

**Geotechnical
Engineering**

**Environmental
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Engineering**

Materials Testing

Building Science

Archaeological Services

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Geotechnical Investigation

Proposed Multi-Storey Hotel
Kanata West Business Park - Block 23
Campeau Drive at Palladium Drive
Ottawa, Ontario

Prepared For

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Report PG4378-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Boreal Hospitality Group to conduct a geotechnical investigation for the proposed multi-storey hotel to be located at Block 23 of the Kanata West Business Park located at the intersection of Campeau Drive and Palladium Drive in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current 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

Details of the proposed project were not available at the time of writing this report. However, it is understood that the proposed project is to consist of a multi-storey hotel building with possibly one underground parking level. Associated at grade parking areas, access lanes, and landscaped areas are further expected.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the most recent investigation was carried out on December 13, 2017. At that time, four (4) boreholes were advanced to a maximum depth of 6.7 m below existing grade. The findings of an additional borehole completed within Block 23 of the Kanata West business park development has also been included. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG4378-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down 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 from the geotechnical division. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing 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 provided 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals in cohesive soils.

The subsurface conditions observed in the boreholes 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.

Groundwater

Flexible standpipes were installed in all boreholes to permit the monitoring of water levels subsequent to the completion of the sampling program.

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 borehole locations were selected and determined in the field by Paterson personnel to provide general coverage of the subject site. Ground surface elevations at borehole locations are referenced to a temporary benchmark (TBM), consisting of the top of grate of catch basin located on the south side of the subject site. A geodetic elevation of 103.39 m was provided for the TBM. The TBM location and ground surface elevation at each borehole location are presented on Drawing PG4378-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. 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 is currently undeveloped and was snow covered at the time of the investigation. The ground surface across the subject site is relatively flat with a gentle downslope towards the southeast corner. The ground surface was observed to be slightly lower than the adjacent roadways. It should be noted that an asphalt covered access road runs south to north across the central portion of the subject site. It should also be noted that a former drainage ravine, running east-west along the south property boundary, was in-filled as part of previous on-site work associated with the overall development.

4.2 Subsurface Profile

Generally, the soil profile encountered at the test hole locations consists a topsoil layer or crushed stone fill at ground surface overlying a very stiff to stiff brown silty clay crust. A stiff grey silty clay was encountered below the above noted layers followed by loose to very loose, grey clayey silt to silt and a glacial till deposit. The glacial till consists of grey silt with some sand and trace clay, gravel and cobbles. Practical refusal to augering was encountered at BH 1-17 and BH 2-17 at a depth of 6.4 and 6.1 m, respectively. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and shale of the Verulam formation. The overburden drift thickness is estimated to be between 5 and 25 m depth.

4.3 Groundwater

The groundwater levels (GWL) were recorded at the piezometer within the borehole locations on December 20, 2017. The measured groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole. Groundwater levels can also be estimated based on experience within the local area, the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated between 3.5 to 4.5 m depth. It should be noted that the groundwater levels are subject to seasonal fluctuations. Therefore, groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey hotel. It is anticipated that the proposed building can be founded by conventional shallow footings placed on undisturbed, stiff silty clay bearing surface. However, if the provided soil bearing resistance values are not sufficient for the design building loads, a deep foundation, such as end bearing piles, may need to be considered.

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 organic materials, should be stripped from under any buildings and other settlement sensitive structures.

Existing fill, free of deleterious materials, should be reviewed by the geotechnical consultant at the time of construction to confirm if the existing fill can remain in place or be re-used as select subgrade fill. For areas where the existing fill is to remain in place, it is recommended that the existing fill, free of deleterious materials, should be proof-rolled by a vibratory roller making several passes and any poor performing areas should be removed.

Fill Placement

Fill used for grading beneath the proposed building and settlement sensitive structures unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the round 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 unless a composite drainage blanket connected to a perimeter drainage system is provided.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular B Type I, Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Shallow Foundation

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, stiff silty clay bearing surface 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 resistance factor of 0.5 was applied to the above noted 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.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay or engineered fill 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 Recommendations

Due to the underlying silty clay deposit, a **permissible grade raise of 2 m** is recommended for grading within the proposed development.

Piled Foundation

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values for concrete-filled steel pipe piles are given in Table 1. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a dynamic monitoring program. For this project, the dynamic monitoring of four (4) piles would be recommended. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

A working pad is recommended during the piling driving operation. It is expected that a 300 mm thick layer of 100 mm minus or Granular B Type II would provide an acceptable surface.

