

**Geotechnical
Engineering**

**Environmental
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Engineering**

Materials Testing

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Geotechnical Investigation

Proposed Commercial Development
West Ottawa Land Holdings 1 and 2
Huntmar Drive at Campeau Drive
Ottawa, Ontario

Prepared For

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Report: PG3115-1

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1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Taggart Group of Companies to conduct a geotechnical investigation for the proposed commercial development to be located at the northwest corner of the intersection of Huntmar Drive and Campeau Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the investigation was to:

- ☐ determine the subsoil and groundwater conditions at this site by means of test holes.
- ☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 PROPOSED DEVELOPMENT

Specific details of the proposed development were not available at the time of issuance of this report. However, it is expected that the proposed commercial development is to consist of commercial buildings of slab on grade construction along with associated access lanes, parking areas and landscaped areas.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the current investigation was carried out on January 9, 10, 13 to 15, 2014. At that time, 26 boreholes were spaced across the subject site. A previous investigation was carried out by Paterson in 2010 and included five (5) boreholes to a maximum 10 m depth. Also, a previous investigation was carried out by Paterson in 2006, and included four (4) test pits excavated to a maximum depth of 4.1 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG3115-1 - 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. The test pits were completed by a rubber tired backhoe at the selected locations across the site. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedures consisted of augering or excavating to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered during drilling from the auger flights or a 50 mm diameter split-spoon sampler. The split-spoon samples were classified on site and placed in sealed plastic bags. Grab samples were collected from the test pits at selected intervals. All samples were transported to our laboratory. The depths at which the auger flight, split-spoon and grab samples were recovered from the boreholes are depicted as AU, SS, and G, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

Overburden thickness was also evaluated during the course of the investigation by completing a dynamic cone penetration test (DCPT) at BH 1 and BH 11. 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.

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 PVC standpipes were installed in all boreholes to permit the monitoring of groundwater 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 by Paterson and laid out in the field by Stantec Geomatics. It is understood that the elevations are 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 PG3115-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were 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 potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The analytical testing results were not available at the time of issuance of this report. An addendum to the report will be issued with the testing results, once the testing is completed.

4.0 OBSERVATIONS

4.1 Surface Conditions

Generally, the ground surface across the subject site slopes downward to the northeast. The majority of the subject site is currently used for agricultural purposes. The west portion of the subject site is occupied by former agricultural land overgrown with brush and sparse trees. An existing quarry is located along the west property boundary of the subject site. A former farmstead was previously located within the northeast portion of the subject site.

A section of Feedmill Creek meanders in a west to east direction toward Carp River along the south property boundary within the southwest portion of the subject site. The subject section of Feedmill Creek is located within a 40 to 50 m wide valley corridor with a 1 to 4 m high valley wall. The majority of the valley corridor along the subject section of Feedmill Creek is not well defined with the valley walls close to 1 m in height or less. It was noted that the watercourse is approximately 0.3 to 0.6 m deep and confined within a 1 to 2 m wide channel, which meanders across the valley corridor floor.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations within the west portion of the subject site consists of topsoil overlying a loose to compact brown silty fine sand overlying a glacial till deposit. The subsurface profile encountered at the test hole locations within the east portion of the subject site consists of topsoil underlain by a silty clay deposit. A stiff to very stiff brown silty clay crust overlies a firm to stiff grey silty clay. Centrally, several test hole locations encountered a stiff to very stiff brown silty clay crust overlying sandy silty/silty sand and a glacial till layer. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam formation. Also, the bedrock surface is expected at depths ranging from 5 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the boreholes from the 2010 investigation in December 2010. The measured groundwater level (GWL) 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 recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level varies between 1.5 to 3 m depth. Groundwater level observations are presented in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed commercial development. It is expected that the proposed commercial buildings can be constructed over conventional shallow footing foundations.

A slope stability analysis was completed for the valley corridor walls of Feedmill Creek within the south portion of the subject site. The results of our analysis and our rationale for the limit of hazard lands designation are discussed in Subsection 6.7. The limit of hazard lands setback line is presented in Drawing PG3115-2 - Limit of Hazard Lands in Appendix 2.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

Fill Placement

Fill used for grading beneath the building footprints, 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 area 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 ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site-excavated soils are not suitable for use as backfill against foundation walls 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

Strip footings, up to 2 m wide, and pad footings, up to 5 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**.

Footings placed on an undisturbed, compact sandy silt/silty sand bearing surface can be designed using a factored bearing resistance value at SLS of **100 kPa** and a bearing resistance value at ULS of **200 kPa**.

Where the silty sand bearing surface is found to be in a loose state of compactness, the area should be proof-rolled using a vibratory compactor and approved by the geotechnical consultant prior to placing footings.

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

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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 stiff silty clay 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.

Settlement/Grade Raise

Based on the undrained vane testing results within the silty clay deposit, it is recommended that a **permissible grade raise restriction of 2 m** be implemented for areas where foundations are placed over the silty clay deposit.

A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations. 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 buildings, etc). It should be noted that building on 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 can be taken as **Class D** for foundations constructed at the subject 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 Slab on Grade Construction

With the removal of all topsoil and deleterious materials, within the footprint of the proposed buildings, the native soil or engineered fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 150 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings 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 Pavement Design

Car only parking areas, local roadways are anticipated at this site. The proposed pavement structures are shown in Tables 1 and 2.

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

Table 2 - Recommended Pavement Structure - Local Roadways	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - 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 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.

For areas where silty clay is encountered at subgrade level, it is recommended that subdrains be installed 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

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 mm to 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 drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the 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 soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by "cut and cover" methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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 should be used for bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). 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.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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 that are located in the areas underlain by silty clay. The seals should be at least 1.5 m long (in the trench direction) 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

It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

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.

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 Limit of Hazard Lands

Slope Conditions

A geotechnical limit of hazard lands setback line has been provided along the top of slope for the valley corridor walls of the Feedmill Creek valley corridor. One slope cross-section was studied as the worst case scenario. The cross section location is presented on Drawing PG3115-2 - Limit of Hazard Lands in Appendix 2. The subject section of Feedmill Creek is located with a 40 to 50 m wide valley corridor with a 1 to 4 m high valley wall. The valley corridor is less defined within the west portion of the site, where the valley walls are close to 1 m or less. The majority of the slope face was noted to be grass covered with minor surficial erosional activities noted. Some sloughing and minor undercutting along the valley wall face was noted where the watercourse has meandered in close proximity to the slope. The 1 to 2 m wide watercourse, which varies in depth between 0.3 to 0.6 m, meanders across the valley floor.

Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-sections were analyzed taking into account a groundwater level at ground surface. Subsoil conditions at the cross-sections were inferred based on the findings at nearby borehole locations and general knowledge of the area's geology.

Static Analysis

The results for the existing slope conditions at Section A are shown in Figure 2 in Appendix 2. The factor of safety was found to be less than 1.5 for Section A when analyzed under static conditions. A stable slope allowance of 2.9 m is required for Section D.

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal seismic acceleration, K_h , of 0.21G was considered for the analyzed sections. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figure 3 for the slope sections. The results indicate that the factor of safety for Section A is greater than 1.1. Based on these results, the slopes are considered to be stable under seismic loading.

Limit of Hazard Lands

The limit of hazard lands includes a 6 m erosion access allowance taken from the top of slope. However, a limit of hazard lands setback is not required within the west portion of the site, where the valley corridor wall of Feedmill Creek becomes less than 2 m high. It should be noted that based on our analysis results, the majority of the slope is considered stable. The limit of hazard lands setback line for the proposed development is indicated on Drawing PG3115-2 - Limit of Hazard Lands in Appendix 2.

The toe erosion allowance for the valley corridor wall slopes was based on the cohesive nature of the soils, the observed current erosional activities and the width and location of the current watercourse. Signs of erosion were noted along the existing watercourse, especially where the watercourse has meandered in close proximity to the toe of the corridor wall. It is considered that a toe erosion allowance of 5 m is appropriate for the corridor walls confining the existing watercourse. The toe erosion allowance should be applied from the top of stable slope, where the watercourse has meandered to within 15 m of the slope toe. The toe erosion allowance should be taken from the bank full water's edge in areas, where the watercourse is greater than 10 m from the toe of the existing slope.

If the existing vegetation needs to be removed along the slope face, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed or an erosional control blanket be placed across the exposed slope face.

7.0 RECOMMENDATIONS

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- ☐ Review master grading plan from a geotechnical perspective, once available.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

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. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. 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 Taggart Group of Companies 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.



David J. Gilbert, P.Eng.



Carlos P. Da Silva, P.Eng.

Report Distribution:

- ☐ Taggart Group of Companies (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

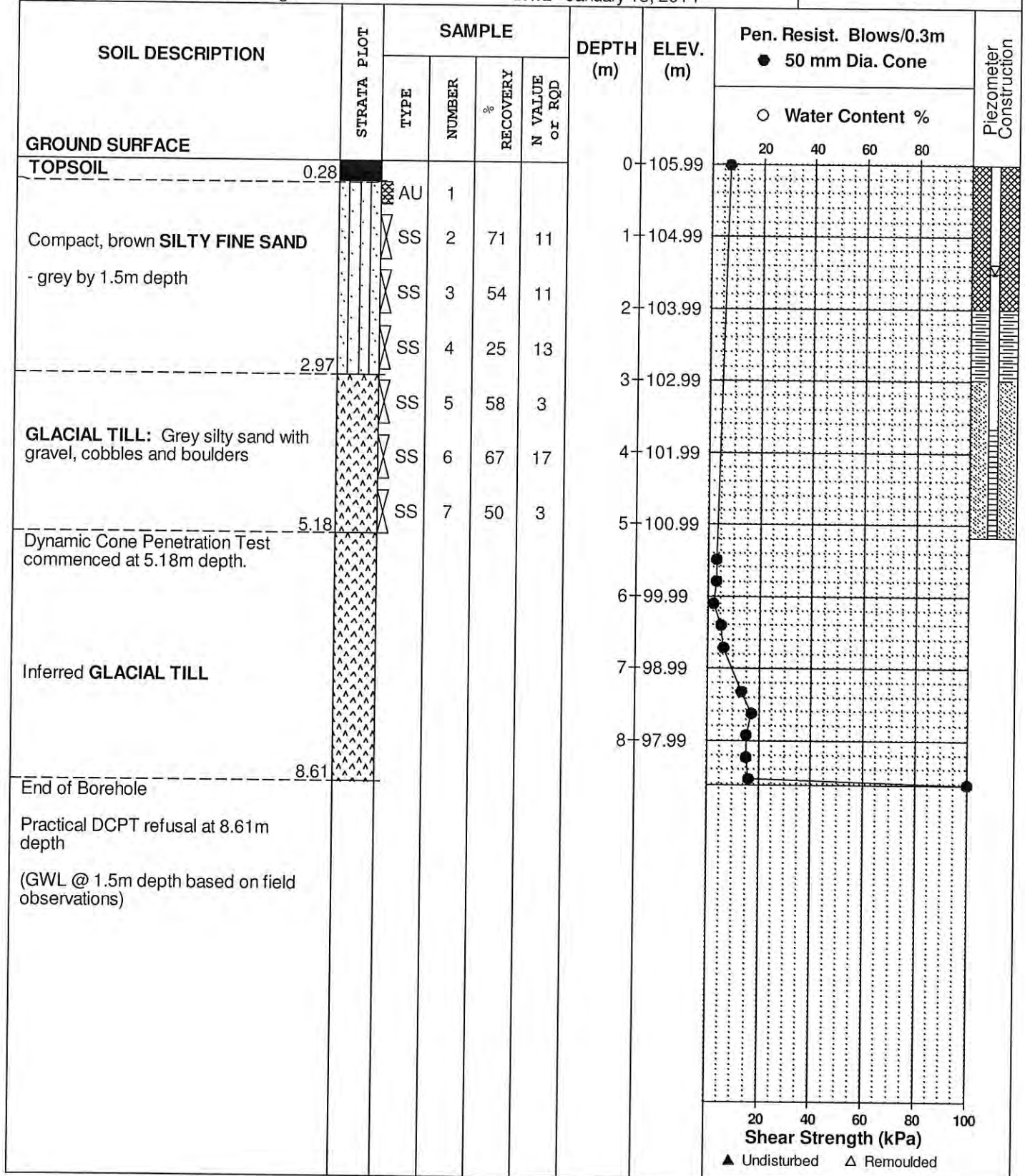
REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

HOLE NO. **BH 1**



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

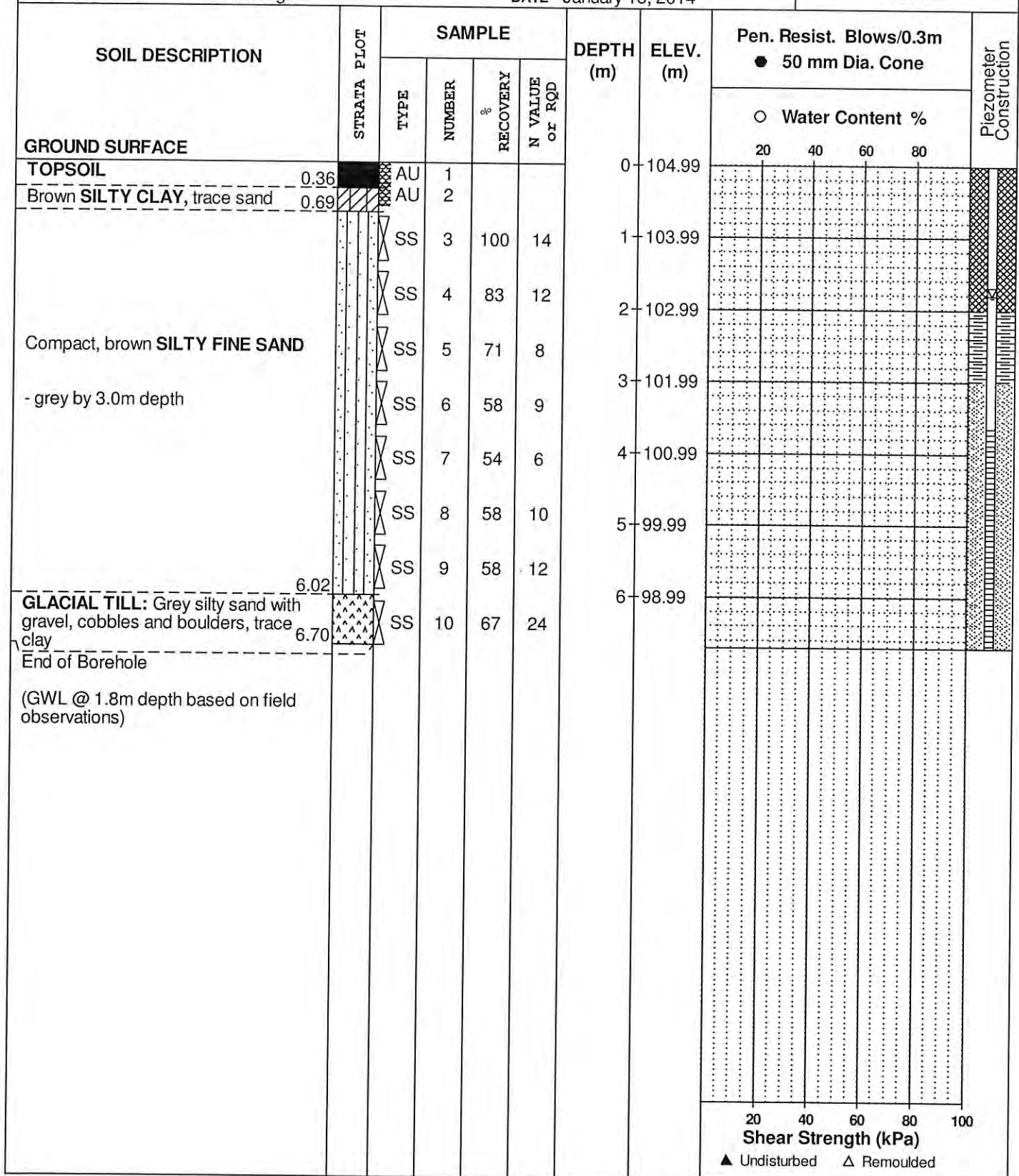
REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

