

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Geotechnical Investigation
Proposed Residential Building
667 Bank Street
Ottawa, Ontario

Prepared For

Milito Investments

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Report PG3825-1 Revision 2

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Milito Investments to conduct a geotechnical investigation for the proposed residential building to be located at 667 Bank Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the investigation were to:

- ❑ Determine the subsurface soil and groundwater conditions 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Project

It is understood that the proposed development consists of a five (5) storey residential building with semi basement. The proposed building is understood to cover the majority of the subject site. Associated landscaped areas and access lanes are also anticipated.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on May 18, 2016. The investigation consisted of four (4) boreholes, advanced to a maximum depth of 14.4 m below ground surface. The test hole locations were selected in a manner as to provide general coverage of the proposed development. The test hole locations are shown on Drawing PG3825-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down with a truck-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected location, 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 soil samples were transported to our laboratory. The depths at which the split spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

In conjunction with the recovery of the split spoon samples, the Standard Penetration Test (SPT) was conducted. 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 completed at regular intervals in cohesive soils.

Overburden thickness was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at the BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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.

The subsurface conditions observed in the borehole was recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Flexible polyethylene standpipes were installed in BH 1 and BH 2 and a 51 mm monitoring well was installed in BH 3 and BH 4 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at the borehole locations were referenced to a temporary benchmark (TBM) consisting of the top of spindle of the fire hydrant located at the northeast corner of Clemow Avenue and Bank Street. An assumed elevation of 100.00 m was provided for the TBM. The location and ground surface elevations at the boreholes are presented on Drawing PG3825-1 - 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. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

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 samples were analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in section 6.7 and shown in appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is currently gravel/asphalt covered. The ground surface is relatively flat and at grade with Bank Street.

4.2 Subsurface Profile

Generally, the subsoil conditions at the test hole locations consist of fill material, consisting of silty sand with crushed stone. Native silty sand followed by a silty clay deposit was encountered below the fill in all the test holes. Practical refusal to DCPT was encountered at BH 3 at a depth of 14.4 m below ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the borehole location.

Based on geological mapping, the bedrock consists of shale from the Billings Formation. The overburden drift thickness is approximately 10 to 25 m.

4.3 Groundwater

Groundwater level readings were taken at the borehole locations on May 27, 2016. Our groundwater measurements are presented in the Soil Profile and Test Data sheets. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole due to the seasonal changes (spring thaw), which can lead to water perching inside the boreholes which results in a higher water levels than noted during the investigation. The long-term groundwater level can also be estimated based on moisture levels and colour of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater level is expected at a 4 to 5.5 m depth.

Groundwater is subject to seasonal fluctuations and therefore, groundwater 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 development. It is expected that the proposed building can be founded by conventional style shallow foundations over stiff to very stiff silty clay deposit. However, due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions.

Our permissible grade raise recommendations are discussed in Subsection 5.3. If higher than permissible grade raises are required, preloading with or without surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Engineering fill may be required where existing fill is located below proposed footing level. The approved engineered fill should be proof-rolled using suitable compaction equipment under dry conditions and reviewed by Paterson personnel.

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

5.2 Site Preparation

Stripping Depth

Topsoil and fill, containing significant amounts of organic or deleterious materials, should be removed from within the perimeter of the proposed building and other sensitive structures.

Fill Placement

Fill used for grading purposes beneath the proposed building, 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 in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These

materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless deemed acceptable by the geotechnical consultant and placed in conjunction with a composite drainage membrane.

5.3 Foundation Design

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using 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**.

Footings placed on an approved engineered fill can be designed using bearing resistance value at SLS of **150 kPa** and factored bearing resistance value at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

The bearing resistance value given for footings 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. Above the groundwater level, adequate lateral support is provided to a stiff silty clay when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered fill.

Permissible Grade Raise

A permissible grade raise restriction has been determined for the subject site based on the undrained shear strength values completed within the silty clay deposit. Based on the testing results, a permissible grade raise restriction of **1 m** above existing ground surface is recommended for the subject site.

To reduce potential long term liabilities, consideration should be given to accounting for a larger groundwater lowering and to providing means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the stores, etc). It should be noted that building over silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking as compared to unreinforced foundations.

5.4 Design for Earthquakes

The site class for seismic site response is a **Class D** for the foundations considered. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4 A) for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of the existing fill, topsoil layer and fill containing organic matter, within the footprint of the proposed building, the native soil or engineered fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone for slab on grade construction. All backfill material 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 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 30 degrees and a bulk (drained) unit weight of 20 kN/m^3 .

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, 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/m^3)

H = height of the wall (m)

g = gravity, 9.81 m/s^2

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

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be considered for the project, the recommended flexible pavement structures shown in Tables 1 and 2 would be applicable.

Table 1 - Recommended Flexible Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill

Table 2 - Recommended Flexible Pavement Structure - Access Lanes	
Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter drainage system is recommended 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 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.

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 composite drainage system, such as Miradrain G100N or Delta Drain 6000. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for backfill material.

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 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 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 Temporary Shoring and Excavation Side Slopes

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of $0.65 K \gamma H$ for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of $K \gamma H$ for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

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.