Table 1 - Pile Foundation Design Data					
Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the shallow foundations considered at this site. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials, within the footprint of the proposed building, the native soil 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 a 19 mm clear crushed stone for the basement slab. 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.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3 . The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3 , where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5

γ = unit weight of fill of the applicable retained soil (kN/m^3)

H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2 / g$ where:

$$a_c = (1.45 - a_{max}/g) a_{max}$$

γ = unit weight of fill of the applicable retained soil (kN/m³)

H = height of the wall (m)

g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 K_o \gamma H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\} / P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Car only parking and heavy truck parking areas, as well as, access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
	SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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 98% 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 impervious nature 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

A perimeter foundation drainage system is required for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at underside of 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.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. It is expected that the greater part of the site excavated materials can be re-used provided a composite drainage blanket, such as Delta Drain 6000 or equivalent is in place. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, can also be used for this purpose.

Underfloor Drainage

It is anticipated that underfloor drainage may be required to control water infiltration. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 of 1.5 m of soil cover alone, or a 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 excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is 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). Temporary shoring may be required where sufficient space is unavailable.

The subsoil at this site is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

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

At least 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. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials 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) 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

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.

A temporary Ministry of the Environment and Climate Change (MOECC) 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 MOECC.

For typical ground or surface water volumes, being pumped during the construction phase, 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 MOECC review of the PTTW application.

The groundwater flow rate into the excavation through the overburden should be moderate to high for expected founding levels of the proposed building.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 10,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

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

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should 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 materials 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 grading plan, drawings and specifications are completed.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

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 Boreal Hospitality Group, or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Faisal I. Abou-Seido, P.Eng.



David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Boreal Hospitality Group (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Block 23 - Campeau Drive at Palladium Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the south side of subject site, along Campeau Drive. Elevation = 103.39m.