HOLE NO. **BH 2**



154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

**Proposed Commercial Development - Huntmar Road
Ottawa, Ontario**

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content % 20 40 60 80				
GROUND SURFACE												
TOPSOIL	0.36	AU	1			0	104.68					
Brown SILTY CLAY , trace sand	0.69											
Compact, brown SILTY FINE SAND - grey by 2.2m depth		SS	2	67	12	1	103.68					
		SS	3	83	20	2	102.68					
		SS	4	83	13							
		SS	5	54	19	3	101.68					
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, trace clay	4.27	SS	6		26	4	100.68					
End of Borehole	5.21					5	99.68					
Practical refusal to augering at 5.21m depth												
(GWL @ 1.8m depth based on field observations)												

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

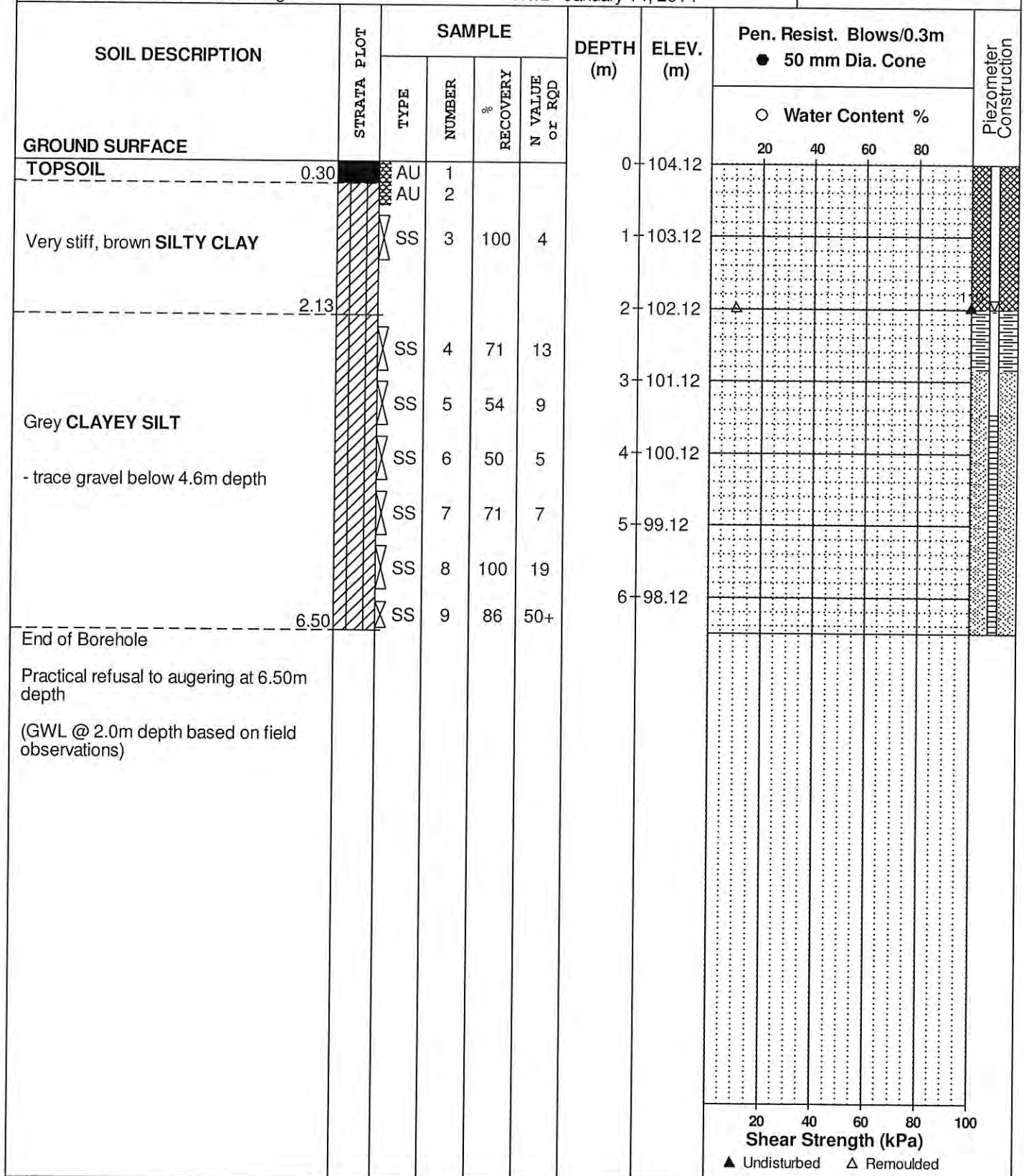
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

FILE NO. **PG3115**

HOLE NO. **BH 4**



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

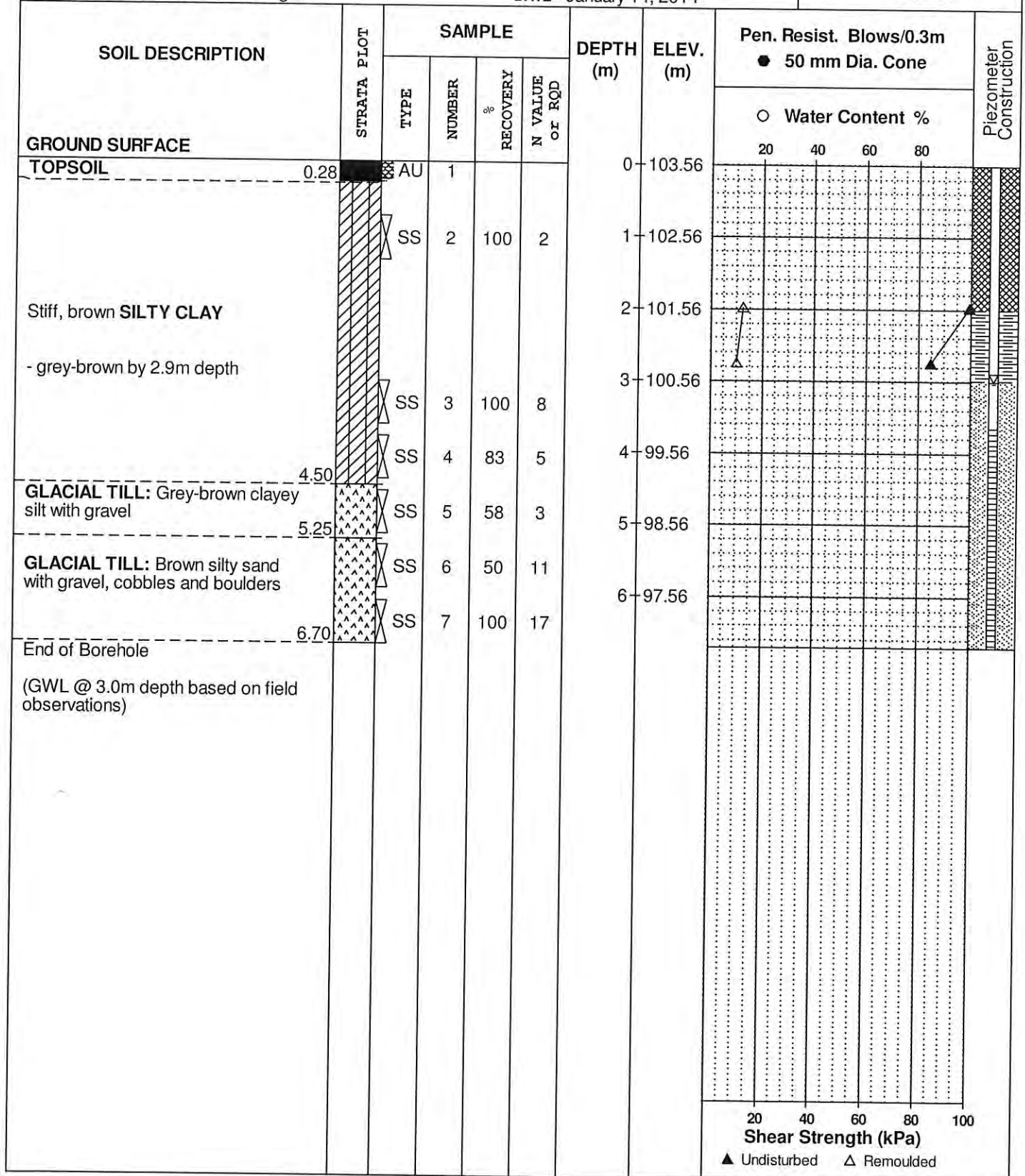
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

FILE NO. **PG3115**

HOLE NO. **BH 5**



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

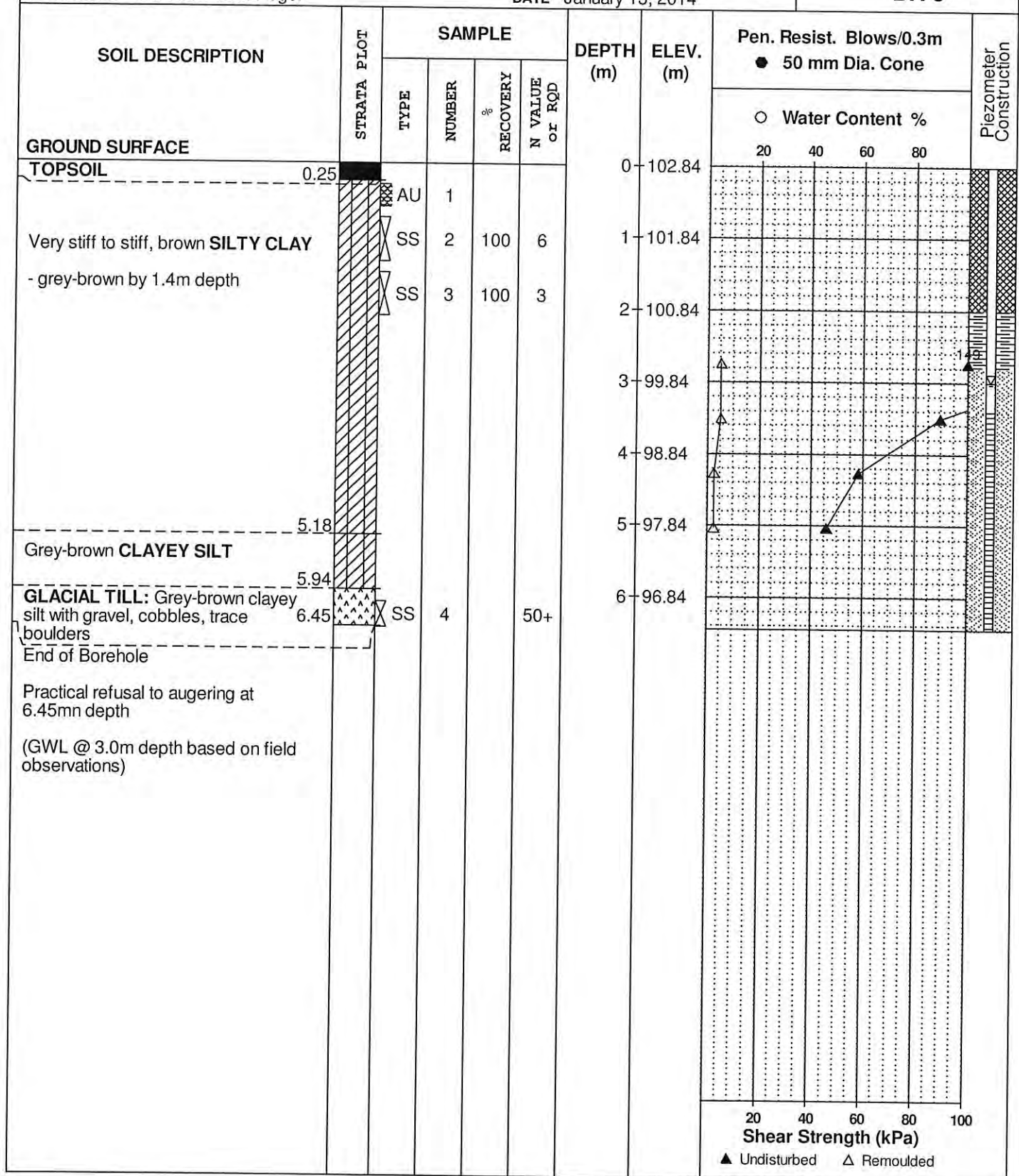
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 13, 2014

