

Geotechnical
Engineering

Environmental
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Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Residential Development
5331 Fernbank Road
Ottawa, Ontario

Prepared For

Claridge Homes

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March 5, 2021

Report: PG5683-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed residential development to be located at 5331 Fernbank Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

- ❑ determine the subsurface soil and groundwater conditions by means of boreholes.
- ❑ provide geotechnical recommendations for the foundation design of the proposed buildings and provide geotechnical construction precautions which may affect the design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of several residential townhouse structures. It is anticipated that the proposed residential structures will have a basement level. Access lanes, car parking areas and landscaped areas are also anticipated for the proposed development. It is further anticipated that the site will be serviced by future municipal services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out on February 9, 2021. At that time, 5 boreholes were advanced to a maximum depth of 6.7 m below the existing ground surface. Previous field investigations were completed by Paterson for the subject site in January 2018 and May 2006. A total of 9 boreholes were advanced to a maximum depth of 14.6 m during the previous investigations. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG5683-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track 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 testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, using 73 mm diameter thin walled (TW) Shelby tubes in conjunction with a piston sampler, or from the auger flights. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. The depths at which the auger, split spoon, Shelby tube and grab samples were recovered from the test holes are shown as AU, SS, TW 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 was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 8-18 and BH 2. 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.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

All boreholes were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring. The groundwater level reading were obtained after a suitable stabilization period subsequent to the completion of the field investigation. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples from the current investigation 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 test hole locations were determined by Paterson personnel and surveyed in the field by Paterson. The test hole elevation are referred to the geodetic datum. The locations of the boreholes are presented on Drawing PG5683-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 2 grain size distribution analyses and 5 Atterberg limits tests were completed on selected soil samples from the current investigation. Furthermore, 2 soil samples from the previous investigations were submitted for an Atterberg limits test. The results of our testing are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing and Atterberg Limit's Results sheets presented in Appendix 1.

A total of 4 Shelby tube samples from the previous investigations were submitted for unidimensional consolidation. The results of the consolidation testing are presented on the Unidimensional Consolidation Test Results sheets presented in Appendix 1 and are further discussed in Subsection 5.3.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of undeveloped agricultural land. The subject site is relatively flat and the ground surface elevation is approximately 0.5 m below grade of surrounding roadways. A former roadway with an adjacent approximately 0.5 m deep ditch was noted to traverse the subject site along the east property boundary. The former roadway was observed to consist of a gravel pathway overgrown with light brush. The subject site is bordered to the north by Cope Drive, to the east by a residential subdivision, to the south by Fernbank Road and to the west by Terry Fox Drive.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil profile encountered at the test hole locations consisted of a topsoil layer underlain by interbedded brown silty sand with stiff brown clayey silt to silty clay. Underlying the above noted layers is a deep deposit of firm grey silty clay with some sand.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

A DCPT was completed at BH 2 and BH 8-18. Practical refusal to DCPT was encountered at a depth of 36.7 m below the existing ground surface at BH 2. Practical refusal was not encountered within the upper 30.5 m below existing ground surface at BH 8-18. Based on available geological mapping, the bedrock in the area is part of the Gull River formation, which consists of interbedded limestone and dolomite with an overburden drift thickness ranging between 25 to 50 m.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing was completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results					
Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1-21	0.8 - 1.4	24	20	4	ML
BH 2-21	1.5 - 2.3	25	21	4	ML
BH 3-21	1.5 - 2.3	26	23	3	ML
BH 4-21	1.5 - 2.3	22	19	3	ML
BH 5-21	1.5 - 2.3	28	25	3	ML
BH 2	4.6 - 5.2	46	22	24	CL
BH 2	12.2 - 12.8	39	20	19	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
 ML: Inorganic Silts of Low Plasticity; CL: Inorganic Clay of Low Plasticity

The results of the shrinkage limit testing indicate a shrinkage limit of 6.8% and a shrinkage ratio of 2.21.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on two (2) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis					
Test Hole	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-21	1.5 - 2.3	0	4.5	76.8	18.7
BH 4-21	0.8 - 1.4	0	13.5	72.5	14

4.3 Groundwater

Groundwater level readings were recorded on February 19, 2021. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater level can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between 1.5 to 2.5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels 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 residential development. It is expected that the proposed buildings will be founded over conventional shallow footings placed over an undisturbed silty sand or firm silty clay bearing surface.