REMARKS

BORINGS BY CME 55 Power Auger

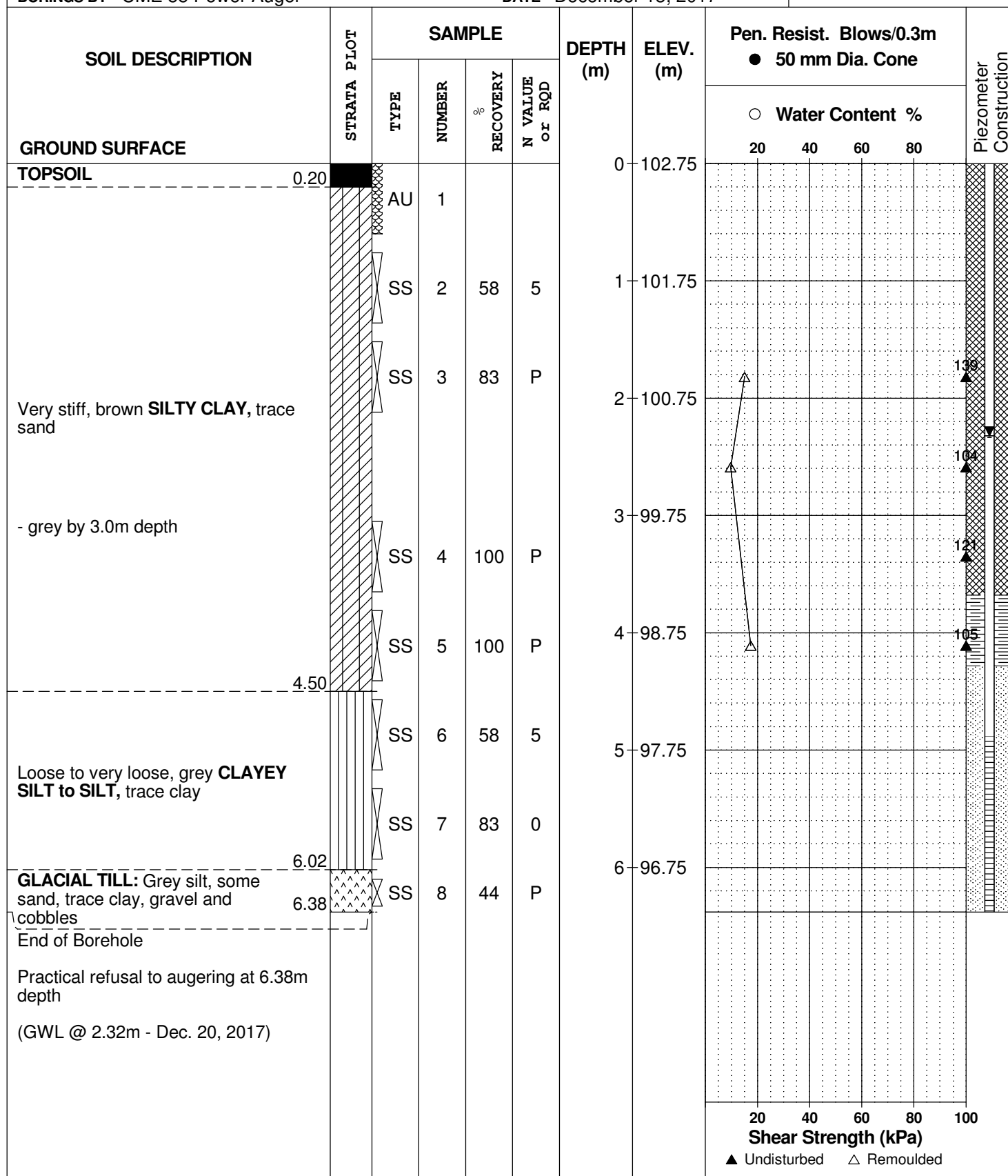
DATE December 13, 2017

FILE NO.

PG4378

HOLE NO.

BH 1-17



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Block 23 - Campeau Drive at Palladium Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the south side of subject site, along Campeau Drive. Elevation = 103.39m.

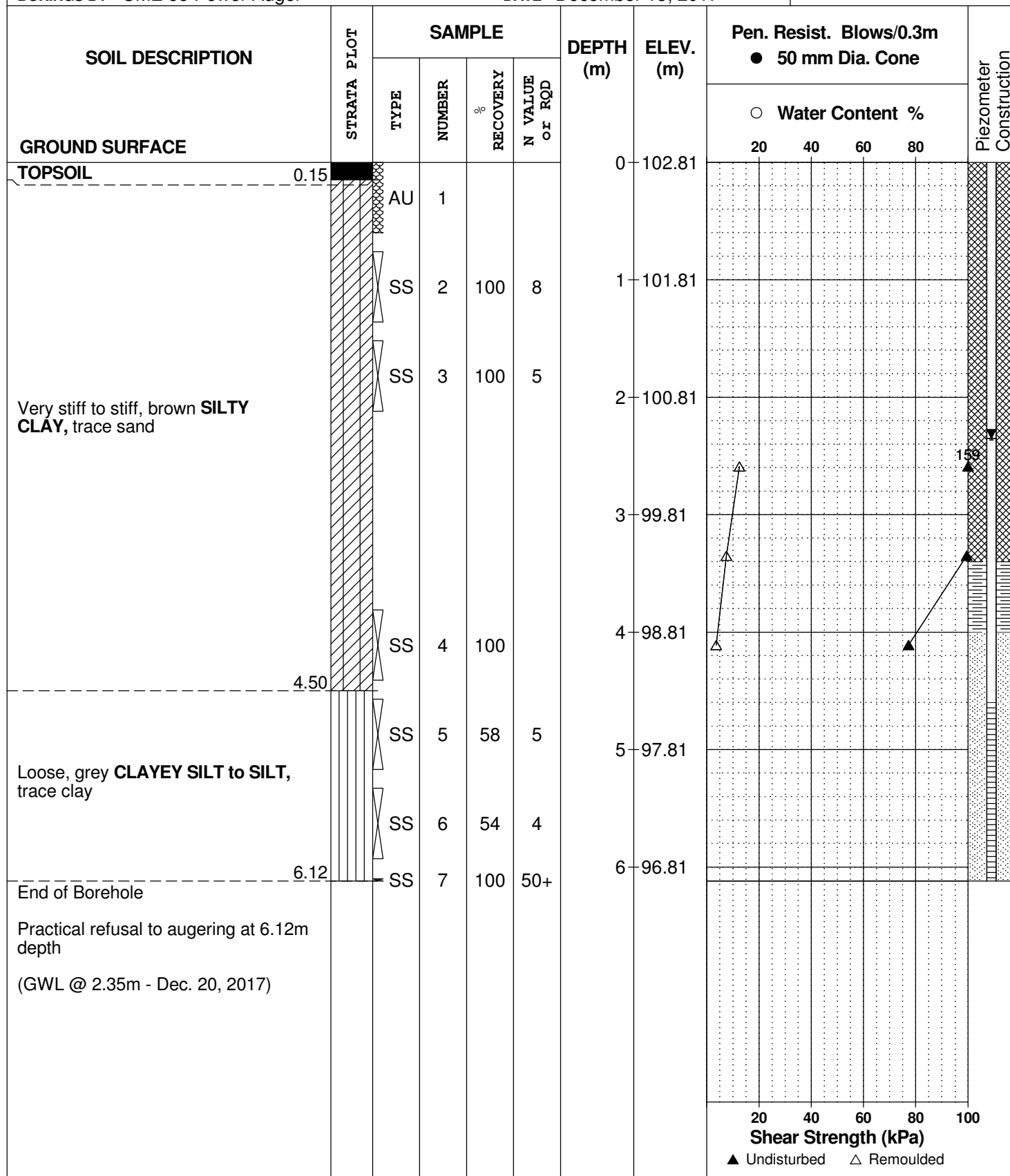
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 13, 2017