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 and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's 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

Construction Dewatering

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.

The rate of flow of groundwater into the excavation through the overburden should be low to moderate for the expected at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Permit to Take Water

Since the excavation will most likely not intercept the groundwater table, a temporary MOE permit to take water (PTTW) should not be required for this project. A PTTW is only required if more than 50,000 L/day are to be pumped during the construction phase.

Impacts on Neighbouring Structures

It is understood that only a semi basement is planned for the proposed building. Based on the existing groundwater level (between 4 and 5.5 m below the existing grade), no dewatering is expected. Therefore, no adverse effects to adjacent buildings or structures are expected due to short term or long term groundwater lowering.

Seasonal Fluctuations

It should be noted that groundwater levels are subject to seasonal fluctuation. Therefore, the groundwater level could vary at the time of construction. If higher volumes of groundwater are encountered than expected at the time of construction, then an alternative groundwater arrangement can be considered.

6.6 Winter Construction

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

The subsurface soil conditions at this site mostly consist of frost susceptible materials. In 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. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are in close proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is installed, 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 analytical testing results are presented in Table 3 along with industry standards for the applicable threshold values. These results are indicative that Type 10 Portland cement (Type GU, or normal cement) would be appropriate for this site.

Table 3 - Corrosion Potential			
Parameter	Laboratory Results	Threshold	Commentary
	TP5-GR5		
Chloride	626 µg/g	Chloride content less than 400 mg/g	Negligible concern
pH	7.23	pH value less than 5.0	Neutral Soil
Resistivity	9.39 ohm.m	Resistivity greater than 1,500 ohm.cm	Moderate Corrosion Potential
Sulphate	298 µg/g	Sulphate value greater than 1 mg/g	Negligible Concern

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.
- Observation of the placement of the foundation insulation, if applicable.
- 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 and follow-up 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 are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A 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, we request immediate notification 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 Milito Investments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Stephanie A. Boisvenue, P.Eng.

David J. Gilbert, P.Eng.



Report Distribution:

- Milito Investments(3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

DATUM TBM - Top spindle of fire hydrant. An arbitrary elevation of 100.00m was assigned to the TBM.

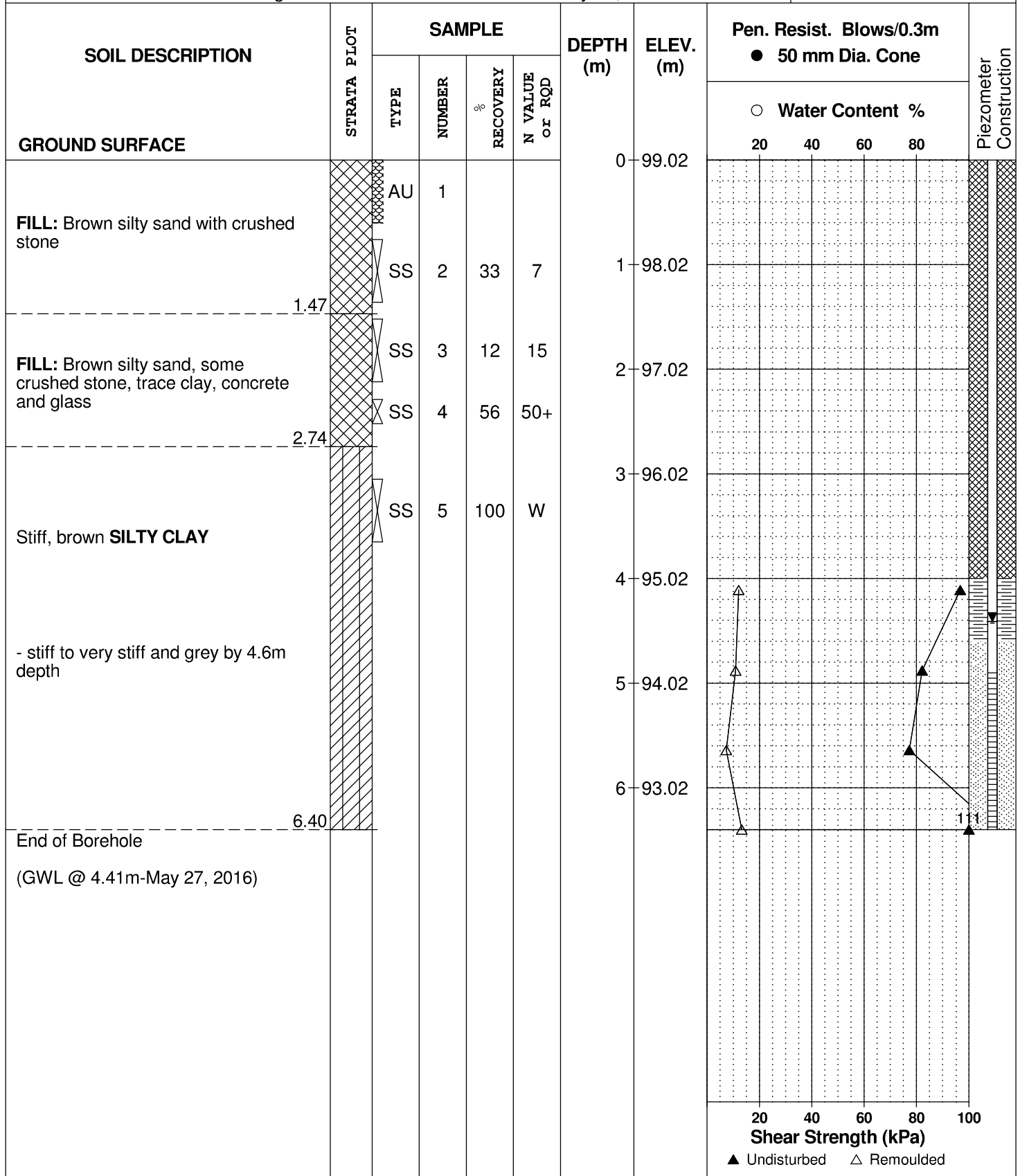
REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2016