Due to the presence of the sensitive silty clay deposit, the proposed development will be subjected to grade raise restrictions. Permissible grade raise recommendations have been designed for the subject site. The recommended permissible grade raise areas are presented in Subsection 5.3. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb subgrade soils during site preparation activities.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II or approved alternative. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the 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 at least compacted by the tracks of the spreading equipment to minimize voids.

Where non-specified imported fill material is considered to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 95% of the material's SPMDD. The non-specified fill should be compacted using a suitably sized vibratory roller, while clay-dominant soils should be compacted using a suitably sized sheepfoot roller. Importing and placement of non-specified fill should be reviewed and approved by the geotechnical consultant at the time of construction.

All non-specified engineered fill should be placed **under dry conditions and above freezing temperatures** and should be approved by Paterson at the time of placement. It is further recommended that all soil, stone and other fill particles with a diameter greater than 300 mm in their longest dimension be segregated from the fill prior to placement of the fill to minimize voids created by their presence in the fill layers.

Non-specified existing fill and 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.

In-Filling Existing Ditches

In-filling the site's existing ditches should be completed in a stepped fashion within the lateral support zone of the proposed buildings. The fill should consist of clean imported granular fill, such as OPSS Granular A or Granular B Type II material. The steps should have a minimum horizontal length of 1.5 m and minimum vertical height of 0.5 m and should be compacted using suitable compaction equipment to a minimum of 98% of the material's SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, compact silty sand and stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Any loose or poor performing silty sand material should be proof-rolled and approved by the geotechnical consultant prior to placement of the footings.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, firm grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **70 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **100 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a firm 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.

Permissible Grade Raise Recommendations

Undrained shear strength testing was completed using a vane apparatus at each borehole location. In addition to the shear strength testing, undisturbed silty clay samples were collected using 73 mm diameter thin walled (TW) Shelby tube in conjunction with a piston sampler. The Shelby tube sample was sealed at both ends and transported to our laboratory for unidimensional consolidation testing.

The value for p'_c is the preconsolidation pressure and p'_o is the effective overburden pressure of the test sample. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values for C_{cr} and C_c are the recompression and compression indices, respectively. These soil parameters are a measure of the compressibility due to stress increases below and above the preconsolidation pressures. The higher values for the C_c , as compared to the C_{cr} , illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

Four (4) site specific consolidation tests were conducted as part of the previous investigation. The results of the consolidation tests from our investigation is presented in Table 3 and in Appendix 1.

Table 3 - Summary of Consolidation Test Results							
Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 3-18	TW 3	4.04	111	42	0.013	0.898	A
BH 7-18	TW 3	4.83	78	41	0.021	1.190	A
BH 2	TW 5	5.00	115	52	0.011	0.984	A
BH 2	TW 9	12.60	163	114	0.017	1.319	P
* - Q - Quality assessment of sample - G: Good A: Acceptable P: Likely disturbed							

It should be noted that the values of p'_c , p'_o , C_{cr} and C_c are determined using standard engineering practices and are estimates only.

The effective overburden stress, p'_o , is directly influenced by the groundwater level. The effective overburden stresses for the consolidation test samples were estimated using a conservatively low groundwater depth.

It has been considered that the groundwater level will vary seasonally and may be affected by other factors that could reduce groundwater infiltration as part of development (pavements, storm sewers, etc.) or promote groundwater depletion (trees, dry seasons, etc). As such, our analyses considered the post-development long-term groundwater level at a position 0.5 m lower than the assumed long-term level.

Based on our experience with local clay deposits, consolidation testing results and undrained shear strength values at the borehole locations, the **permissible grade raise recommendation for finished grading within 6 m of a building footprint is 1.5 m** and our permissible grade raise restriction for finished grading along access lanes and parking lots is **2.0 m**.

Where proposed grade raises exceed our permissible grade raise recommendations, several options could be considered for the foundation support of the proposed buildings:

Scenario A

Where the grade raise is close to, but below, the maximum permissible grade raise, consideration should be given to using more reinforcement in the design of the foundation (footings and walls) to reduce the risks of cracking in the concrete foundation. The use of control joints within the brick work between the garage and basement area should also be considered.

Scenario B

Where the grade raise cannot be accommodated with soil fill, the following options could be used alone or in combination.