FILE NO.
PG4378

HOLE NO.
BH 2-17



DATUM TBM - Top of grate of catch basin located on the south side of subject site, along Campeau Drive. Elevation = 103.39m.

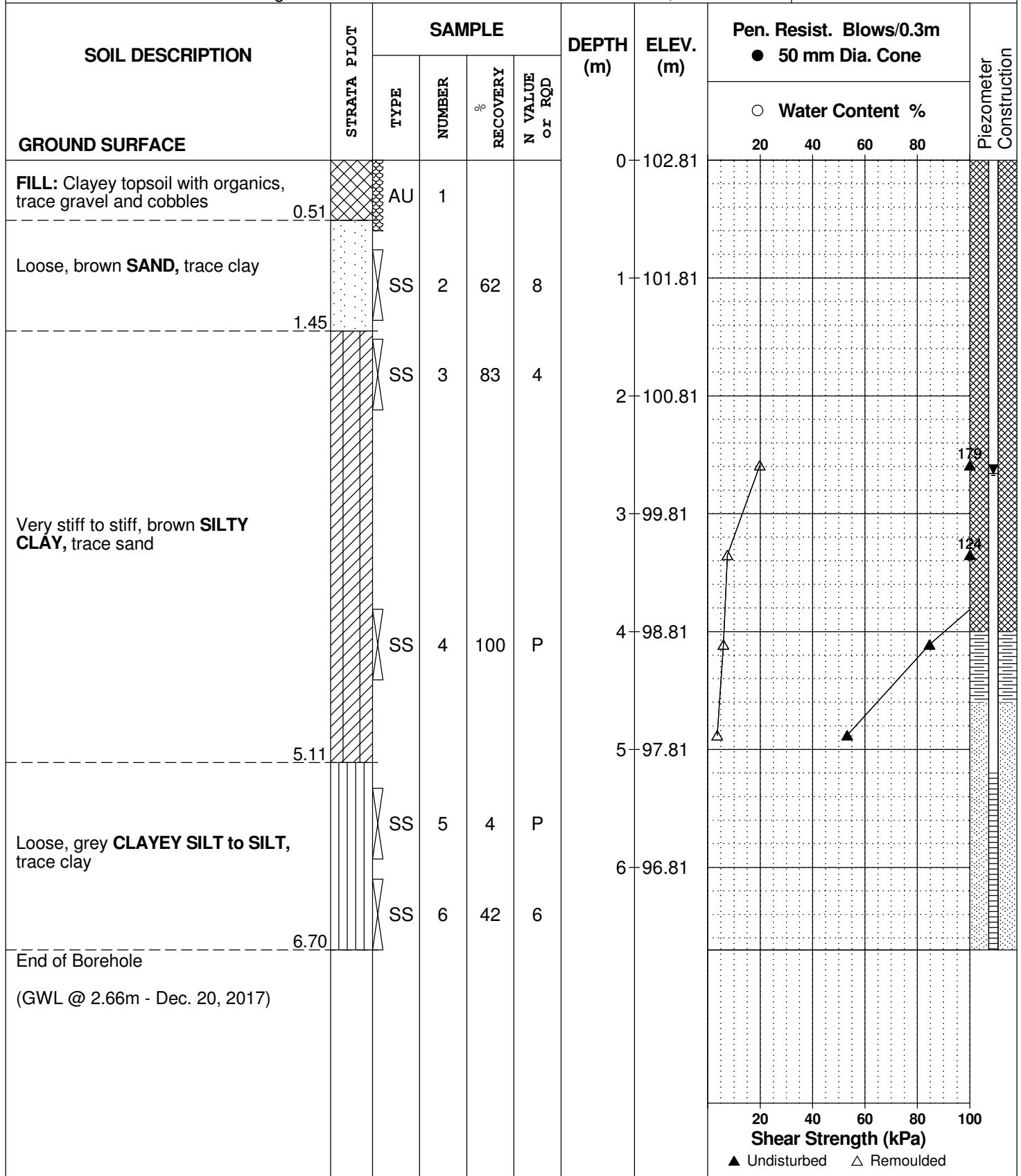
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 13, 2017