FILE NO.
PG3825

HOLE NO.
BH 1



DATUM TBM - Top spindle of fire hydrant. An arbitrary elevation of 100.00m was assigned to the TBM.

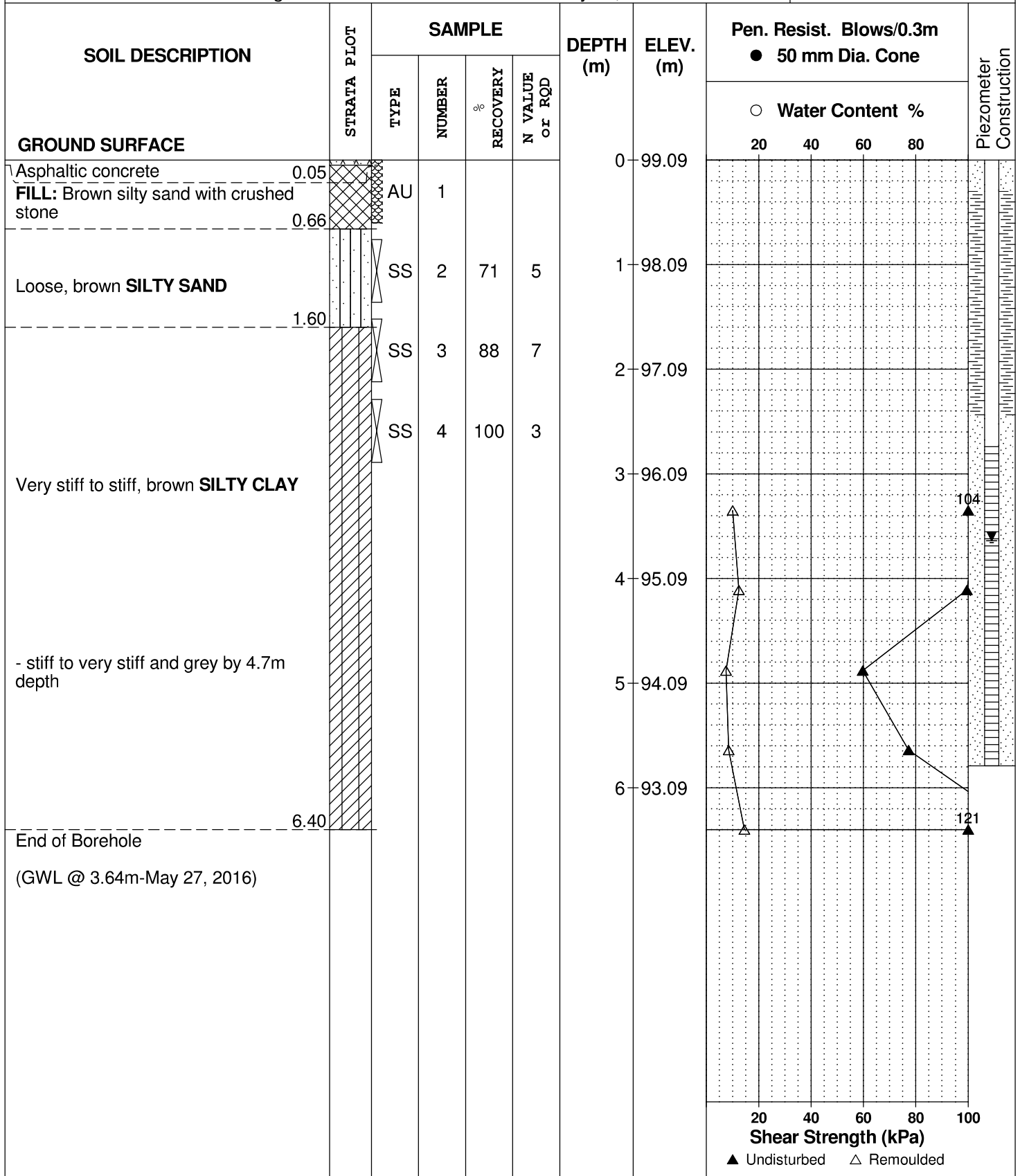
REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2016

FILE NO.
PG3825

HOLE NO.
BH 2



DATUM TBM - Top spindle of fire hydrant. An arbitrary elevation of 100.00m was assigned to the TBM.