Option 1 - Use of Lightweight Fill

Lightweight fill (LWF) can be used, consisting of EPS (expanded polystyrene) Type 12 or 15 blocks or other light weight materials which allow for raising the grade without adding a significant load to the underlying soils. However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the garage and porch areas to reduce the fill-related loads.

Option 2 - Surcharge Fill Settlement Monitoring Program

Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site. Settlement plates to monitor long term settlement should be installed at selected locations. Once the desired settlements have taken place, the surcharged portion can be removed and the site is considered acceptable for development.

Once available, the final grading plan should be reviewed by Paterson and the above options could be further discussed along with further recommendations on specific requirements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations bearing on a is applicable for design of the proposed buildings bearing over a deep silty clay deposit throughout 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.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, undisturbed native soil surface will be considered acceptable subgrade on which to commence backfilling for floor slab construction. 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. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

A clear crushed stone fill is recommended for backfilling below the floor slab for limited span slab-on-grade areas, such as front porch or garage footprints. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone below basement floor slabs.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of the car only parking areas, heavy truck traffic and access lanes anticipated at this site. The pavement structure noted in Table 4 may be considered suitable for the design of fire truck access lanes.

Table 4 - 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 in situ soils or OPSS Granular B Type I or II material placed over in situ soil.	

Table 5 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in situ soils or OPSS Granular B Type I or II material placed over in situ soil.	

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for the proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. 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 Delta Drain 6000 or an approved equivalent. 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 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 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 excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below 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.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

- Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- Piping from water seepage through granular soils, and
- Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b , is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.

s_u - undrained shear strength of the soil below the base level

σ_z - total overburden and surcharge pressures at the bottom of the excavation

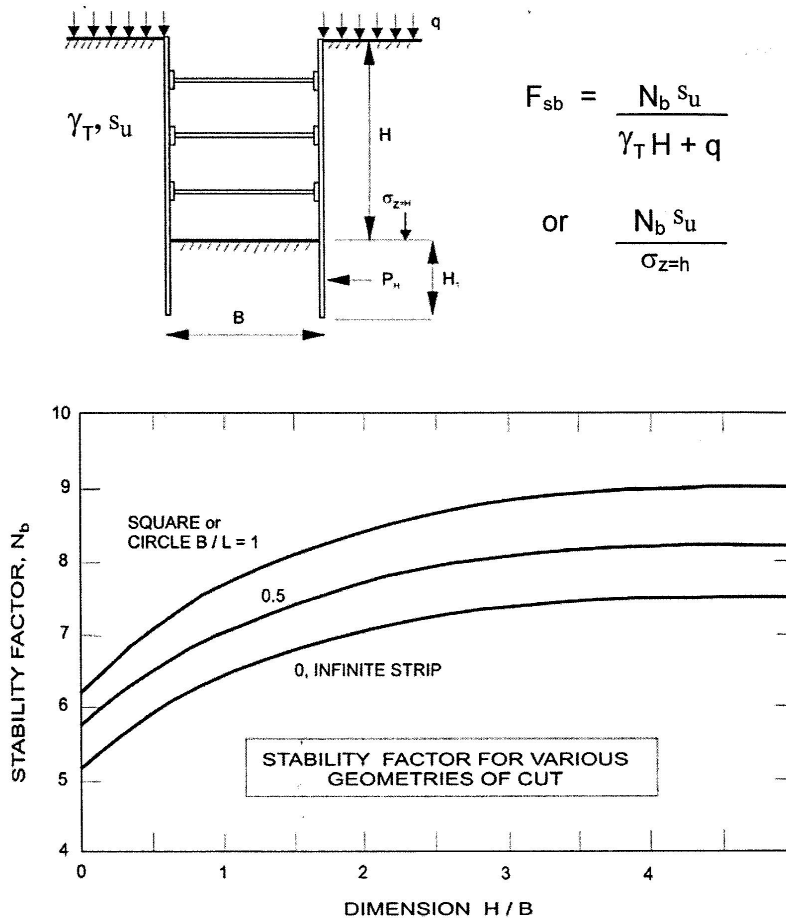


Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

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 pipe bedding for sewer and water pipes. The bedding layer should be increased to a minimum of 300 mm where the subgrade consists of a firm grey silty clay. The bedding material should be placed in maximum 300 mm thick lifts, compacted to a minimum of 95% of its SPMD and extend 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 99% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay and silty clay with sand 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 (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, 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 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. Periodic inspection of the clay seal placement work should be completed by Paterson personnel during servicing installation work.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

Permit to Take Water

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

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

Impacts on Neighboring Structures

Based on the existing groundwater level and low permeability of the adjacent soils, the extent of any significant groundwater lowering will take place within a limited range of the subject site. Based on the proximity of the neighboring residential development and minimal zone impacted by the ground water lowering, the proposed development will not negatively impact the neighboring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

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.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) for trees planted within a public right-of-way (ROW). Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution and hydrometer testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of footing elevation and a 3.5 m depth below finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay and should not be designated as sensitive marine clays.