FILE NO. PG3115

HOLE NO. BH 6



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

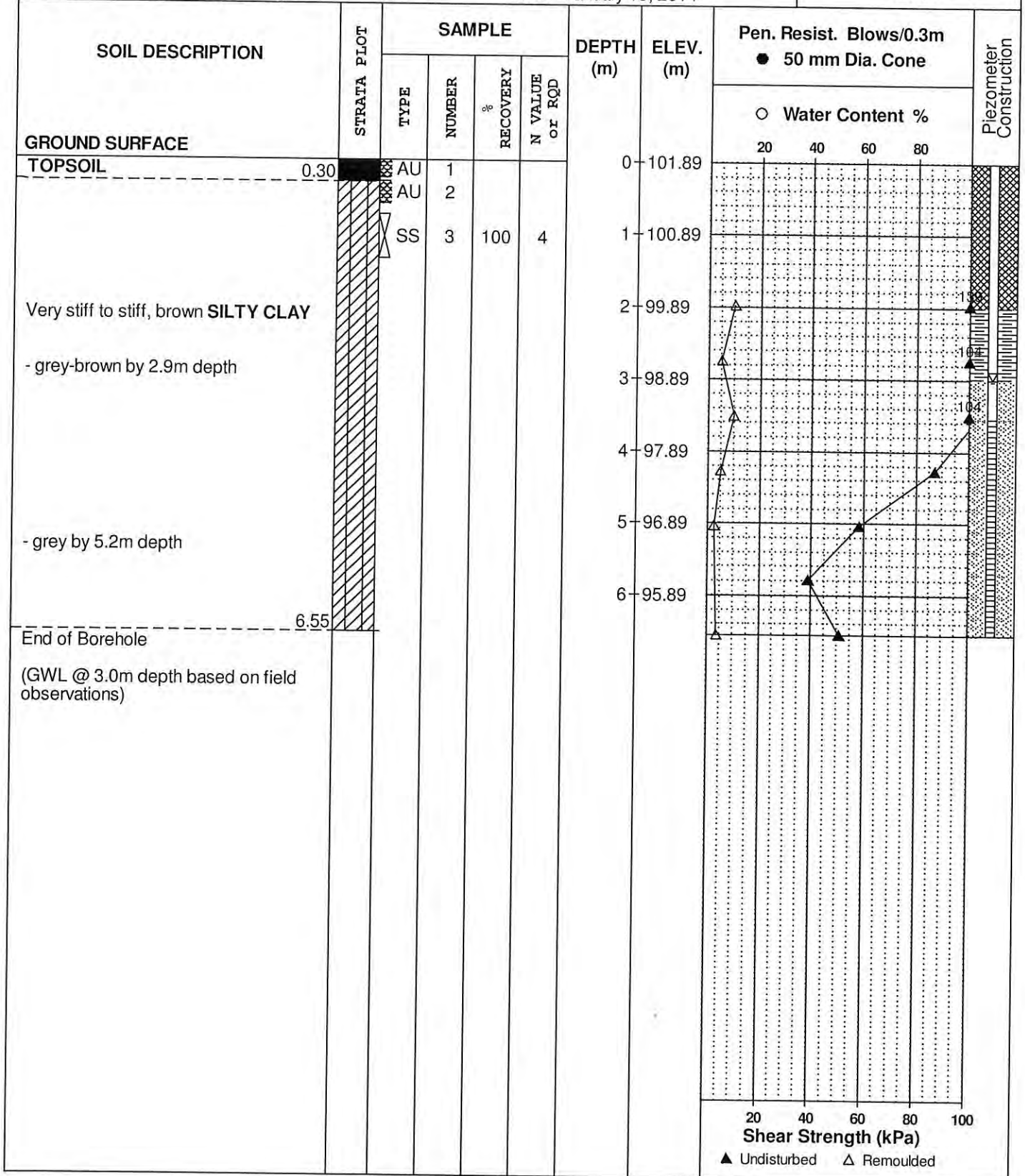
DATE January 13, 2014

FILE NO.

PG3115

HOLE NO.

BH 7



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

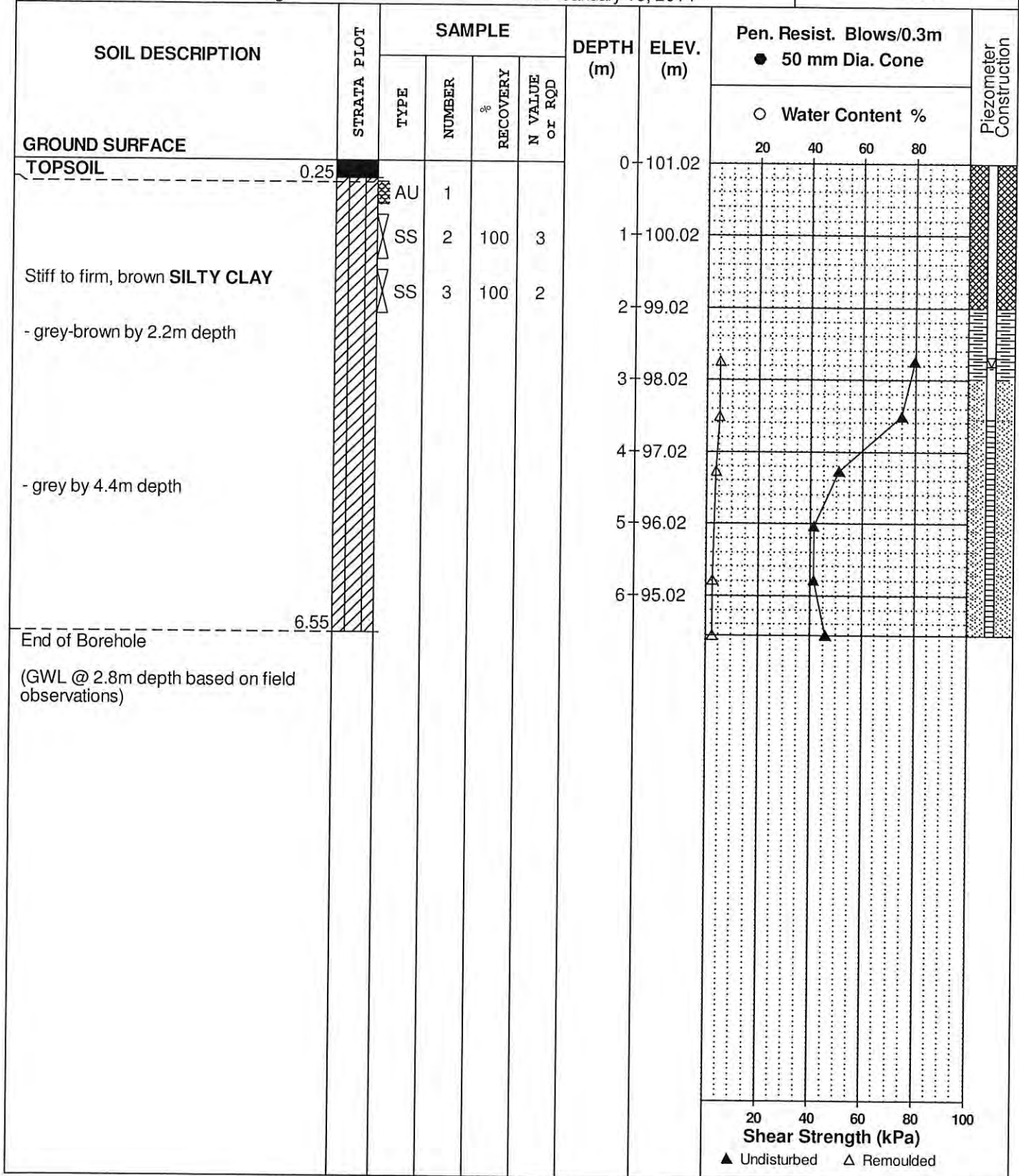
REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 10, 2014

HOLE NO. **BH 8**



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

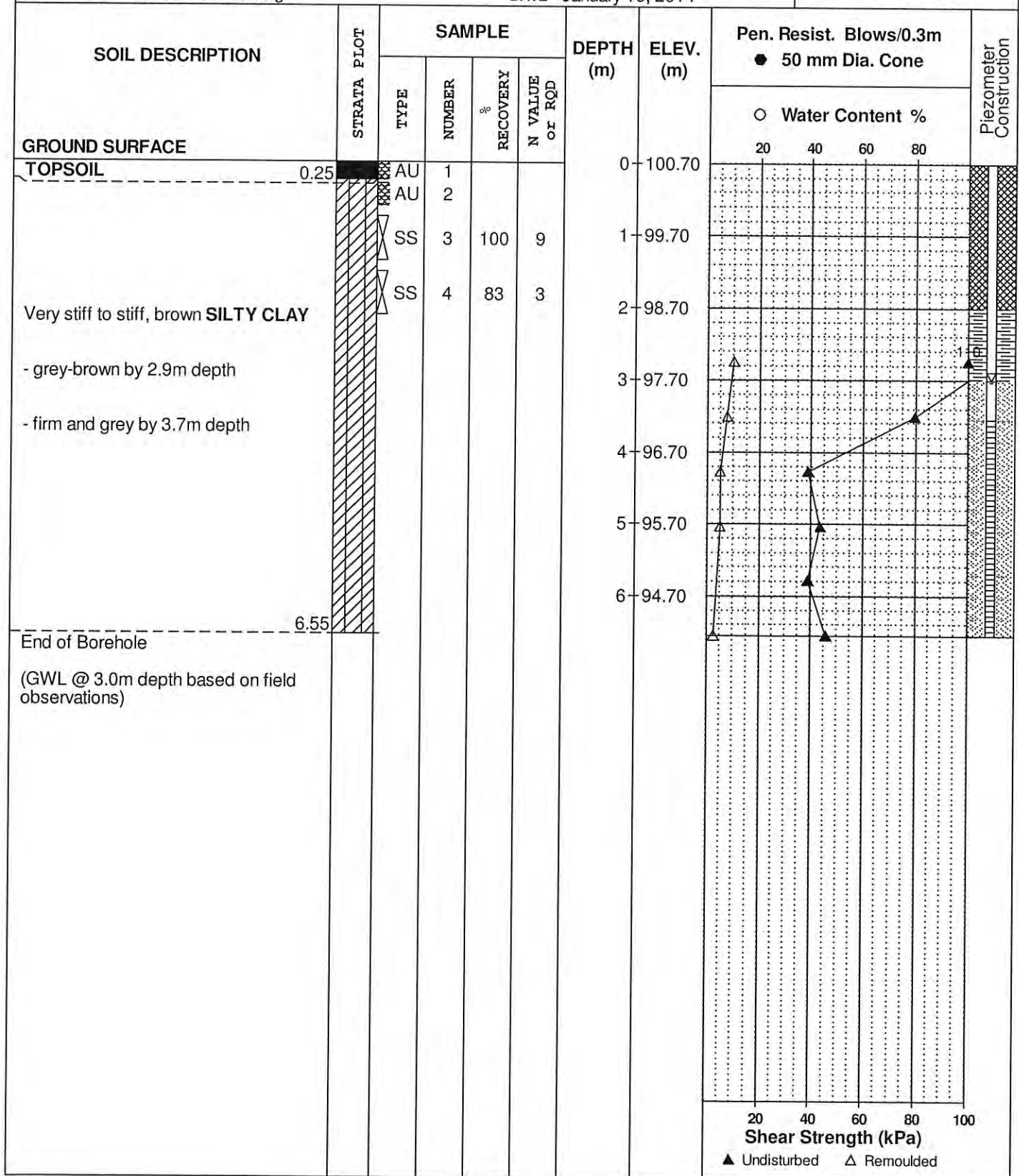
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 10, 2014

FILE NO. PG3115

HOLE NO. BH 9



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

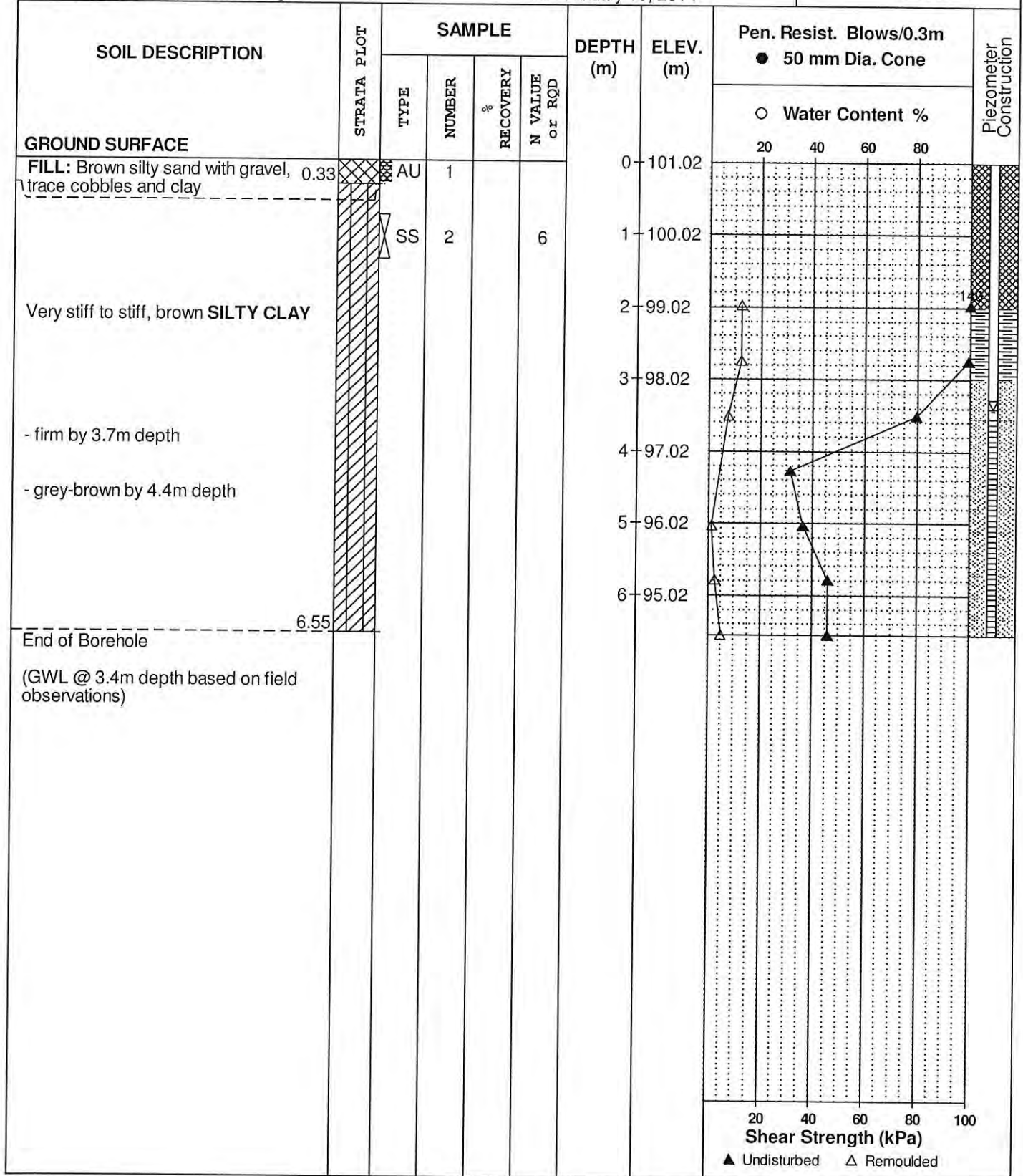
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 10, 2014