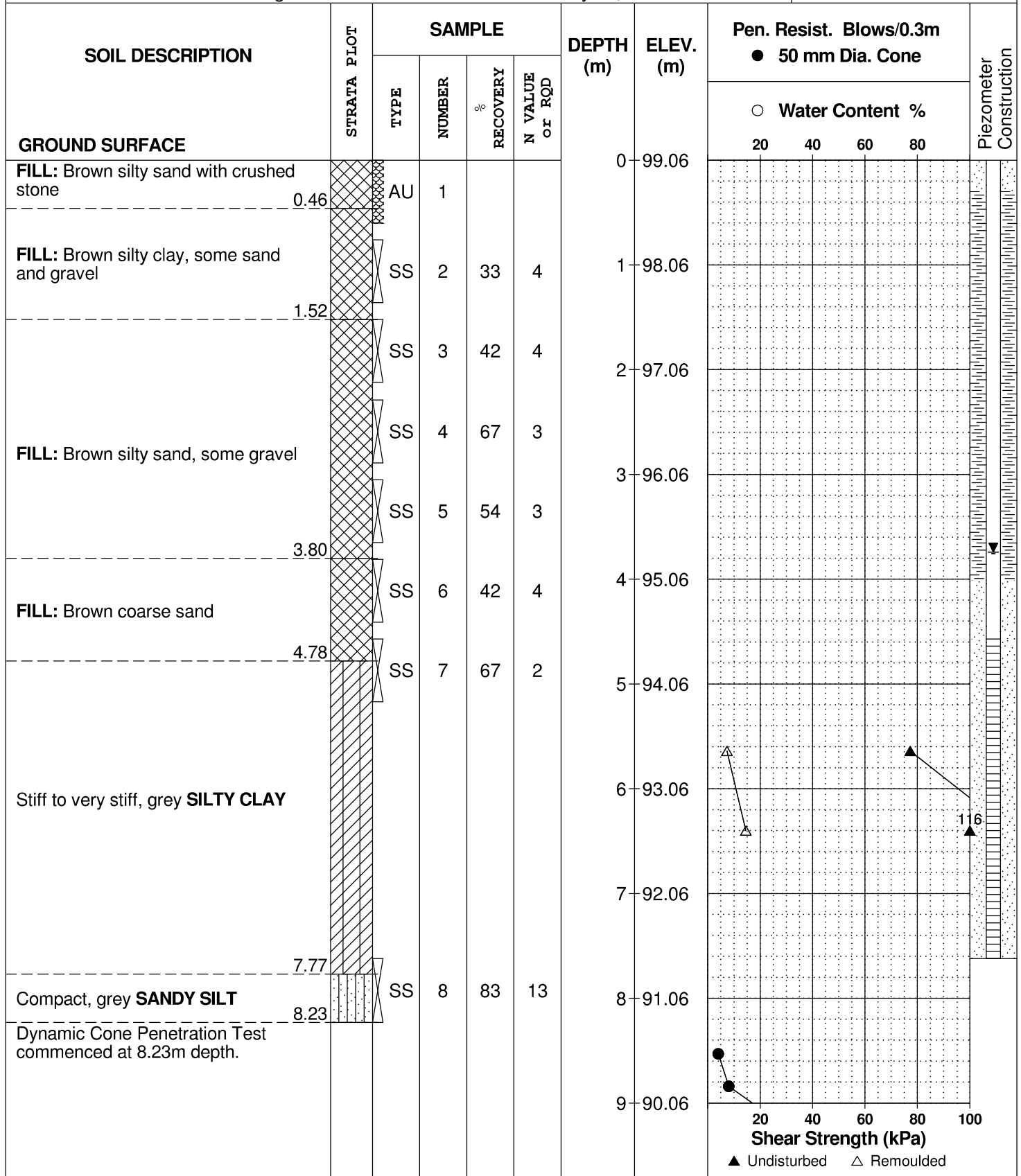
REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2016

FILE NO.
PG3825

HOLE NO.
BH 3



DATUM TBM - Top spindle of fire hydrant. An arbitrary elevation of 100.00m was assigned to the TBM.