Low to Medium Sensitivity Clays

A low to medium sensitivity clay soil was encountered between the anticipated design underside of footing elevations and 3.5 m below finished grade as per City Guidelines for the entire site. Based on our Atterberg limits test results, the modified plasticity index does not exceed 40% across the site. **The following tree planting setback is recommended for the entire subject site due to the presence of low to medium sensitivity clays.** Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the conditions noted below are met.

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.
- ❑ A small tree must be provided with a minimum of 25 m³ of available soil volume while a medium tree must be provided with a minimum of 30 m³ of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.
- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).
- ❑ Grading surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

Aboveground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighboring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer's requirements.

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer's specifications.

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

7.0 Recommendations

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

- Grading plan review from a geotechnical perspective, once the final grading plan is 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 request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests to be notified immediately in order to permit reassessment of the 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 Claridge Homes or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Vincent Duquette, EIT



David J. Gilbert, P.Eng.

Report Distribution:

- Claridge Homes (1 digital copy)
- Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNIDIMENSIONAL CONSOLIDATION TEST SHEETS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TEST RESULTS

DATUM Geodetic

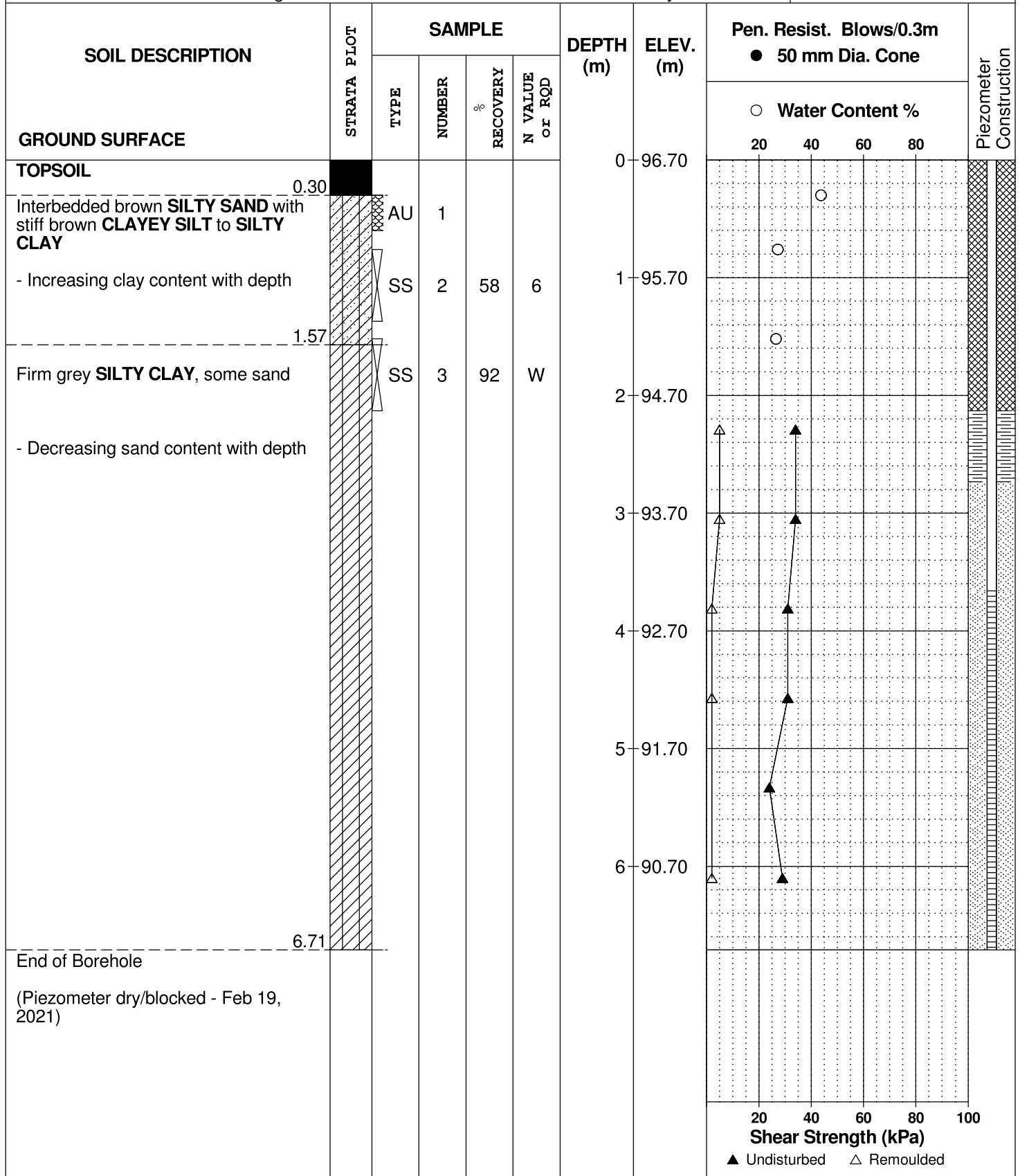
REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 9