FILE NO. PG3115

HOLE NO. BH10



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

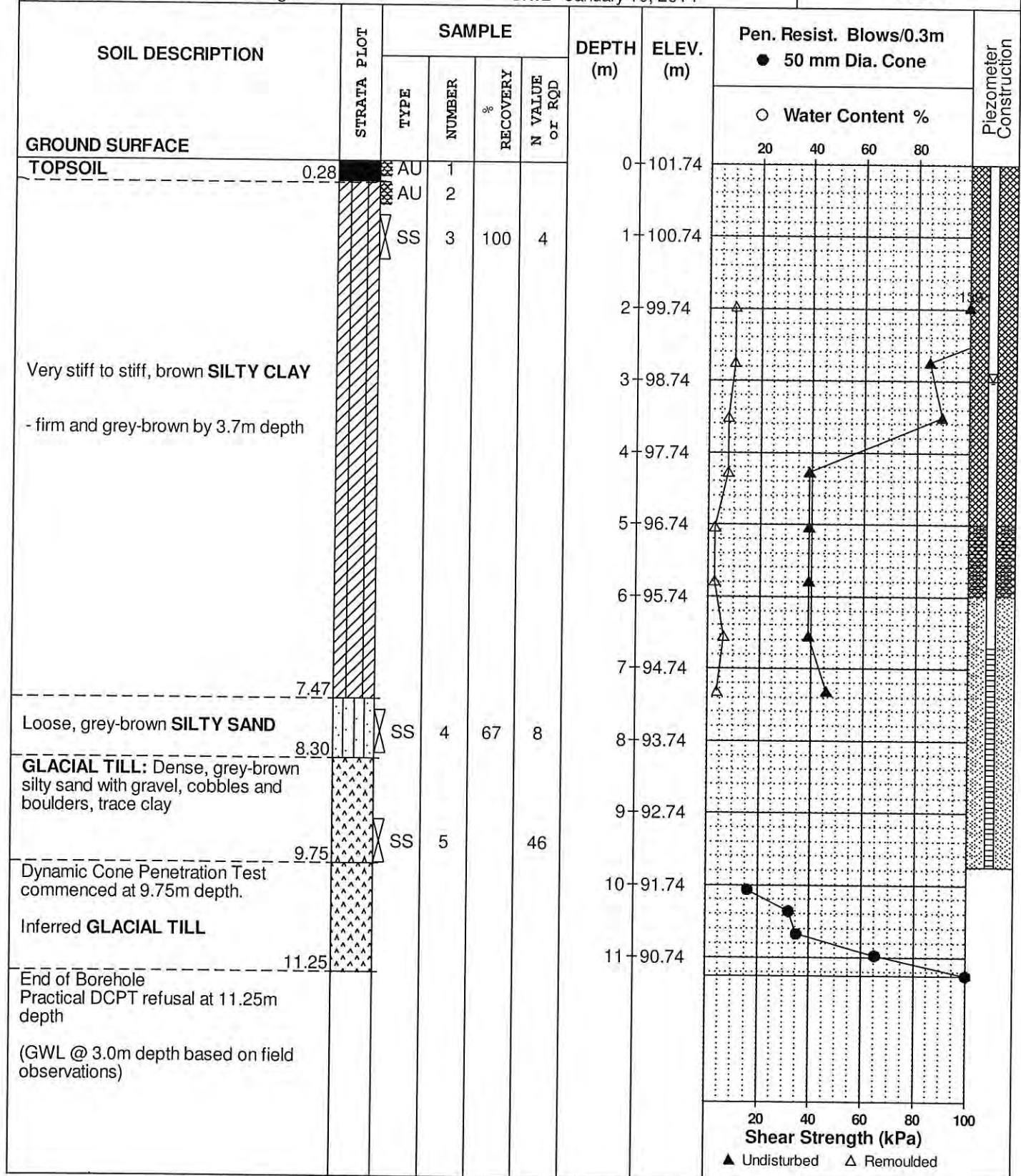
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 10, 2014

FILE NO. PG3115

HOLE NO. BH11



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

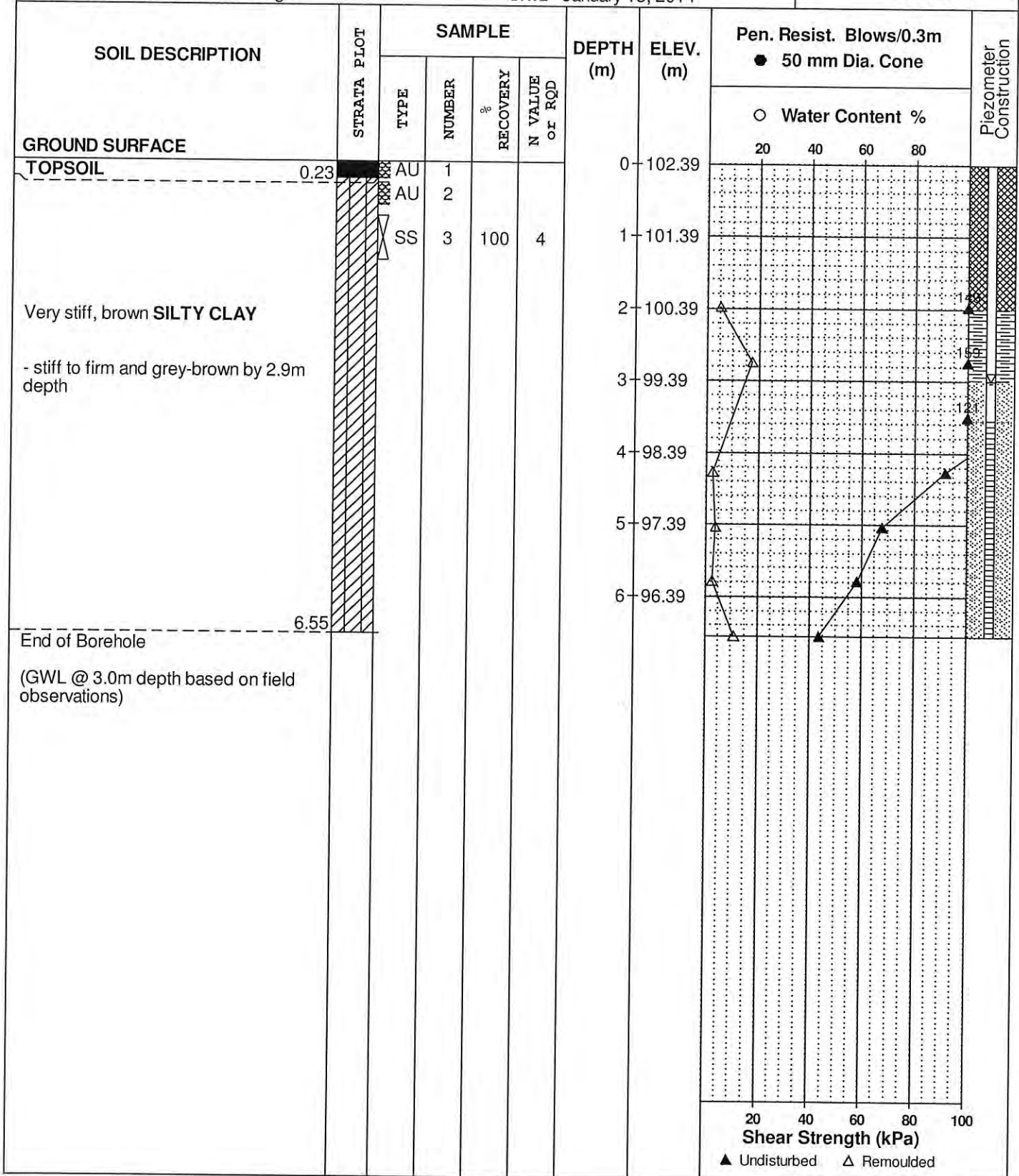
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 13, 2014

FILE NO. PG3115

HOLE NO. BH12



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

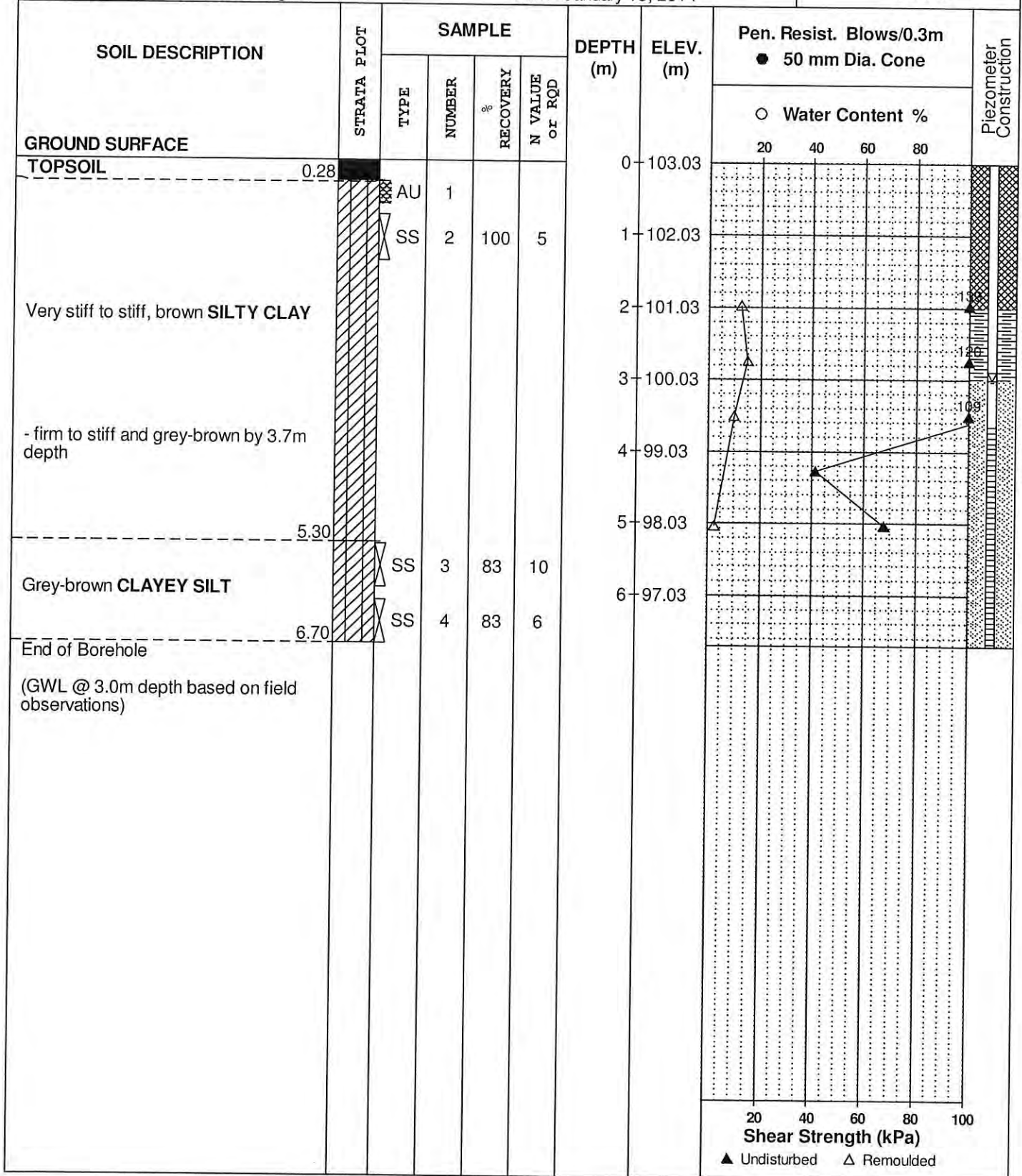
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 13, 2014

FILE NO. PG3115

HOLE NO. BH13



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

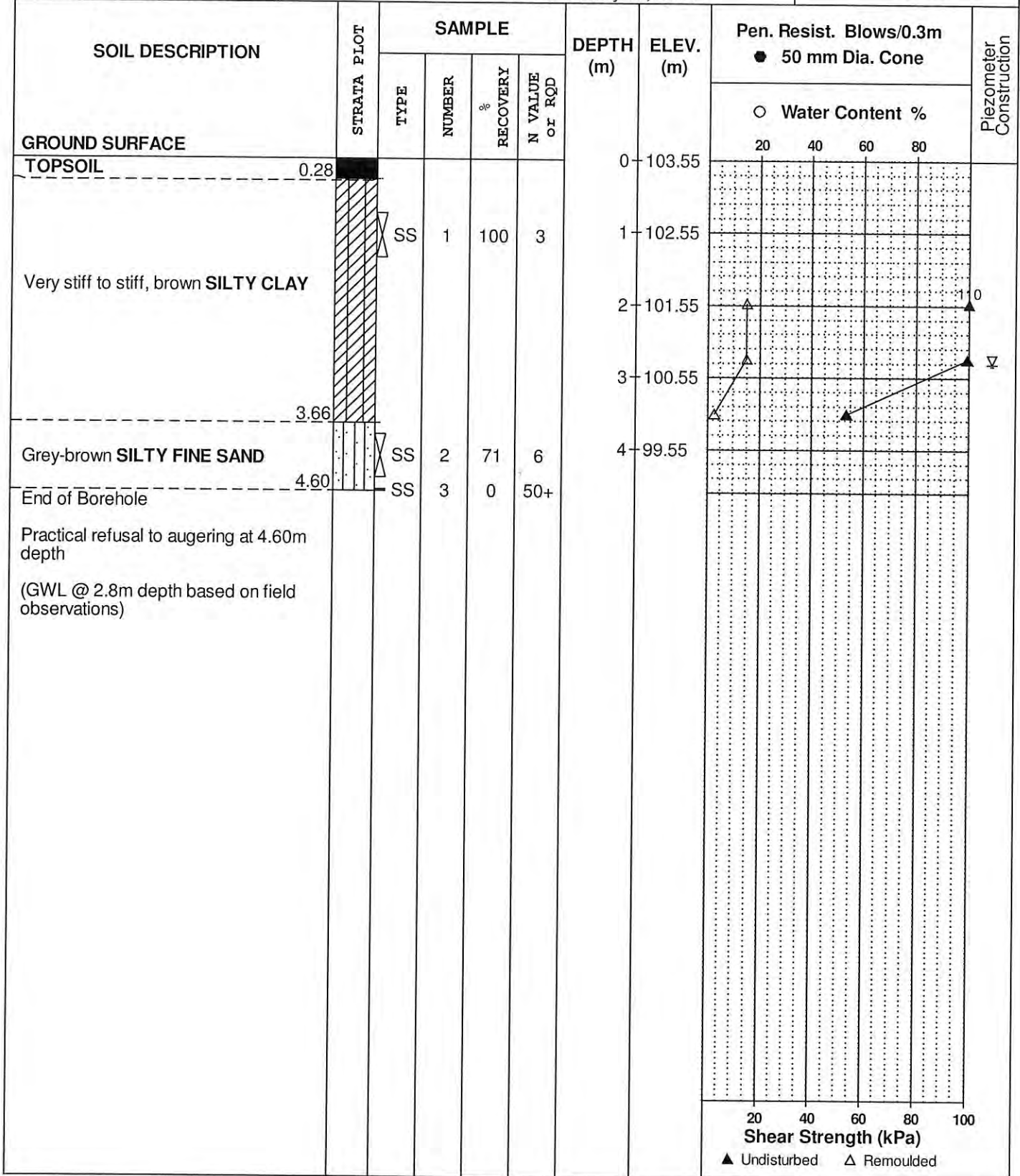
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 13, 2014