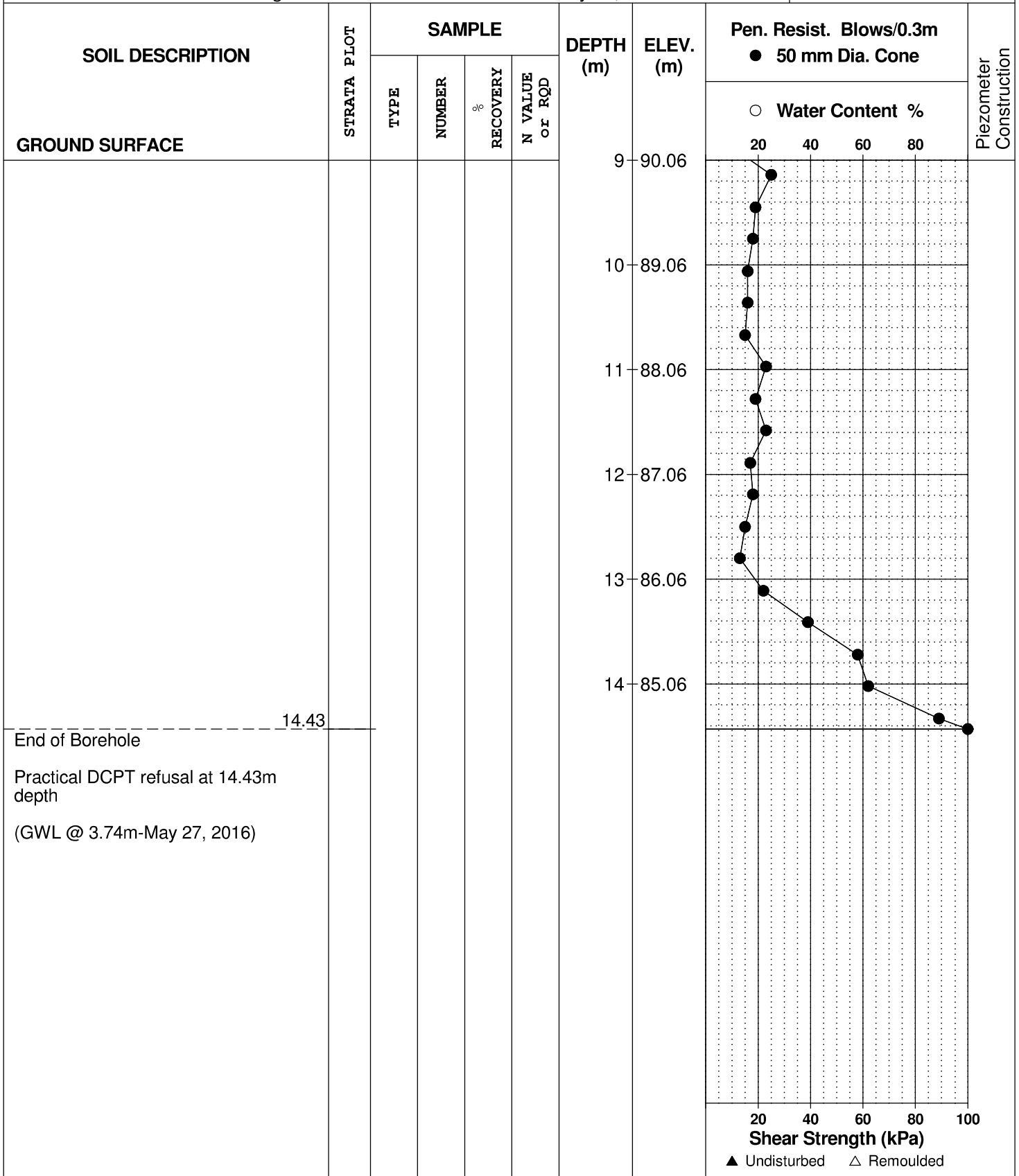
REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2016

FILE NO.
PG3825

HOLE NO.
BH 3



DATUM TBM - Top spindle of fire hydrant. An arbitrary elevation of 100.00m was assigned to the TBM.

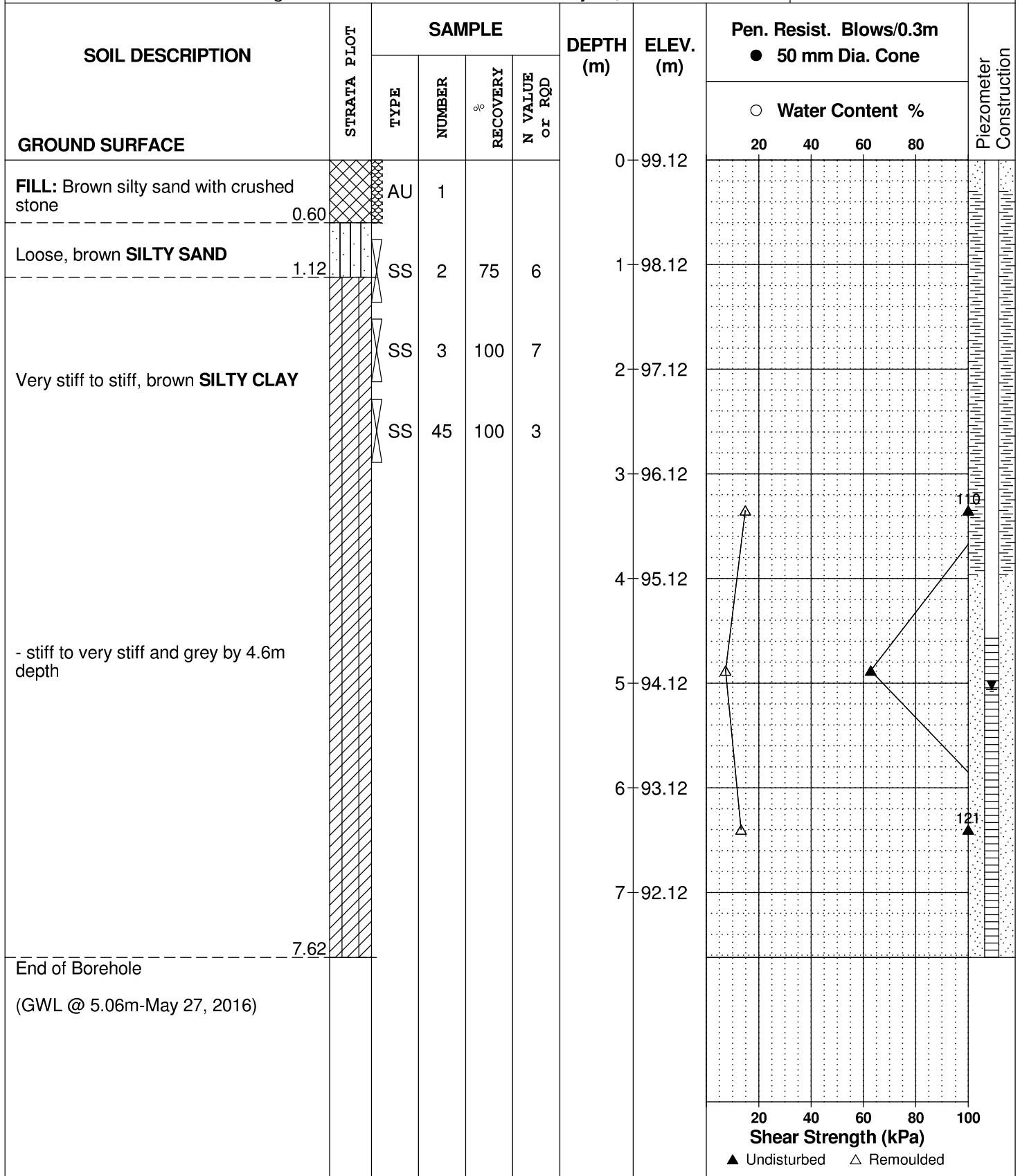
REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2016