FILE NO. PG4378

HOLE NO. BH 3-17



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Block 23 - Campeau Drive at Palladium Drive
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the south side of subject site, along Campeau Drive. Elevation = 103.39m.

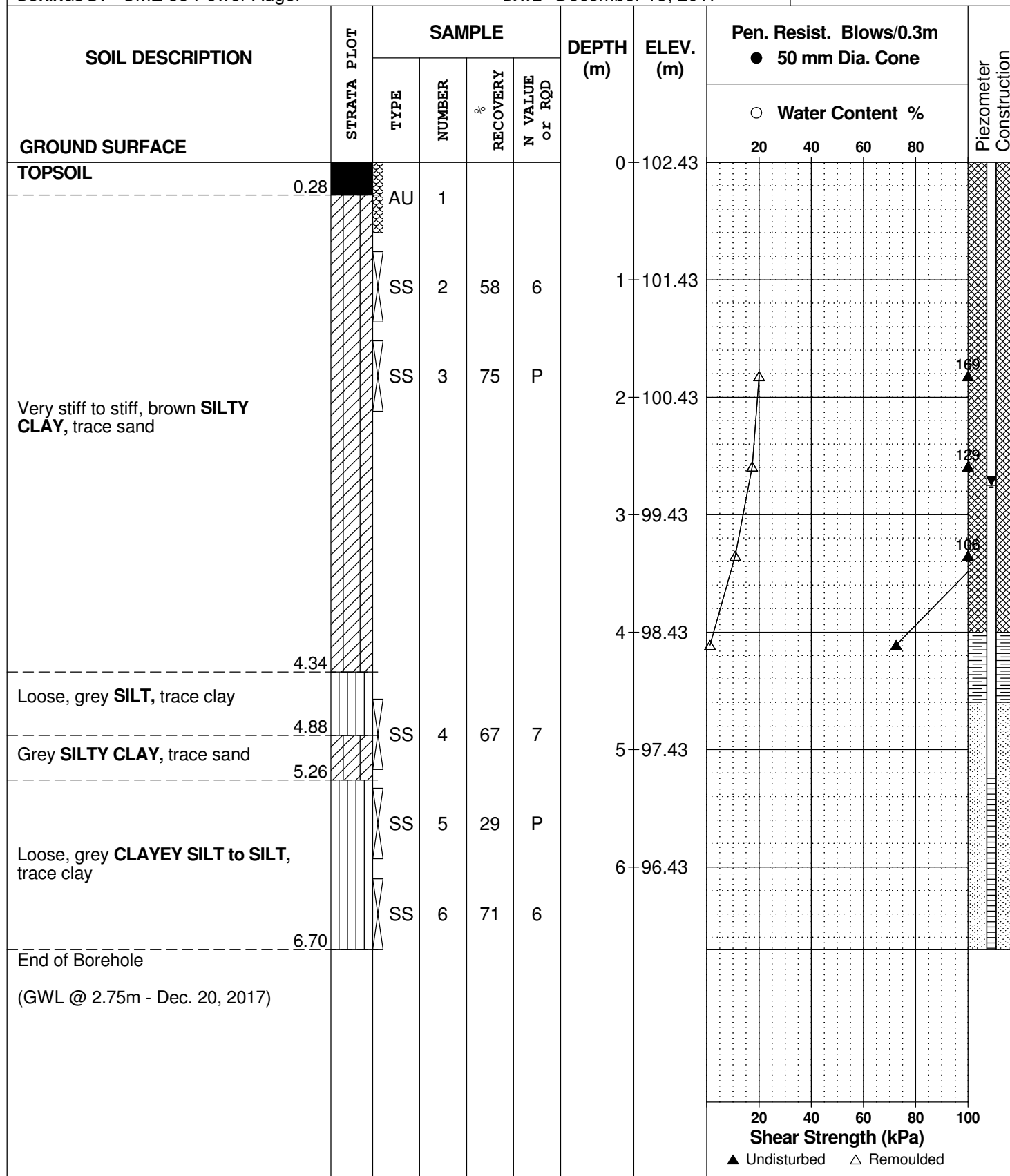
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BORINGS BY CME 55 Power Auger