FILE NO. PG3115

HOLE NO. BH14



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

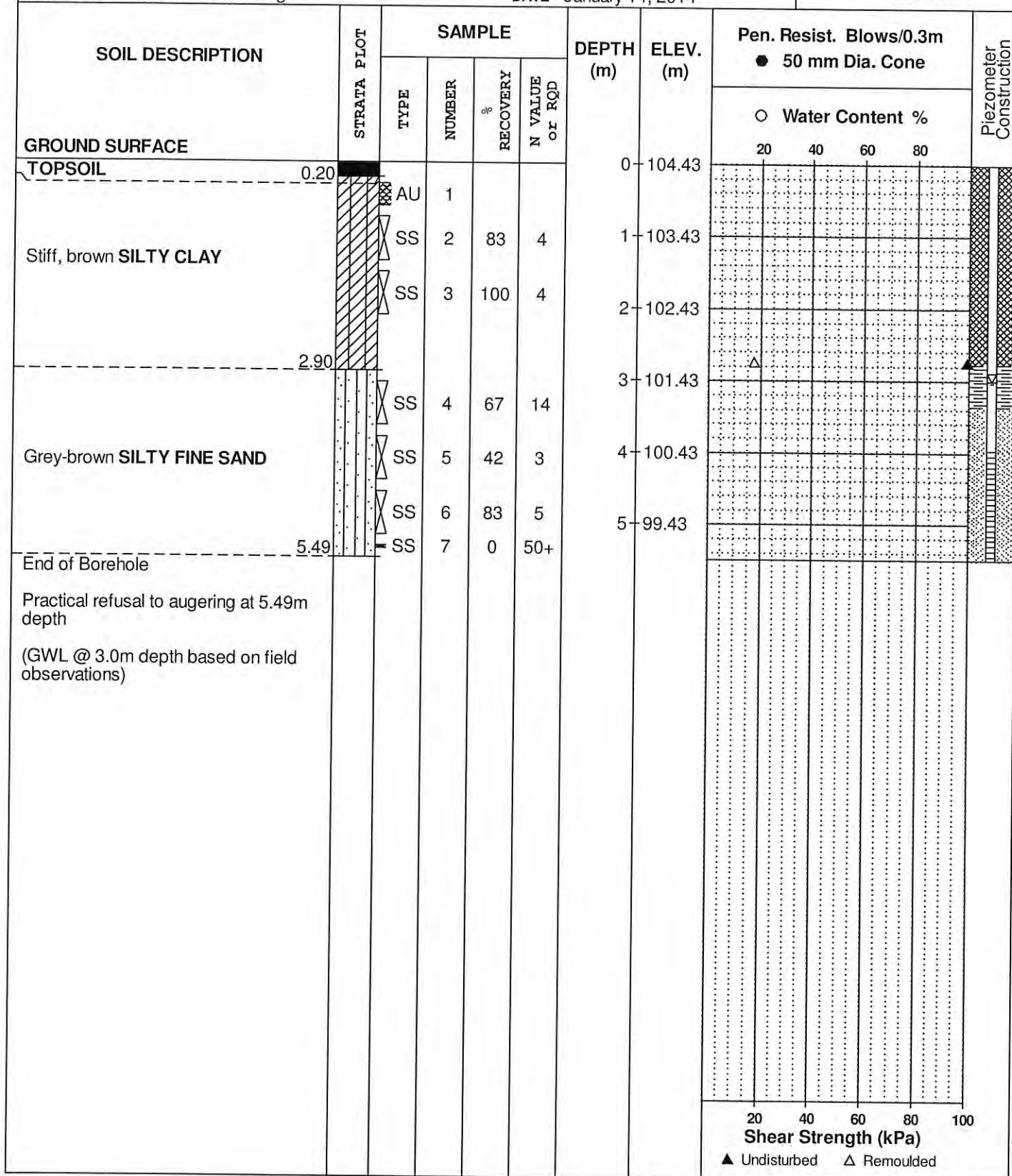
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

FILE NO. **PG3115**

HOLE NO. **BH15**



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

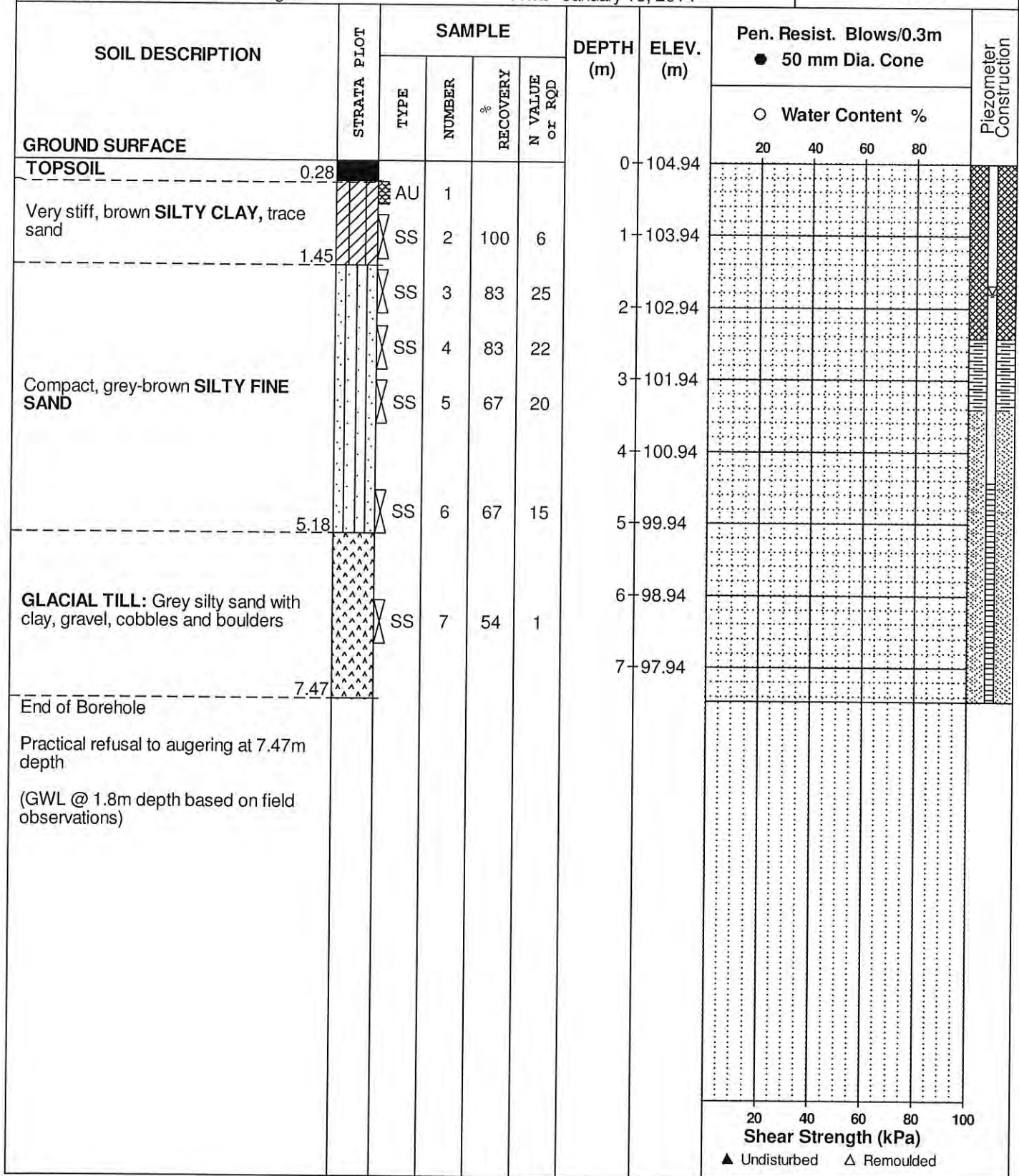
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

FILE NO. PG3115

HOLE NO. BH16



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

FILE NO. **PG3115**

HOLE NO. **BH17**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
TOPSOIL	0.30					0	105.68					
Loose, grey SILTY FINE SAND		SS	1	83	10	1	104.68					
		SS	2	54	8	2	103.68					
		SS	3	67	2	3	102.68					
		SS	4	62	3	4	101.68					
		SS	5	67	8	5	100.68					
		SS	6	67	5	6	99.68					
		SS	7	54	2							
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders	6.00	SS	8	50	3	6	99.68					
End of Borehole	6.70											
(GWL @ 2.0m depth based on field observations)												
								20 40 60 80 100				
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

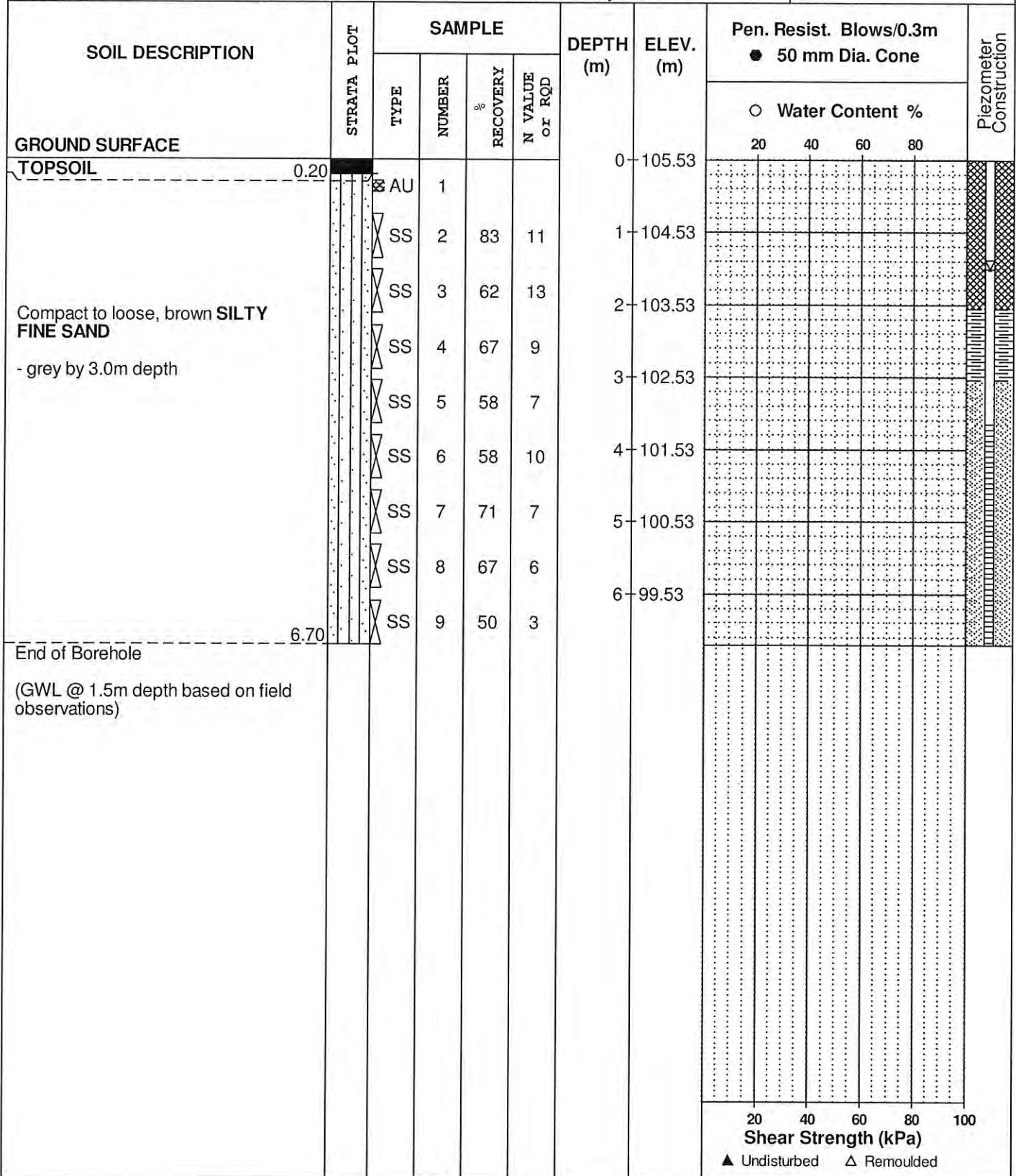
FILE NO. **PG3115**

REMARKS

HOLE NO. **BH18**

BORINGS BY CME 55 Power Auger

DATE January 15, 2014



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

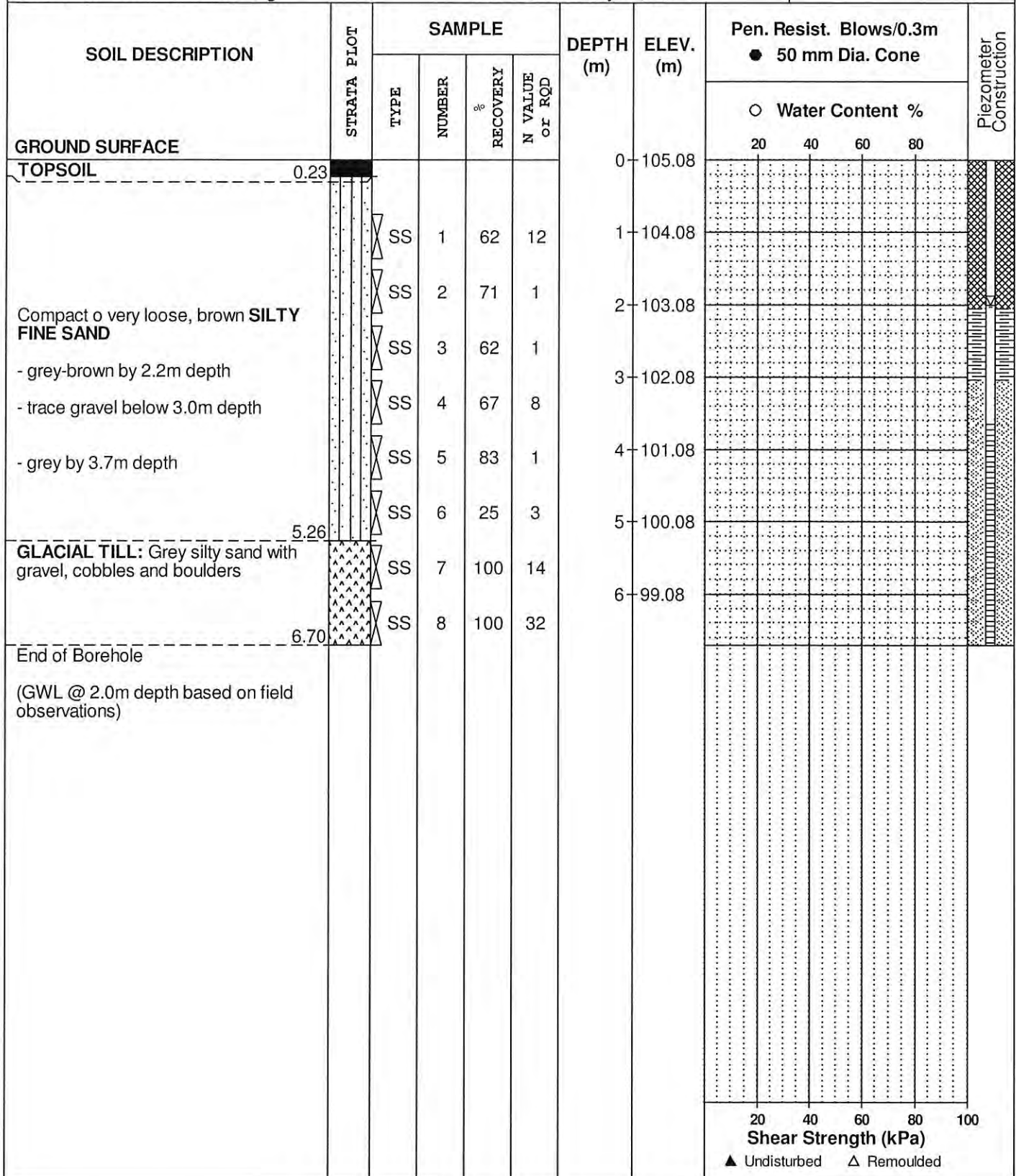
FILE NO. **PG3115**

REMARKS

HOLE NO. **BH19**

BORINGS BY CME 55 Power Auger

DATE January 15, 2014



**Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario**

[illegible]

