	74.98
15.70							
Dynamic Cone Penetration Commenced @ 15.70m depth							
						16	73.98
						17	72.98
Inferred **SILTY CLAY**							
						18	71.98
						19	70.98
19.61							
End of Borehole							
Practical DCPT refusal @ 19.61m depth							
(GWL @ 7.02m-June 5/08)							



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

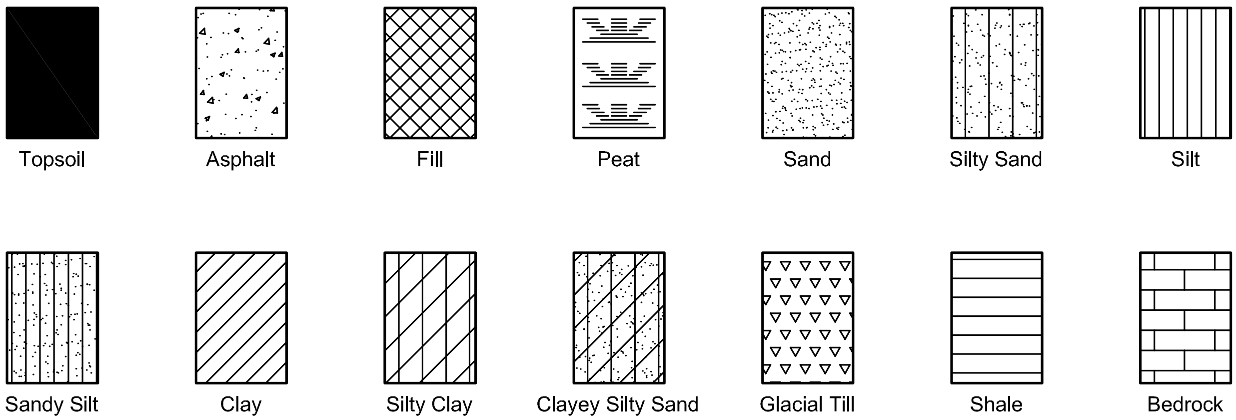
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

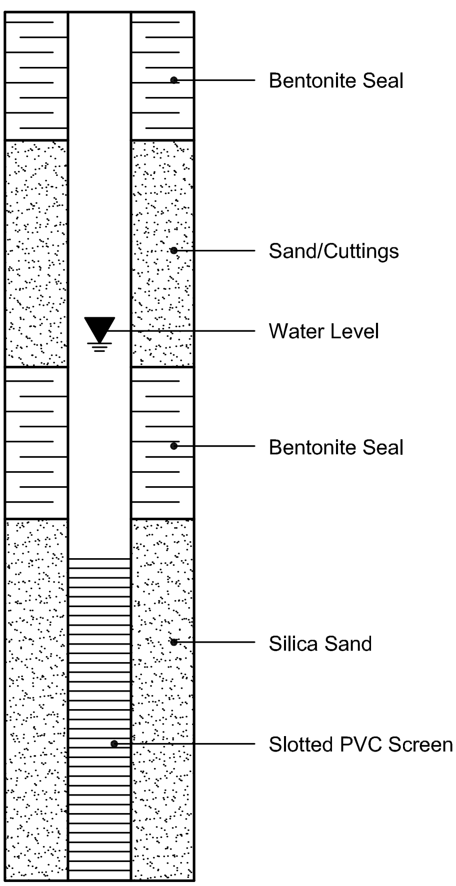
SYMBOLS AND TERMS (continued)