FILE NO. **PG5683**

HOLE NO. **BH 1-21**



DATUM Geodetic

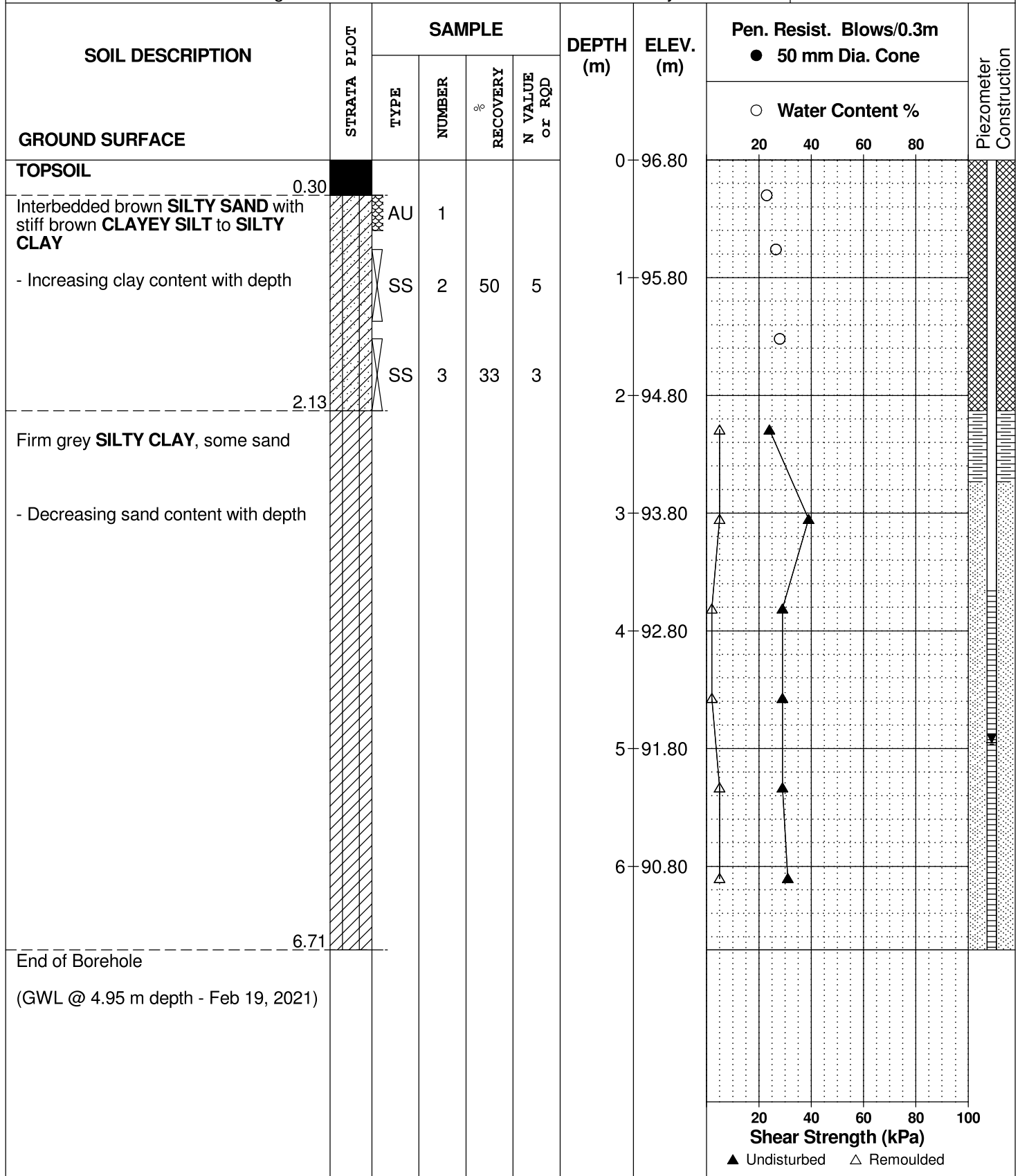
REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 9