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

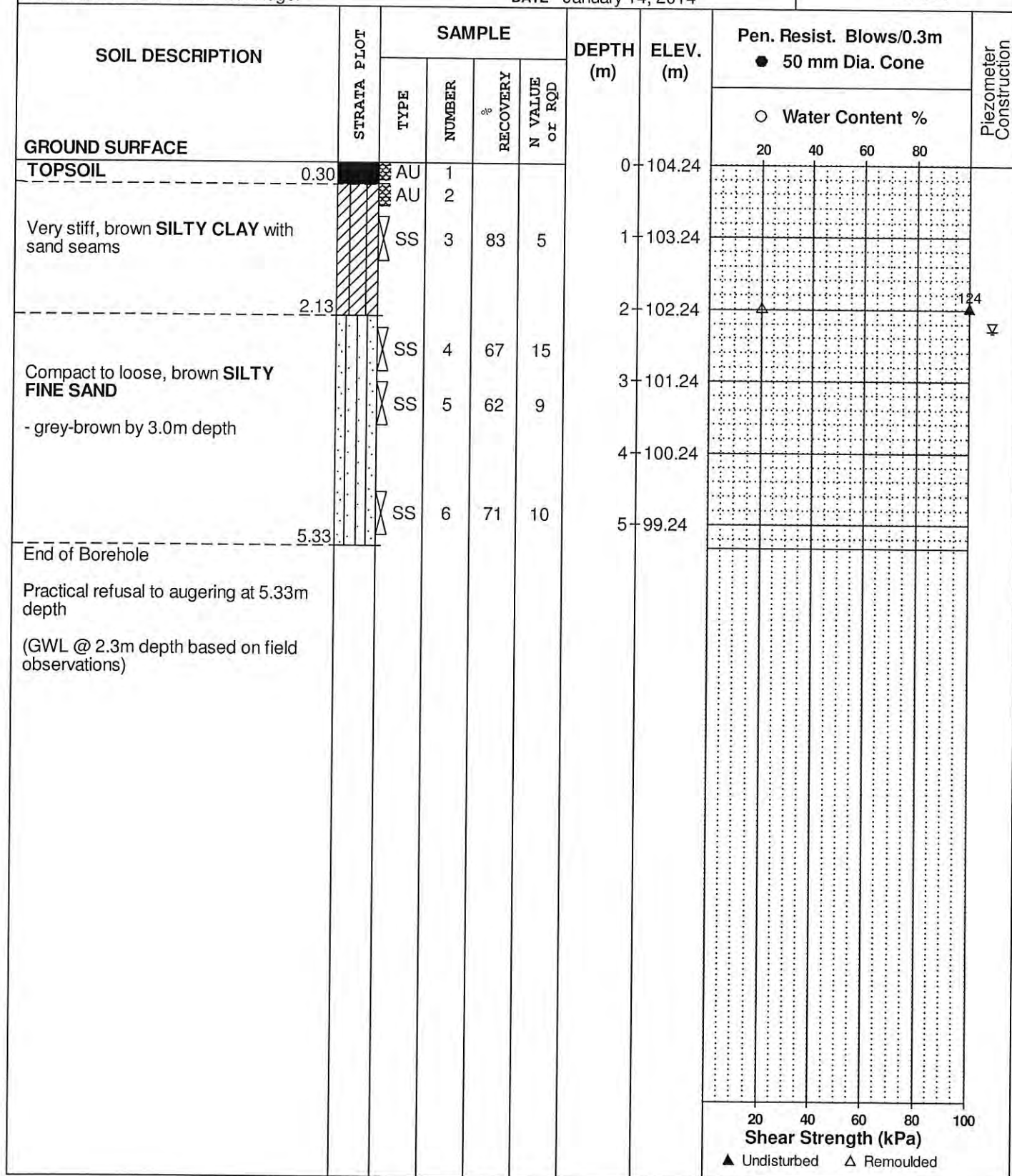
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

FILE NO. PG3115

HOLE NO. BH21



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

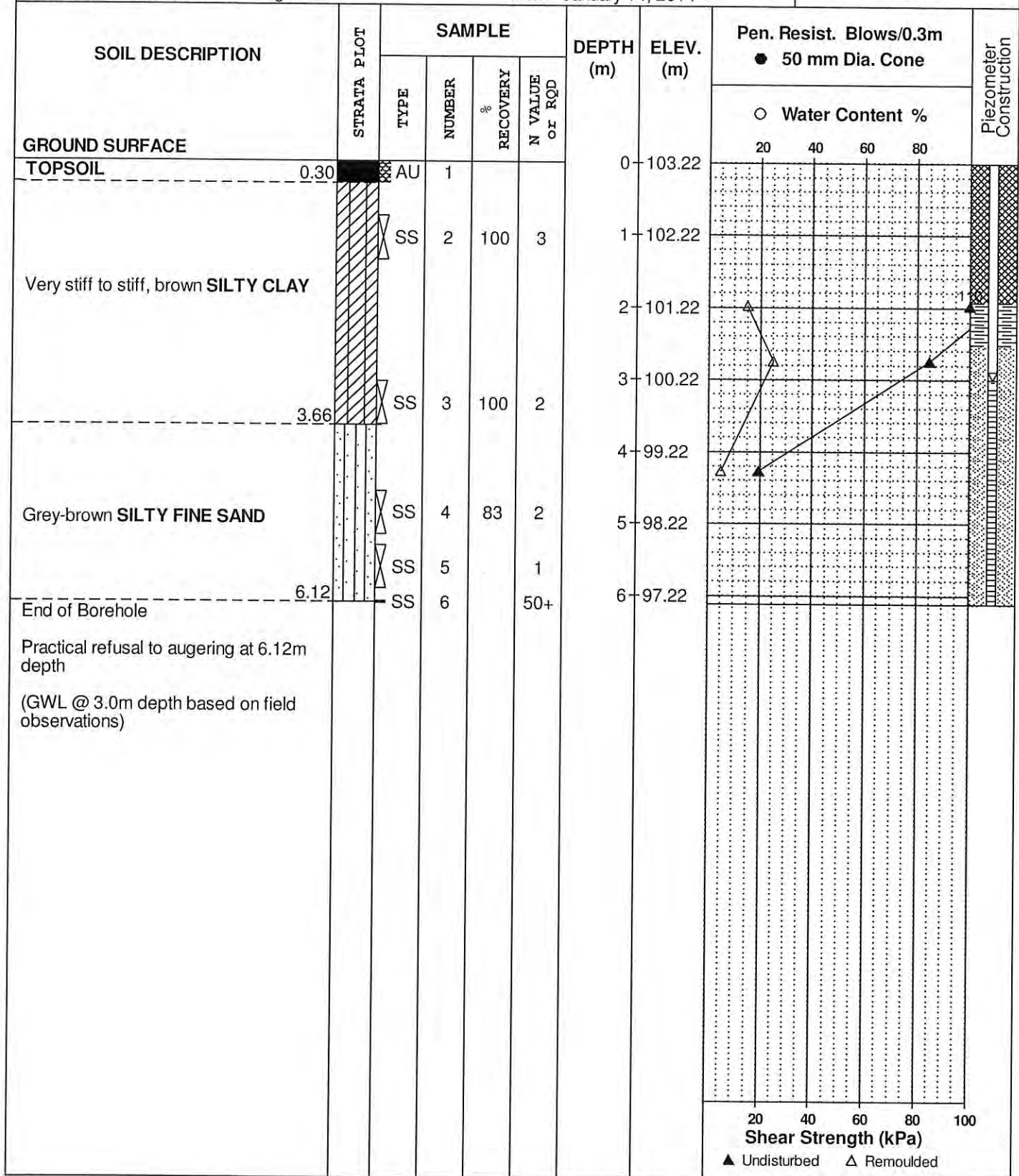
REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 14, 2014

HOLE NO. **BH22**



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

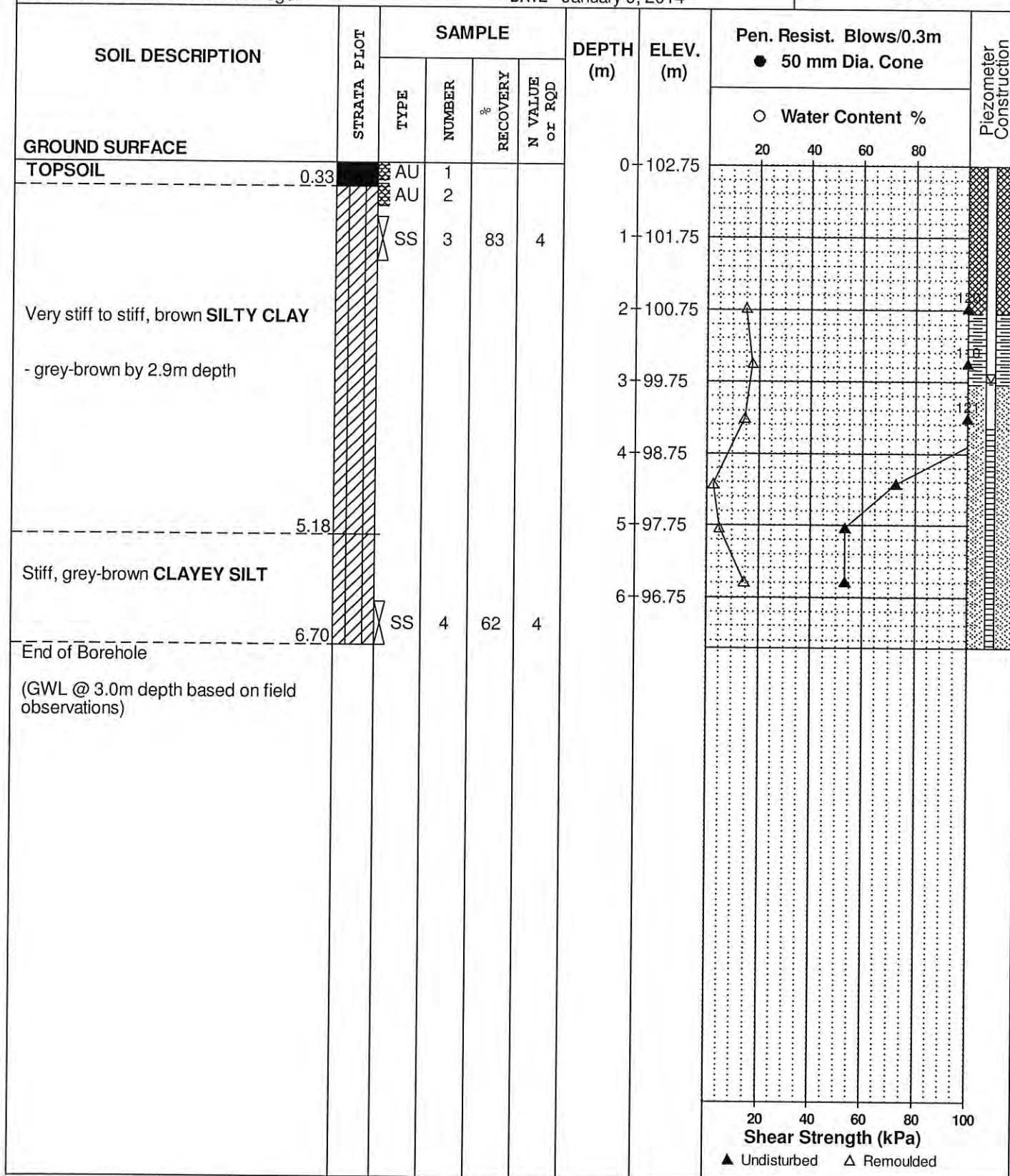
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 9, 2014