STRATA PLOT

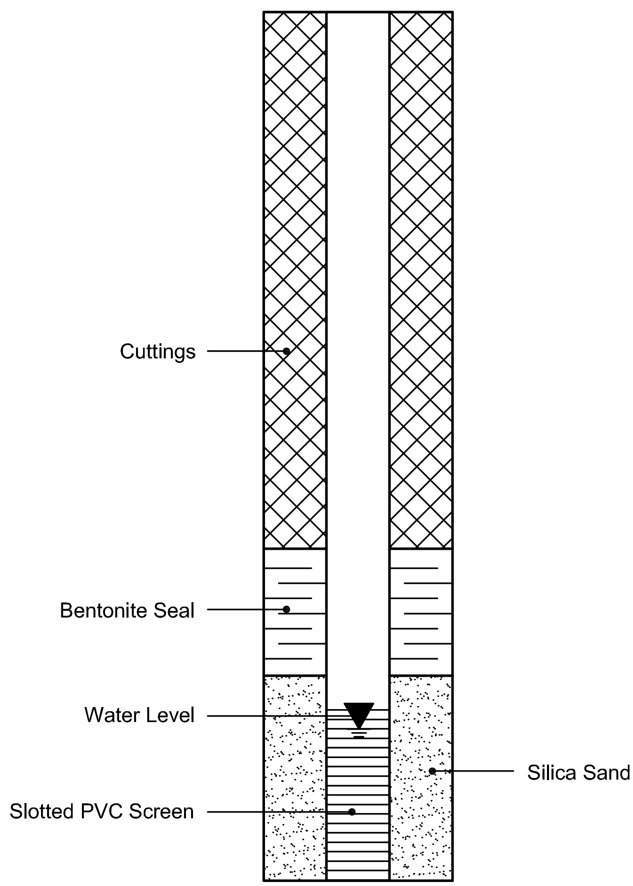


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2320398

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 57533

Report Date: 25-May-2023

Order Date: 18-May-2023

Project Description: PG6668

	Client ID:	BH3-23-SS2	-	-	-
	Sample Date:	17-May-23 09:00	-	-	-
	Sample ID:	2320398-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.81	-	-	-
Resistivity	0.1 Ohm.m	17.3	-	-	-
Anions					
Chloride	10 ug/g dry	221	-	-	-
Sulphate	10 ug/g dry	136	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6668-1 - TEST HOLE LOCATION PLAN