FILE NO. **PG5683**

HOLE NO. **BH 2-21**



DATUM Geodetic

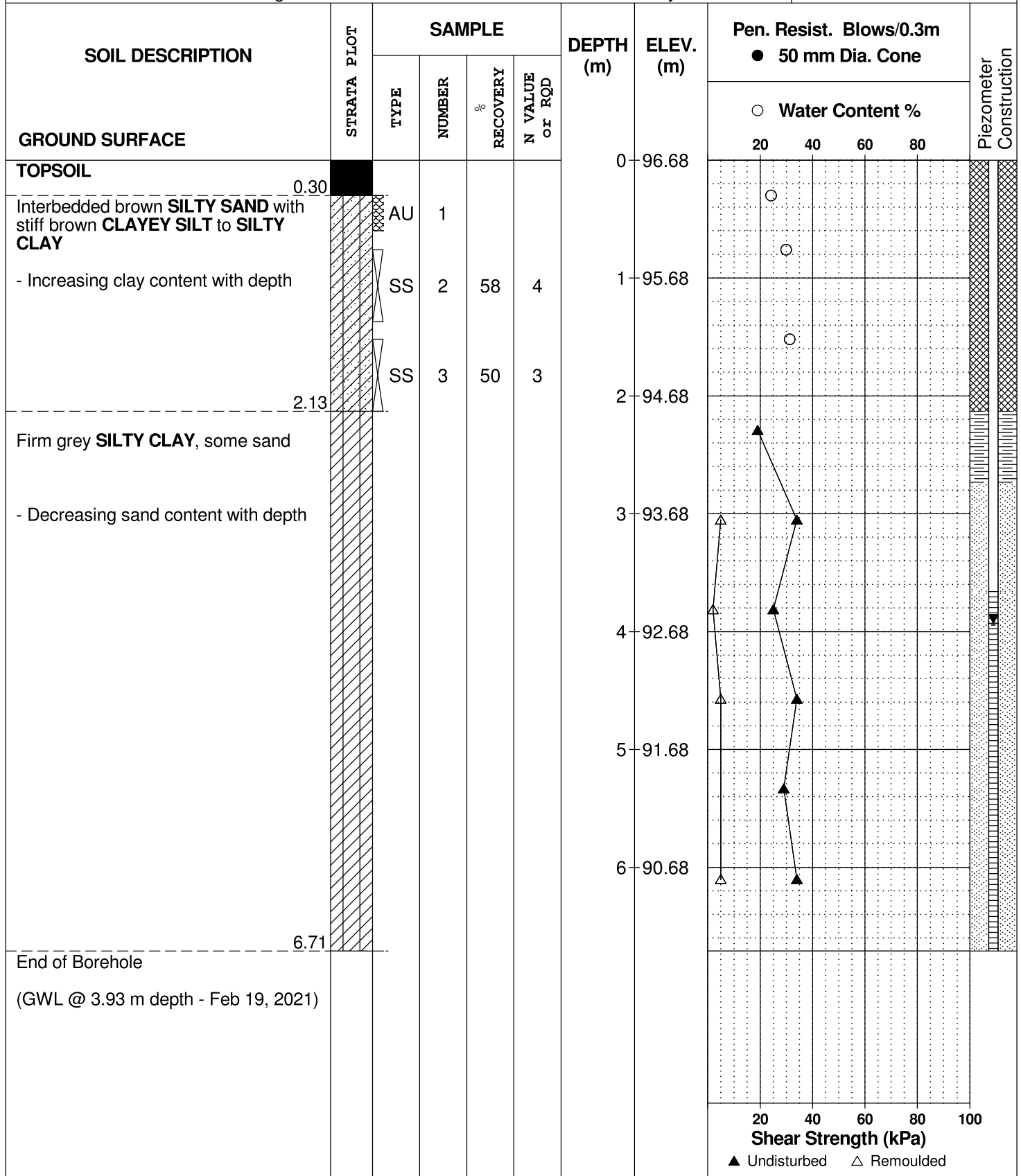
REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 9