FILE NO. PG3115

HOLE NO. BH23



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

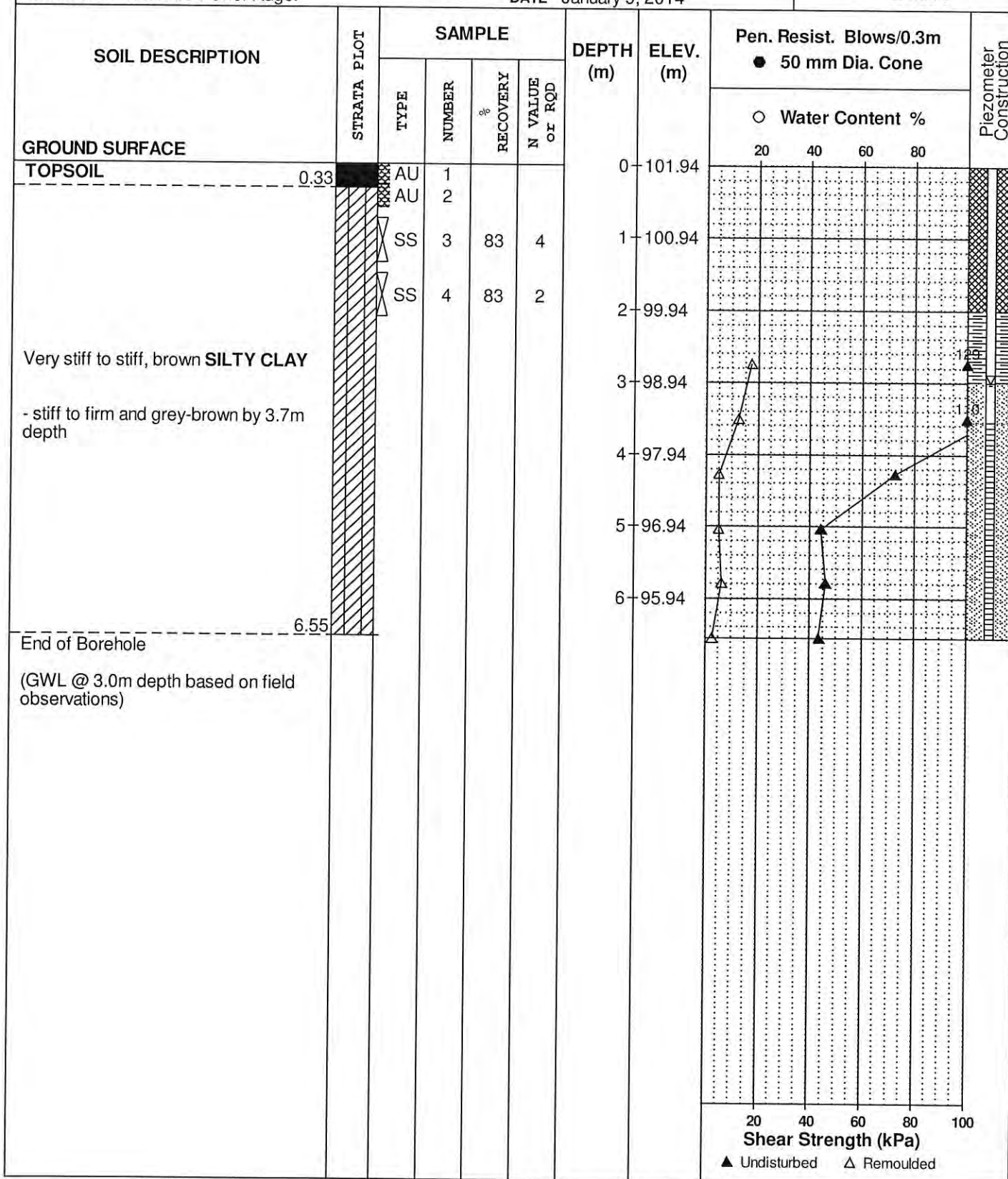
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 9, 2014

FILE NO. PG3115

HOLE NO. BH24



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

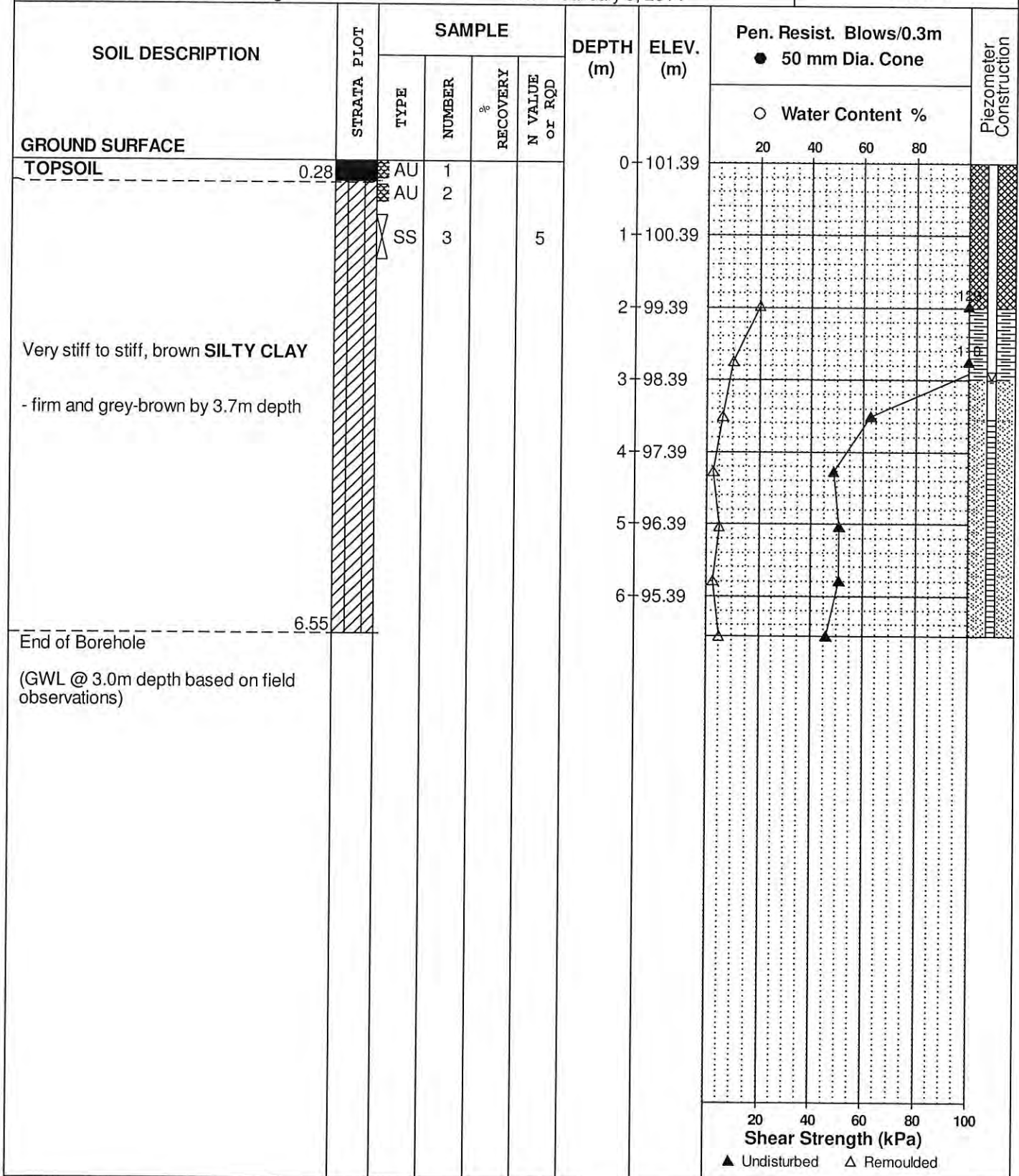
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 9, 2014

FILE NO. PG3115

HOLE NO. BH25



DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

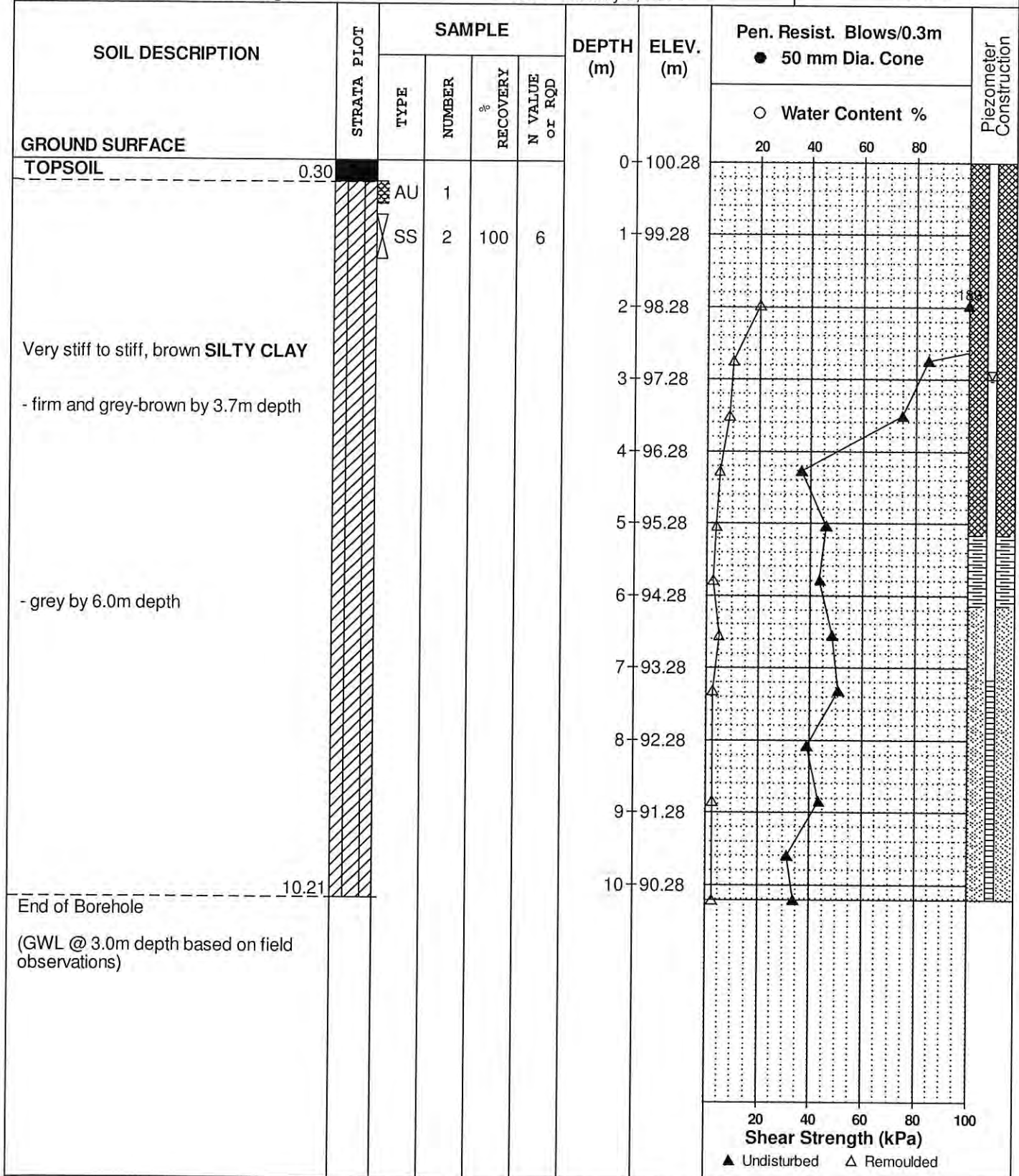
REMARKS

FILE NO. **PG3115**

BORINGS BY CME 55 Power Auger

DATE January 9, 2014

HOLE NO. **BH26**



SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
TOPSOIL	0.28					0	104.87					
Loose to very loose, red-brown SANDY SILT - grey-brown by 0.9m depth - grey by 1.4m depth		SS	1	58	5	1	103.87					
		SS	2	50	6	2	102.87					
		SS	3	58	3	3	101.87					
		SS	4	50	5	4	100.87					
		SS	5	42	3	5	99.87					
		SS	6	33	2	6	98.87					
	5.49					5	99.87					
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders		SS	7	50	21	6	98.87					
	7.01					7	97.87					
End of Borehole												
Practical refusal to augering @ 7.01m depth (GWL @ 0.90m-Dec. 22/10)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

[illegible]