DATE December 13, 2017

FILE NO.
PG4378

HOLE NO.
BH 4-17



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

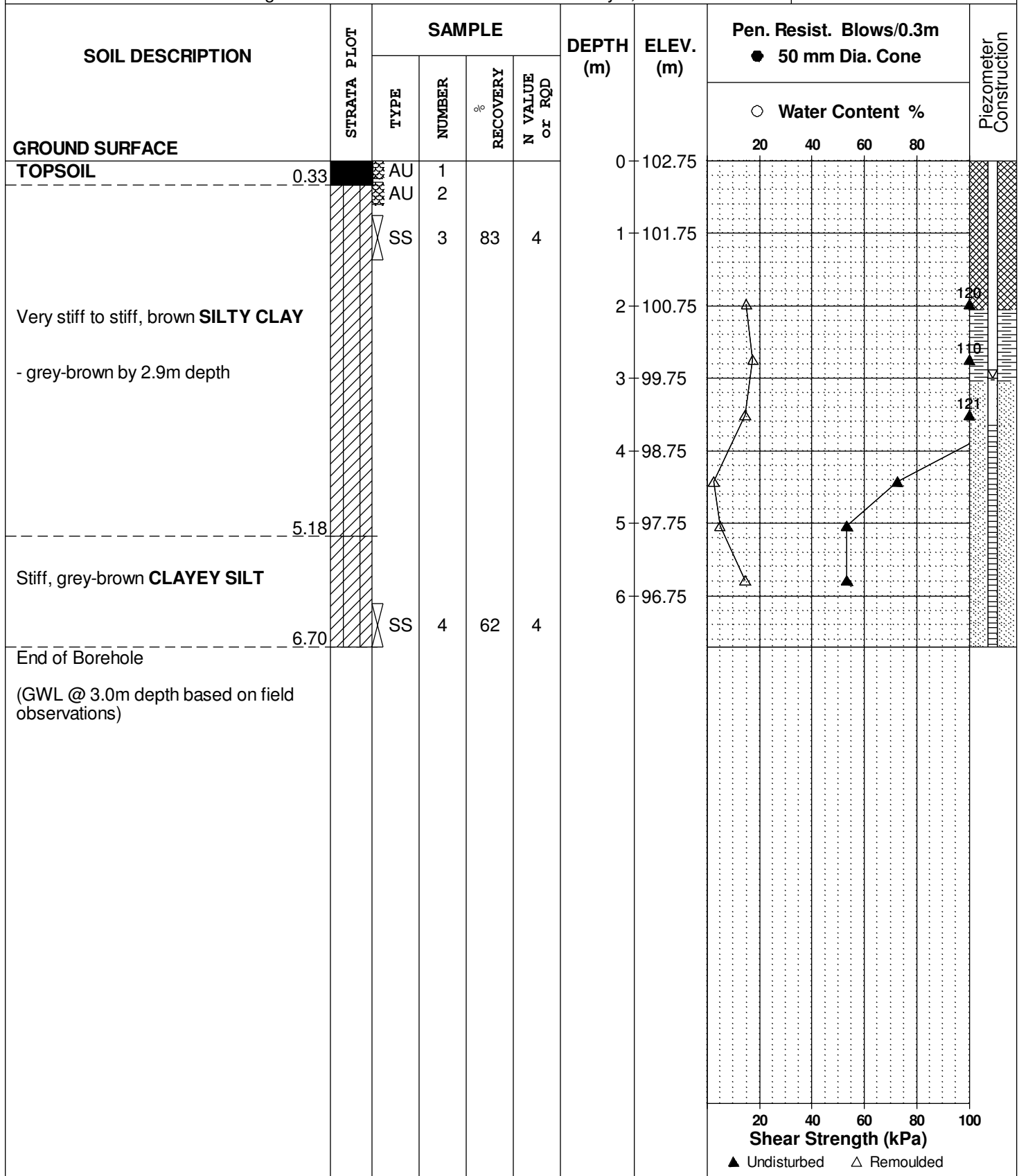
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 9, 2014

FILE NO. PG3115

HOLE NO. BH23



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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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		Overconsolidation 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.
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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: 23193