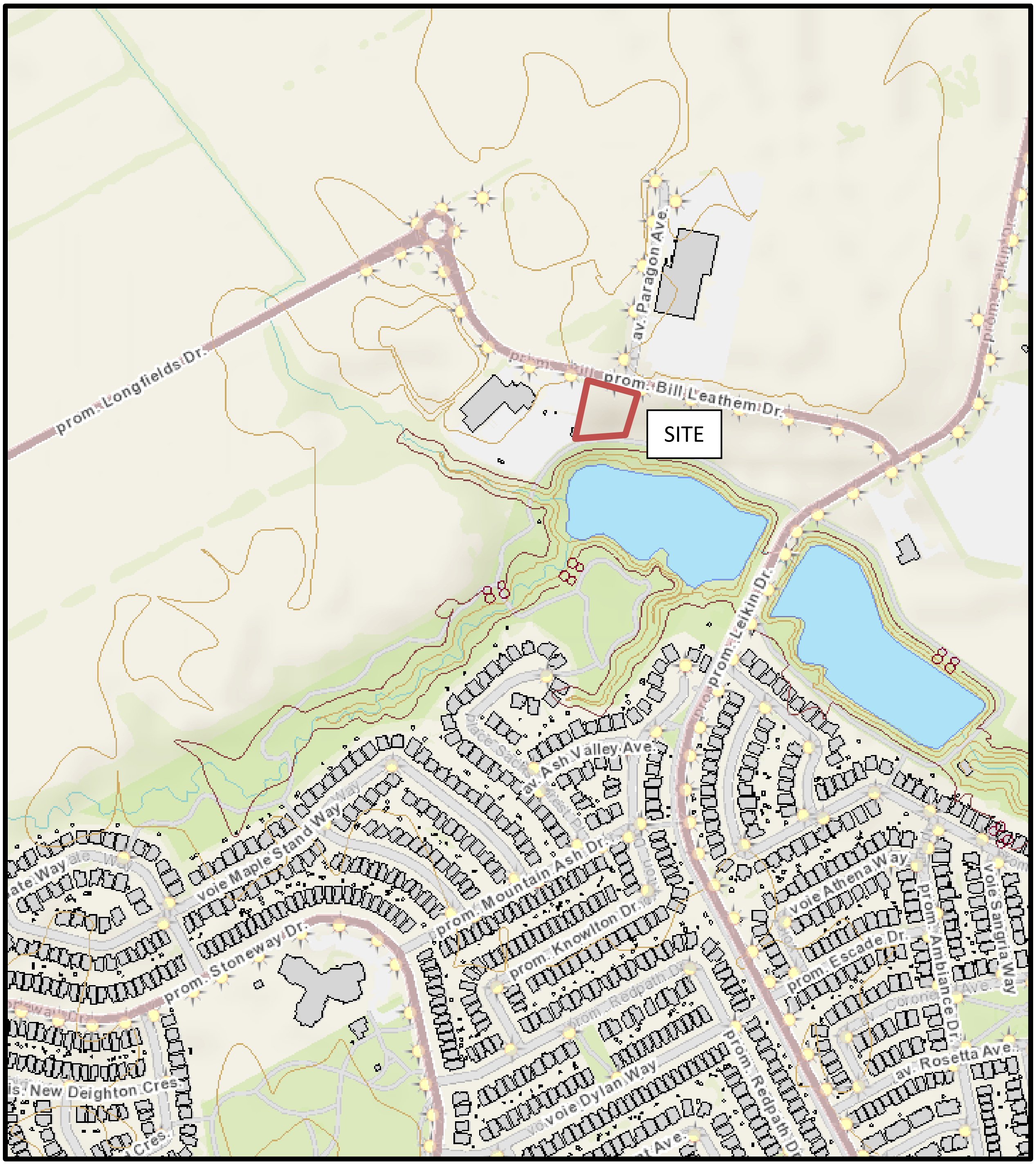
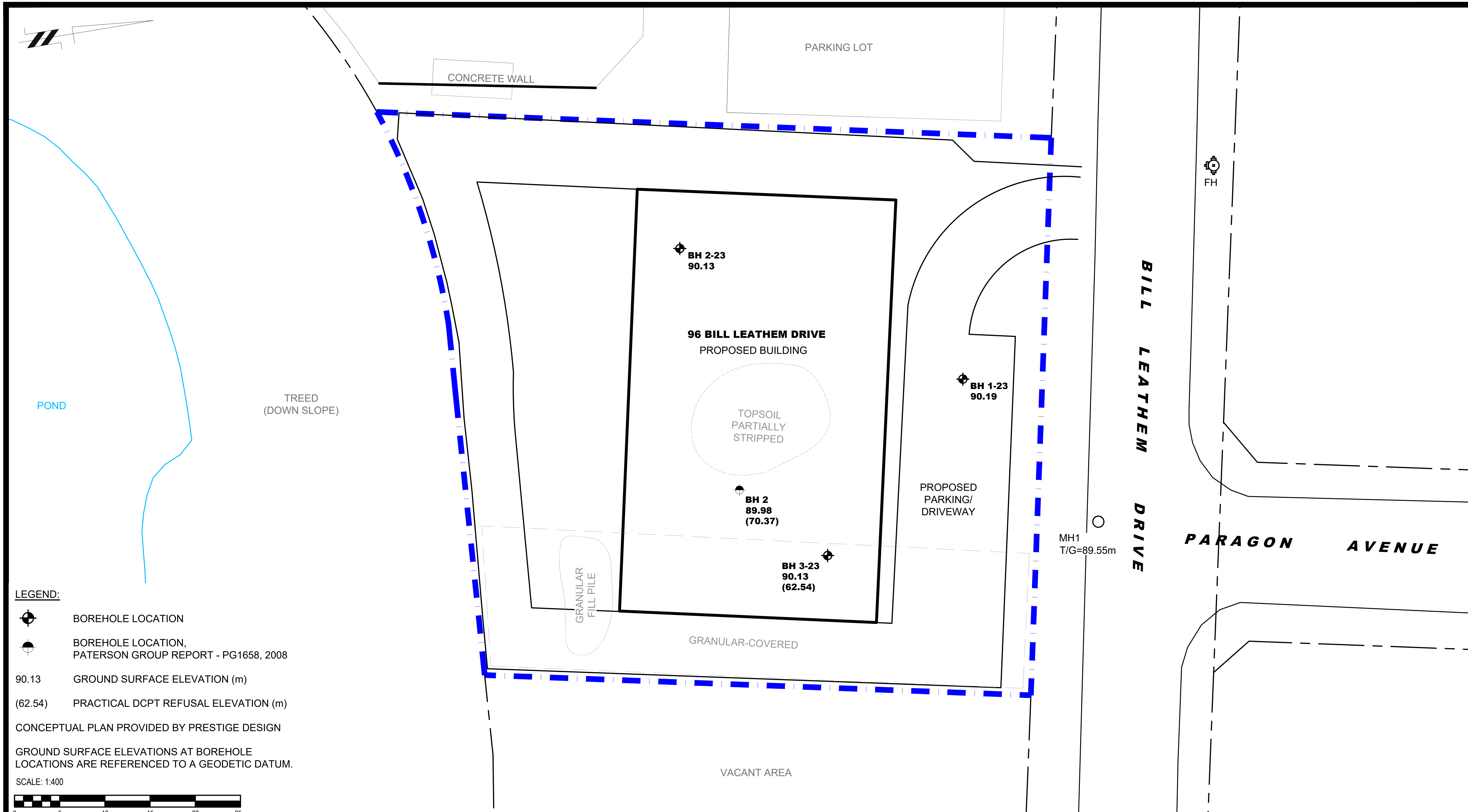


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

BOREHOLE LOCATION

BOREHOLE LOCATION,
PATERSON GROUP REPORT - PG1658, 2008

90.13 GROUND SURFACE ELEVATION (m)

(62.54) PRACTICAL DCPT REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY PRESTIGE DESIGN

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

PRESTIGE DESIGN & CONSTRUCTION LTD.
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
96 BILL LEATHEM DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 05/2023
Drawn by: YA	Report No.: PG6668-1
Checked by: PT	Dwg. No.: PG6668-1
Approved by: JV	Revision No.:

11x17

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