FILE NO.
PG3825

HOLE NO.
BH 4



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

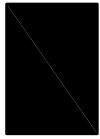
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void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

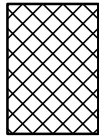
STRATA PLOT



Topsoil



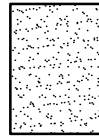
Asphalt



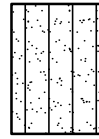
Fill



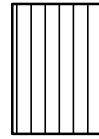
Peat



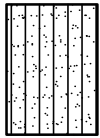
Sand



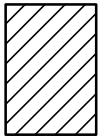
Silty Sand



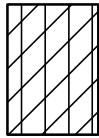
Silt



Sandy Silt



Clay



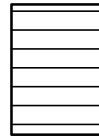
Silty Clay



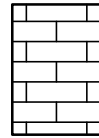
Clayey Silty Sand



Glacial Till



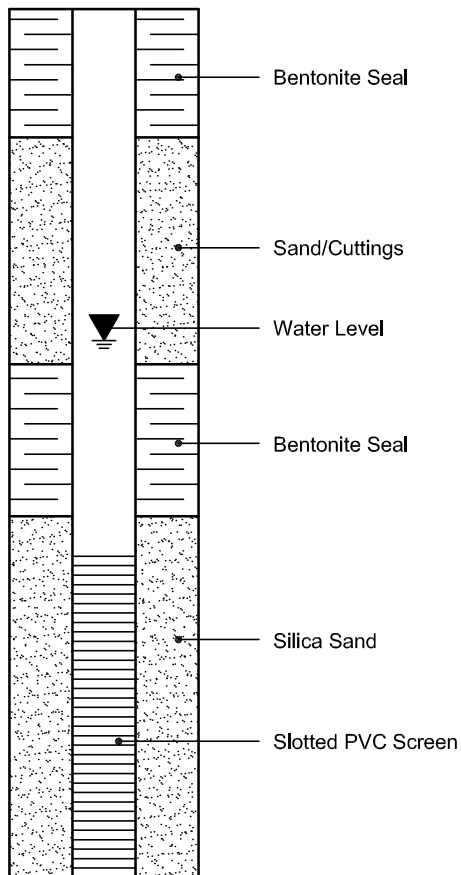
Shale



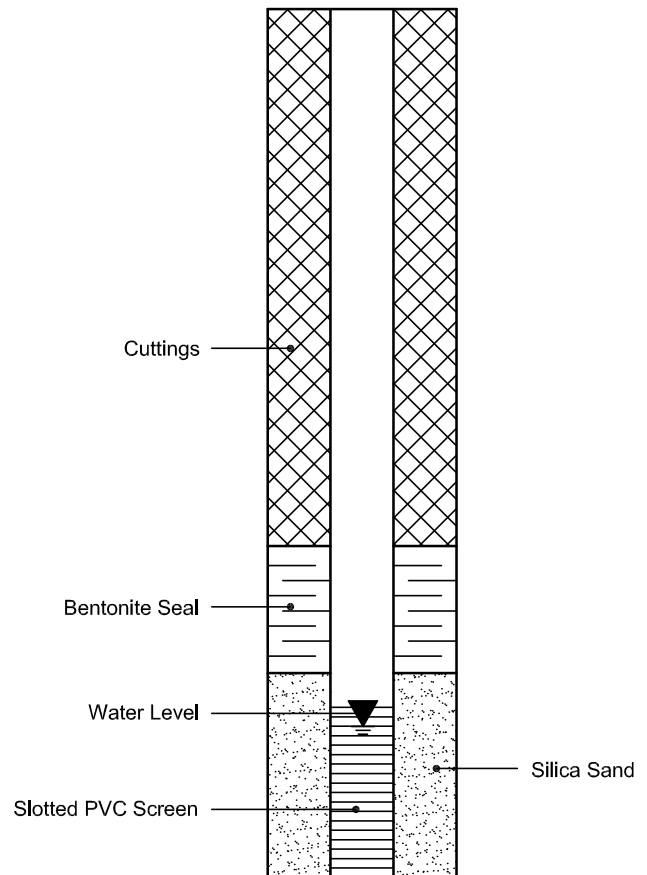
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
 Client: Paterson Group Consulting Engineers
 Client PO: 13453

Report Date: 26-May-2016

Order Date: 20-May-2016

Project Description: PG3825

Client ID:	BH4-SS2	-	-	-
Sample Date:	18-May-16	-	-	-
Sample ID:	1621490-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	69.5	-	-	-
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General Inorganics

pH	0.05 pH Units	7.23	-	-	-
Resistivity	0.10 Ohm.m	9.39	-	-	-

Anions

Chloride	5 ug/g dry	626	-	-	-
Sulphate	5 ug/g dry	298	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3825-1 - TEST HOLE LOCATION PLAN