FILE NO. **PG5683**

HOLE NO. **BH 3-21**



DATUM Geodetic

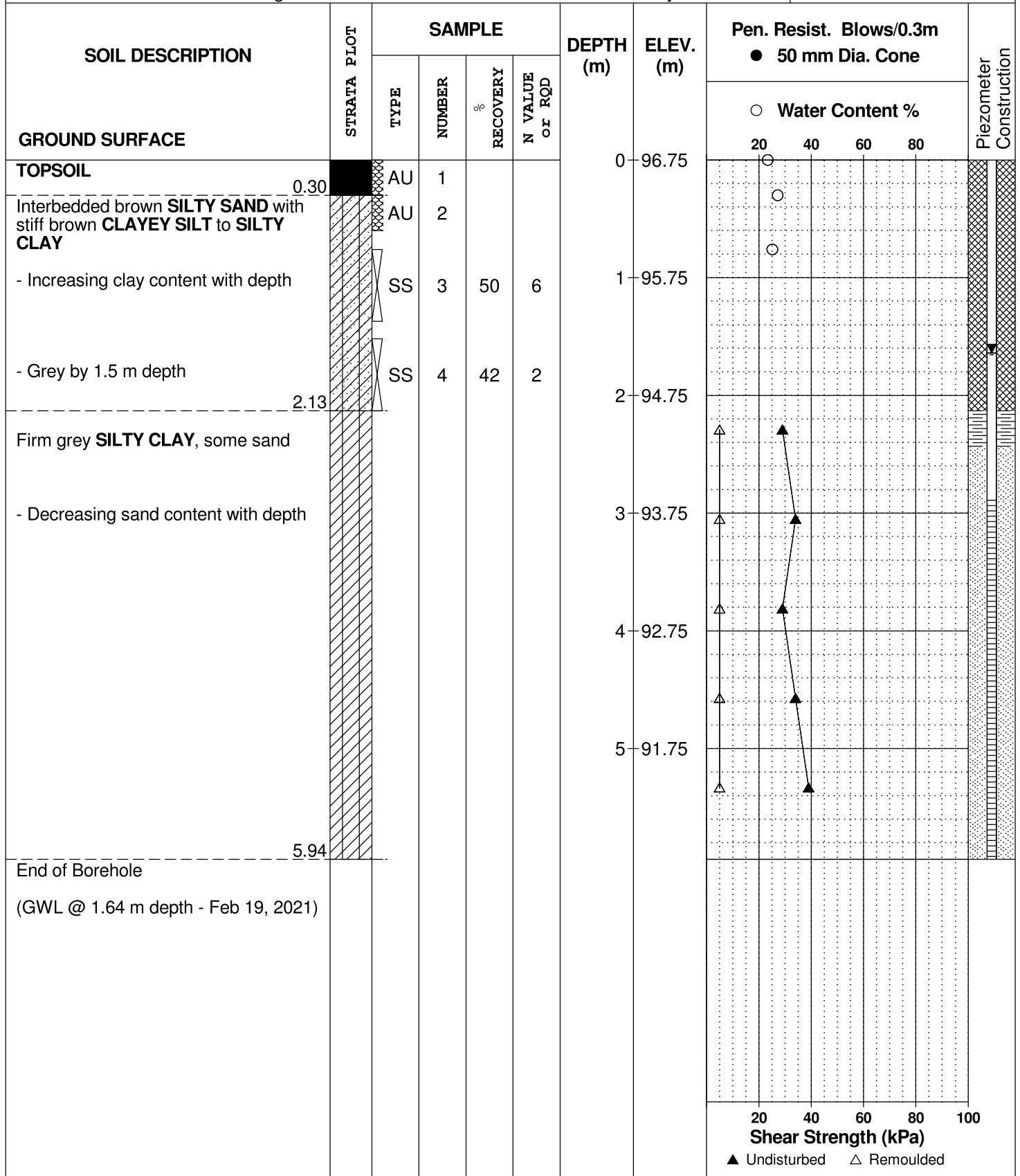
REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 9