DATUM Ground surface elevations provided by Stantec Geomatics

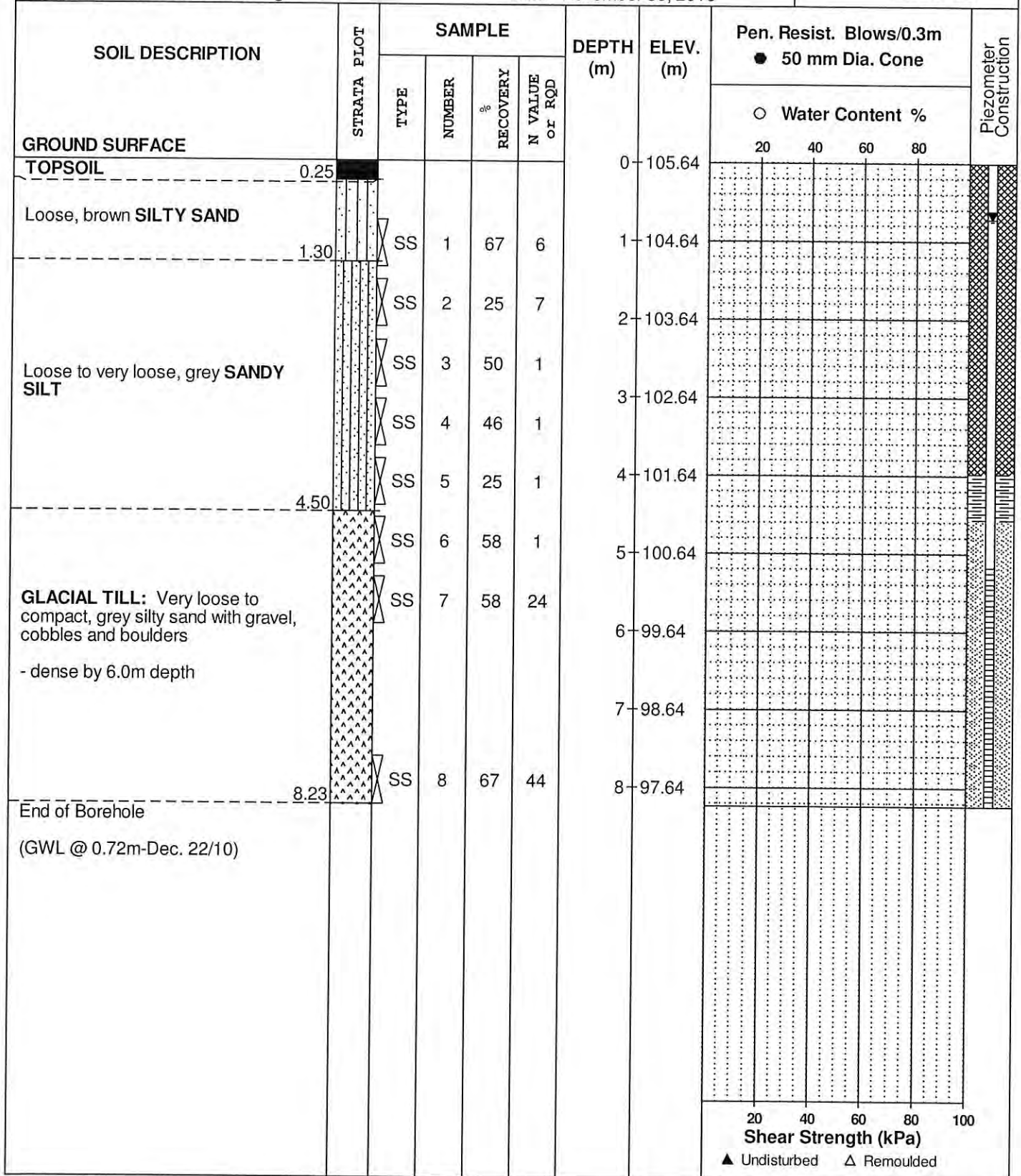
REMARKS

FILE NO. **PG0912**

BORINGS BY CME 55 Power Auger

DATE November 30, 2010

HOLE NO. **BH 3-10**



DATUM Ground surface elevations provided by Stantec Geomatics

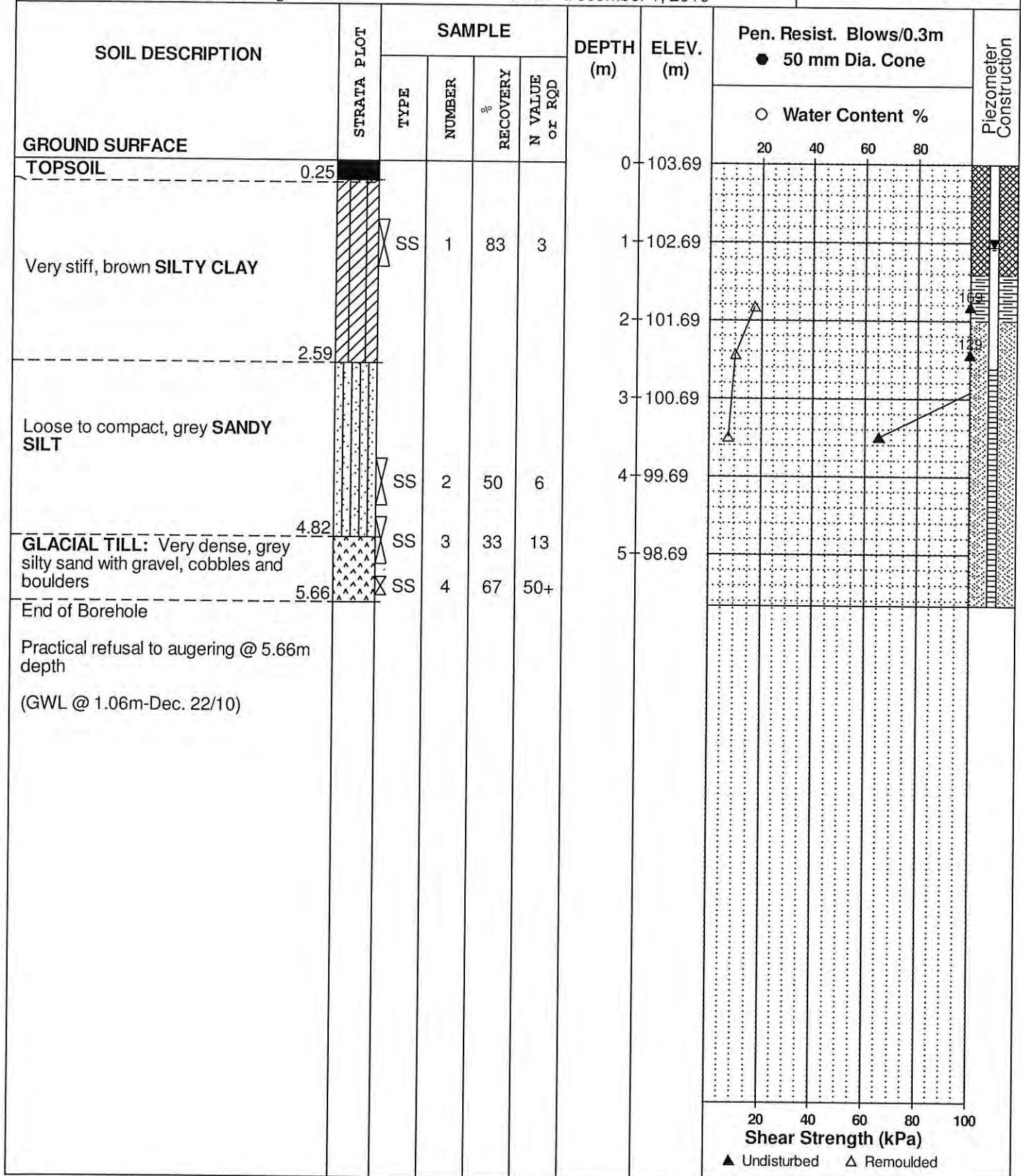
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 1, 2010

FILE NO. **PG0912**

HOLE NO. **BH 4-10**



SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
TOPSOIL	0.28					0	104.01					
Stiff, brown SILTY CLAY	0.97											
Brown CLAYEY SILT to SANDY SILT	1.45	SS	1	83	9	1	103.01					
		SS	2	75	12	2	102.01					
Loose to very loose SANDY SILT		SS	3	92	8	3	101.01					
- very dense by 3.7m depth		SS	4	50	1	4	100.01					
		SS	5	67	50+							
End of Borehole	4.17											
Practical refusal to augering @ 4.17m depth												
(GWL @ 1.51m-Dec. 22/10)												

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

**Terrace Lands - Highway 417 at Huntmar Road
Ottawa, Ontario**

DATUM

REMARKS

BORINGS BY Backhoe

DATE Oct 19, 06

FILE NO.

PG0912

HOLE NO.

TP 1

[illegible]

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

**Terrace Lands - Highway 417 at Huntmar Road
Ottawa, Ontario**

DATUM

REMARKS

BORINGS BY Backhoe

DATE Oct 19, 06

FILE NO.

PG0912

HOLE NO.

TP 2

[illegible]

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

**Terrace Lands - Highway 417 at Huntmar Road
Ottawa, Ontario**

DATUM

REMARKS

BORINGS BY Backhoe

DATE Oct 19, 06

FILE NO.

PG0912

HOLE NO.

TP 3

[illegible]

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Terrace Lands - Highway 417 at Huntmar Road
Ottawa, Ontario**

FILE NO.

PG0912

HOLE NO.

TP 4

DATE Oct 19, 06

[illegible]

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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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

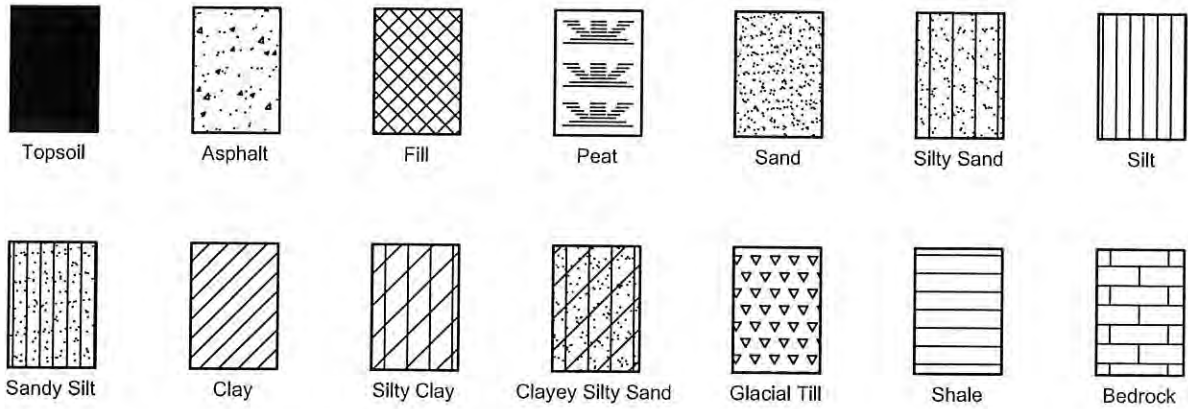
p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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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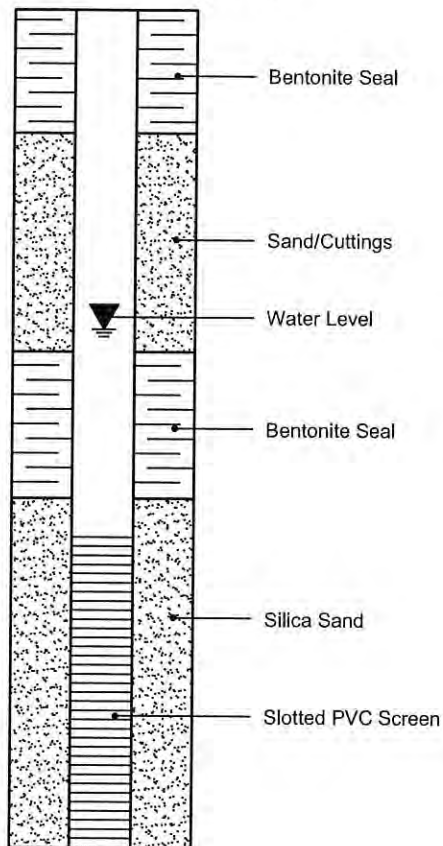
SYMBOLS AND TERMS (continued)

STRATA PLOT

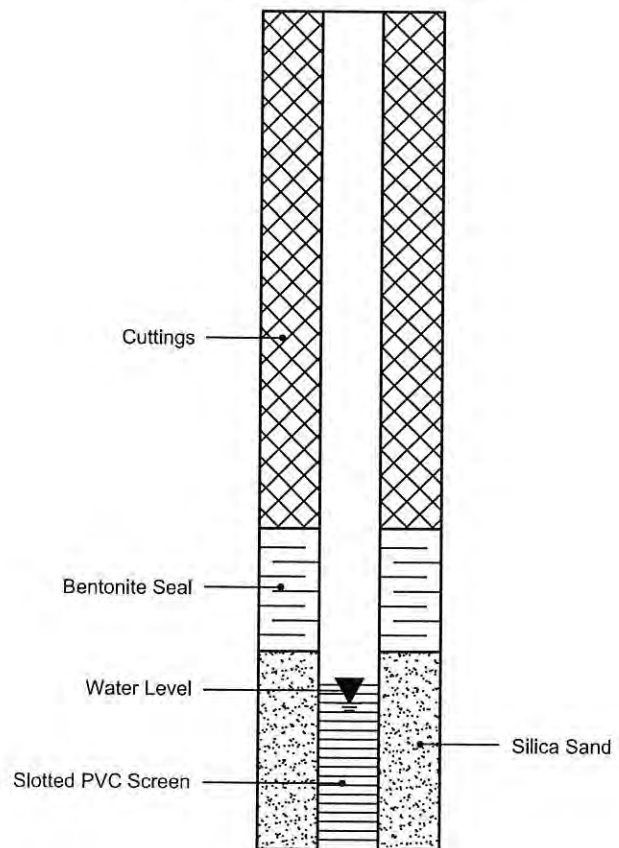


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 3 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG3115-1 - TEST HOLE LOCATION PLAN

DRAWING PG3115-2 - LIMIT OF HAZARD LANDS

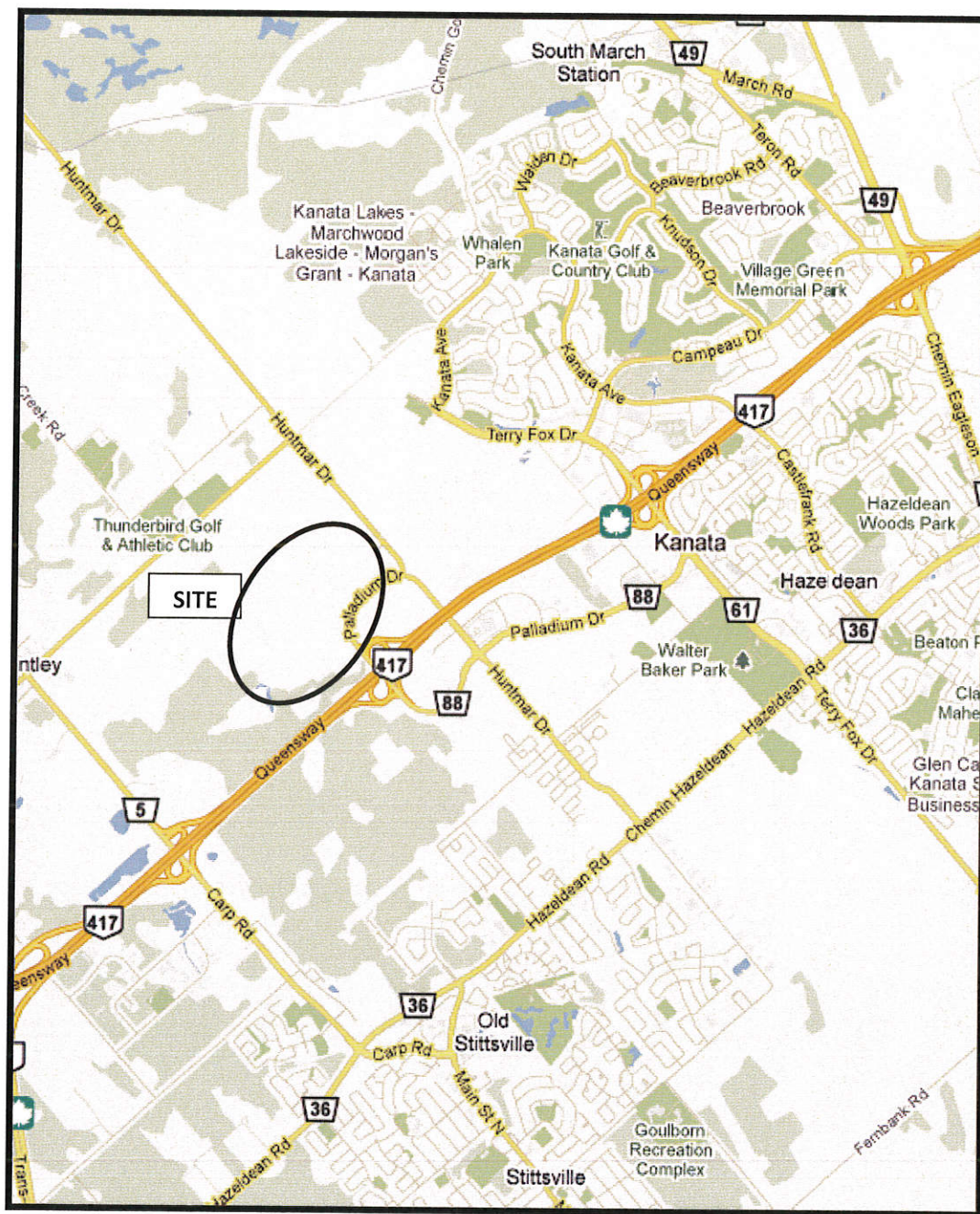


FIGURE 1
KEY PLAN