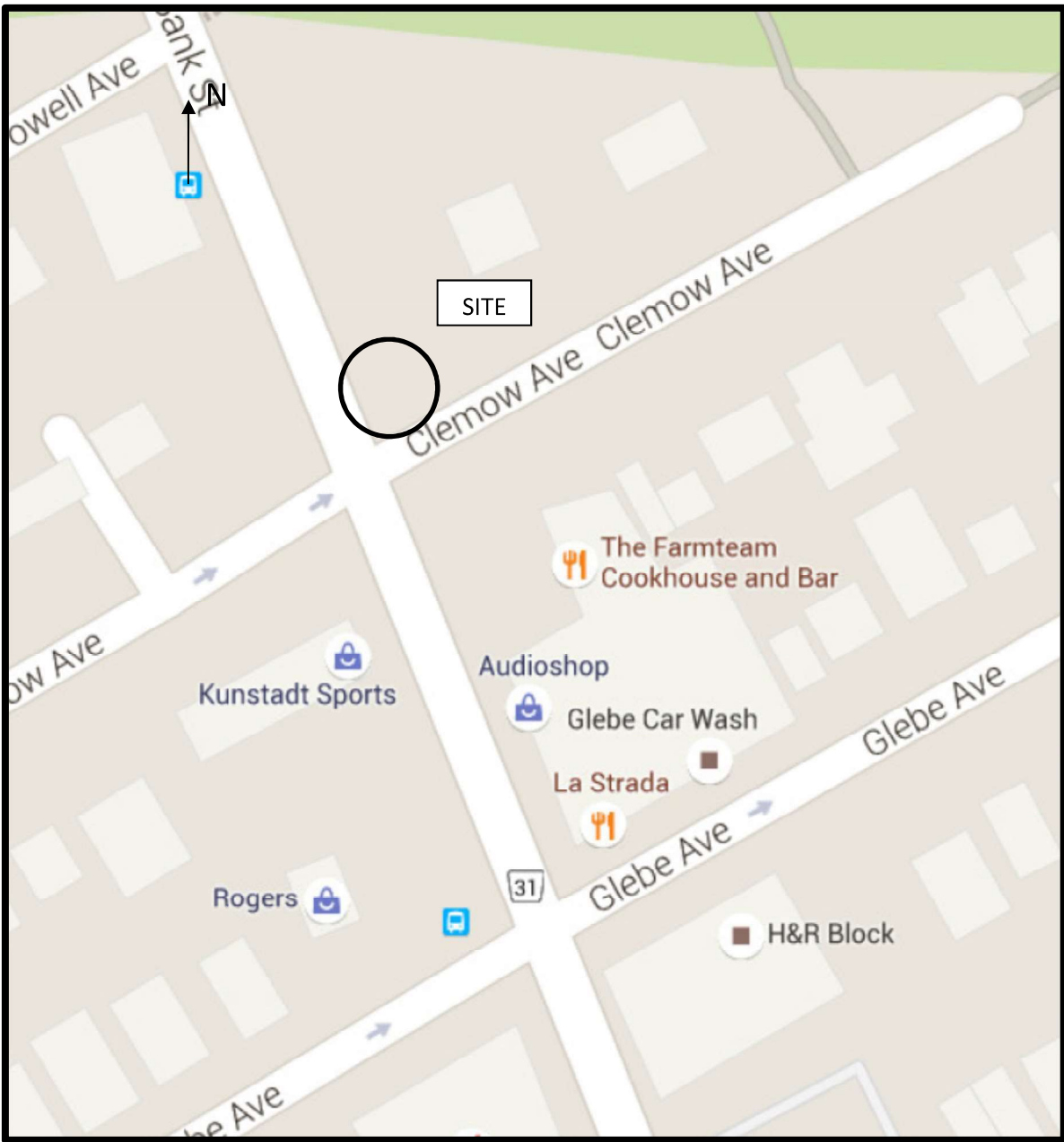
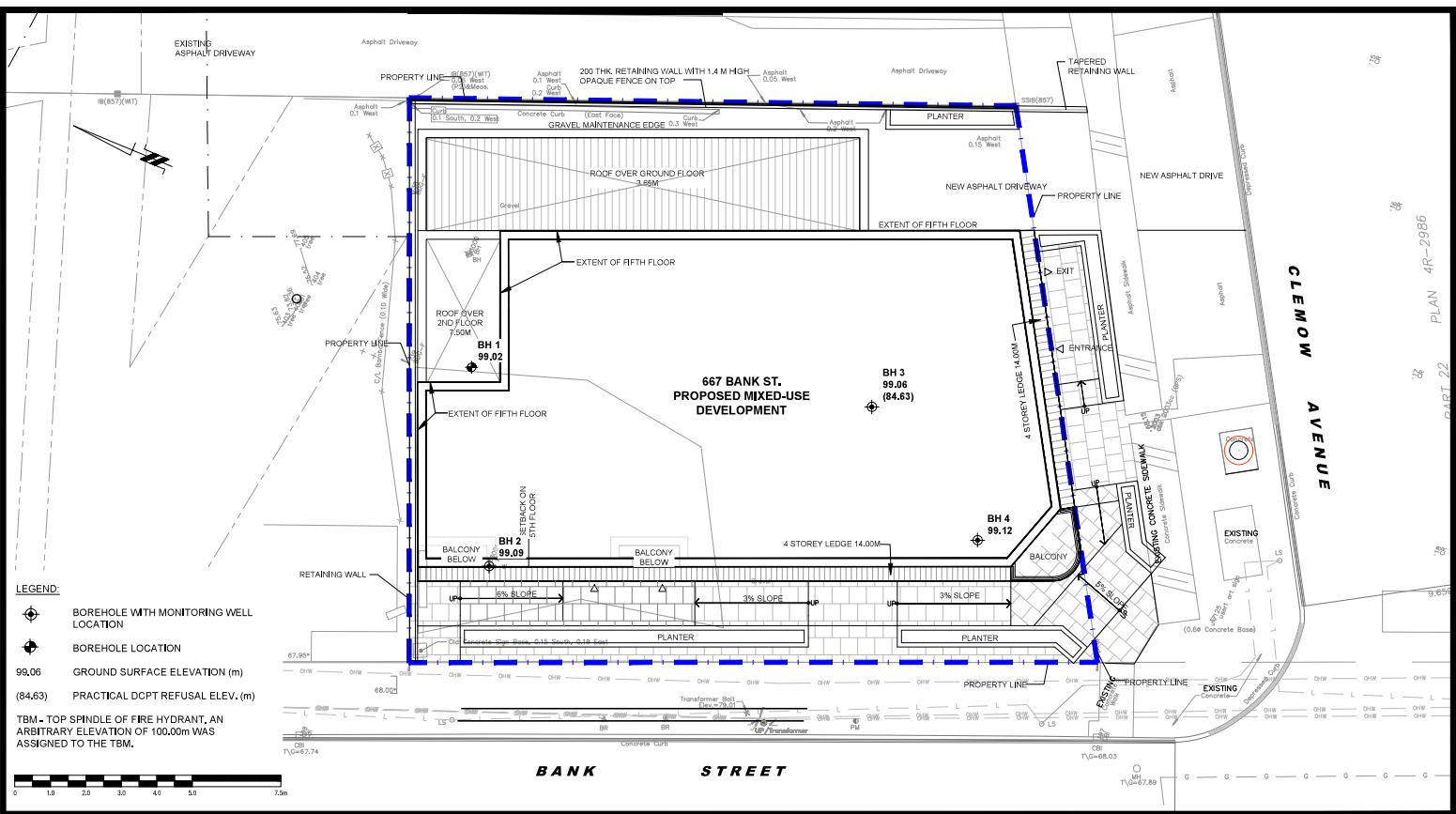
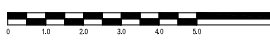


FIGURE 1
KEY PLAN



- LEGEND:**
- BOREHOLE WITH MONITORING WELL LOCATION
 - BOREHOLE LOCATION
 - 99.06 GROUND SURFACE ELEVATION (m)
 - (84.63) PRACTICAL DCPT REFUSAL ELEV. (m)

TBM - TOP SPINDLE OF FIRE HYDRANT, AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.



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NO.	REVISIONS	DATE	INITIAL
1	REVISED TO LATEST CONCEPTUAL PLAN	10/27/2021	BN

MILITO INVESTMENTS
GEO TECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - 667 BANK STREET
OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale:	1:100	Date:	06/2016
Drawn by:	RG	Report No.:	PG3825-1
Checked by:	SB	Dwg. No.:	PG3825-1
Approved by:	DJG	Revision No.:	1