Report Date: 30-Nov-2017

Order Date: 27-Nov-2017

Project Description: PG4278

Client ID:	BH4-17 SS3	-	-	-
Sample Date:	24-Nov-17	-	-	-
Sample ID:	1748095-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	70.7	-	-	-
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General Inorganics

pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	30.3	-	-	-

Anions

Chloride	5 ug/g dry	13	-	-	-
Sulphate	5 ug/g dry	115	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4378-1 - TEST HOLE LOCATION PLAN

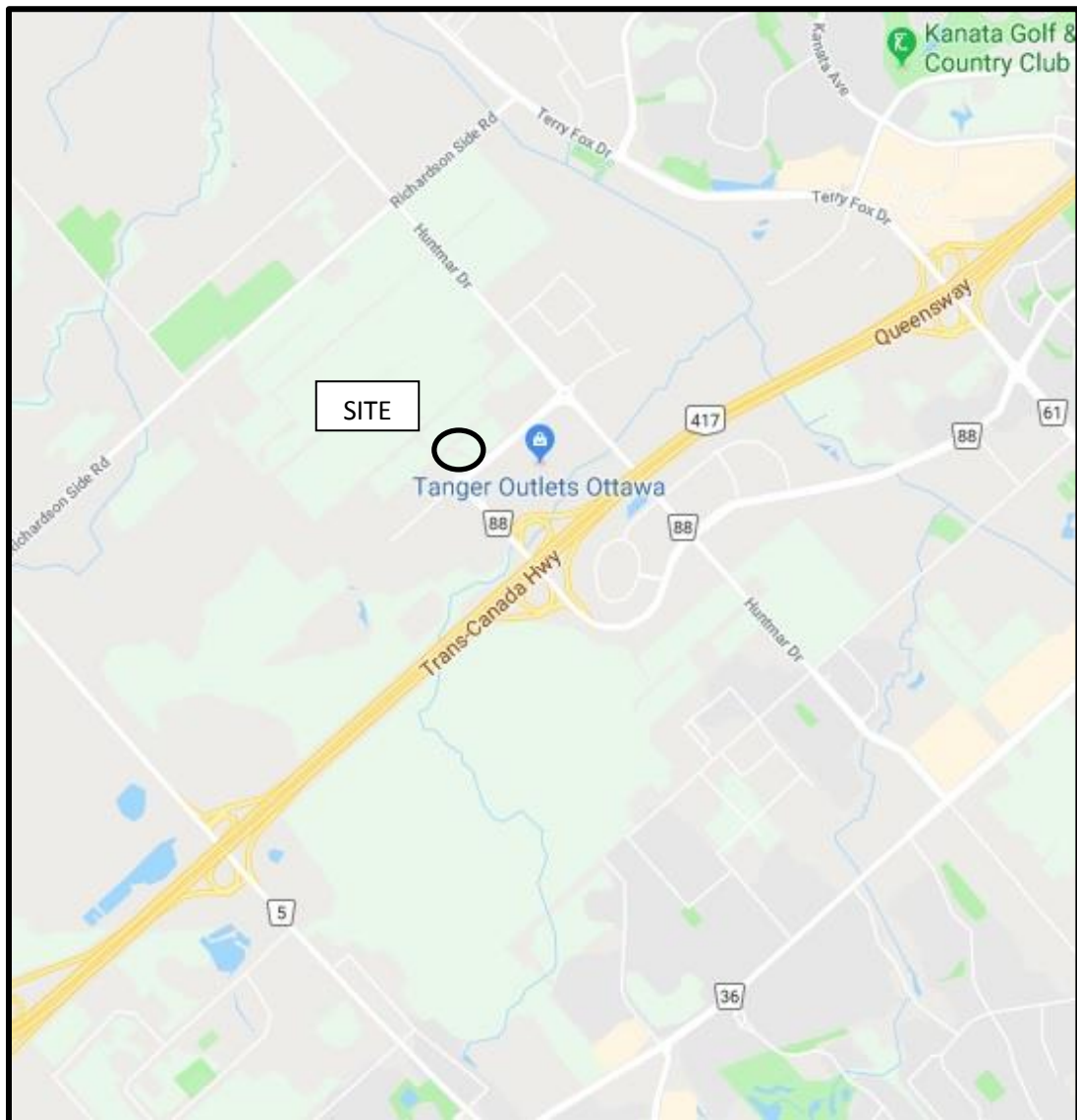
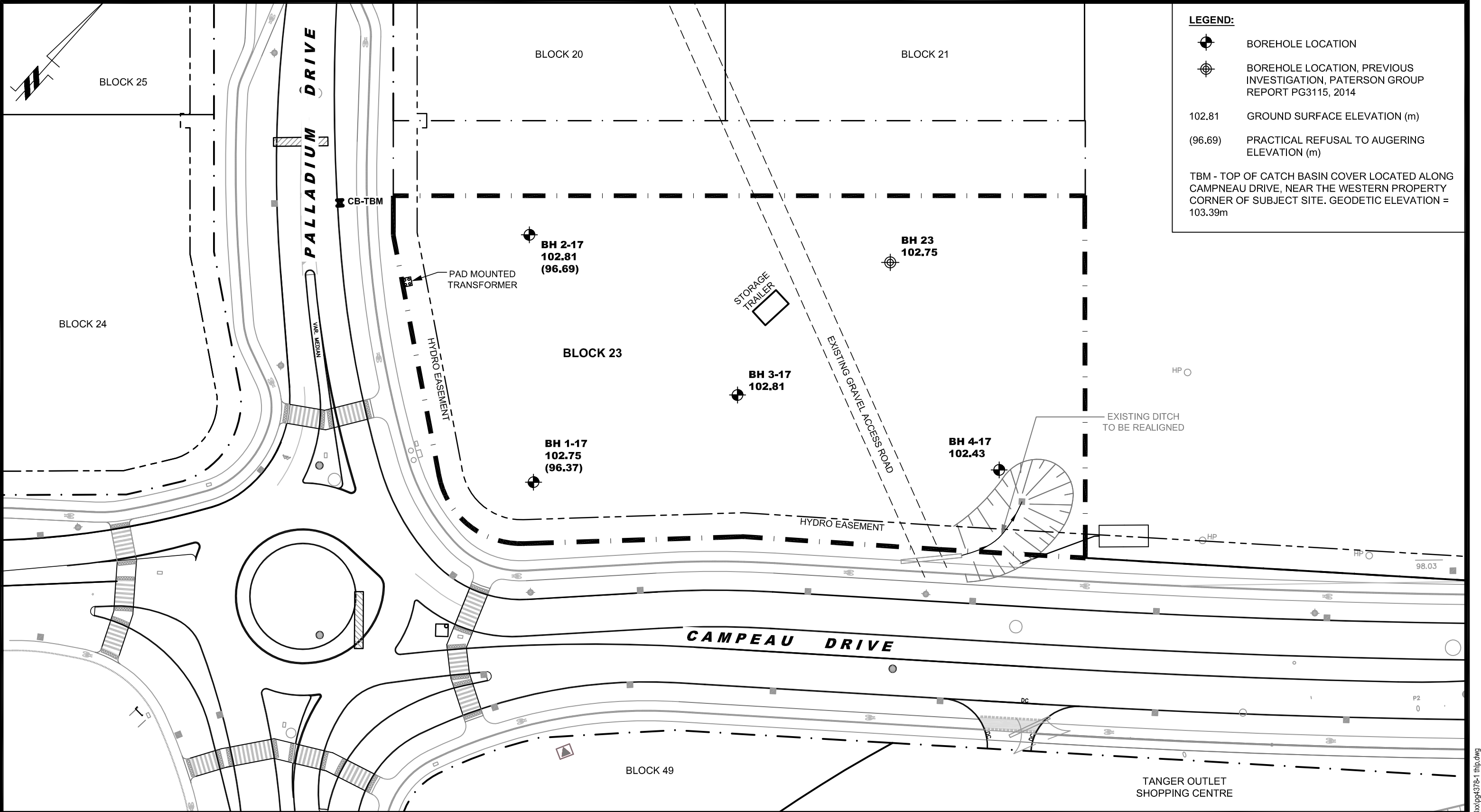


FIGURE 1
KEY PLAN



LEGEND:

BOREHOLE LOCATION

BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG3115, 2014

102.81 GROUND SURFACE ELEVATION (m)

(96.69) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP OF CATCH BASIN COVER LOCATED ALONG CAMPEAU DRIVE, NEAR THE WESTERN PROPERTY CORNER OF SUBJECT SITE. GEODETIC ELEVATION = 103.39m

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0			
NO.	REVISIONS	DATE	INITIAL

BOREAL HOSPITALITY GROUP
GEOTECHNICAL INVESTIGATION
BLOCK 23 - CAMPEAU DRIVE AT PALLADIUM DRIVE

OTTAWA,
Title:

ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:750	Date:	01/2018
Drawn by:	RCG	Report No.:	PE4171-1
Checked by:	FA	Dwg. No.:	PG4378-1
Approved by:	DJG	Revision No.:	0

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