FILE NO. **PG5683**

HOLE NO. **BH 4-21**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

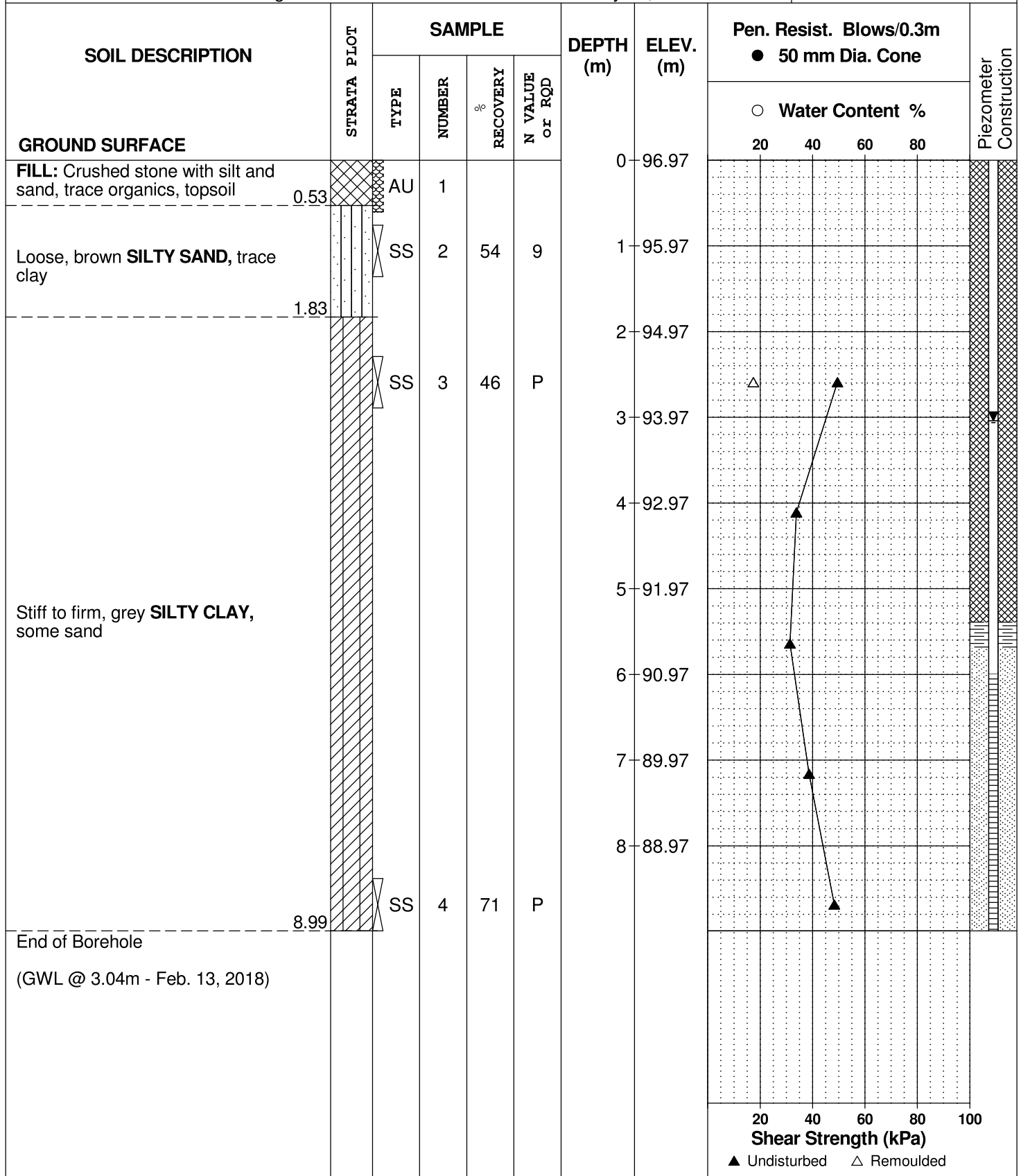
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2018

FILE NO. **PG4411**

HOLE NO. **BH 1-18**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

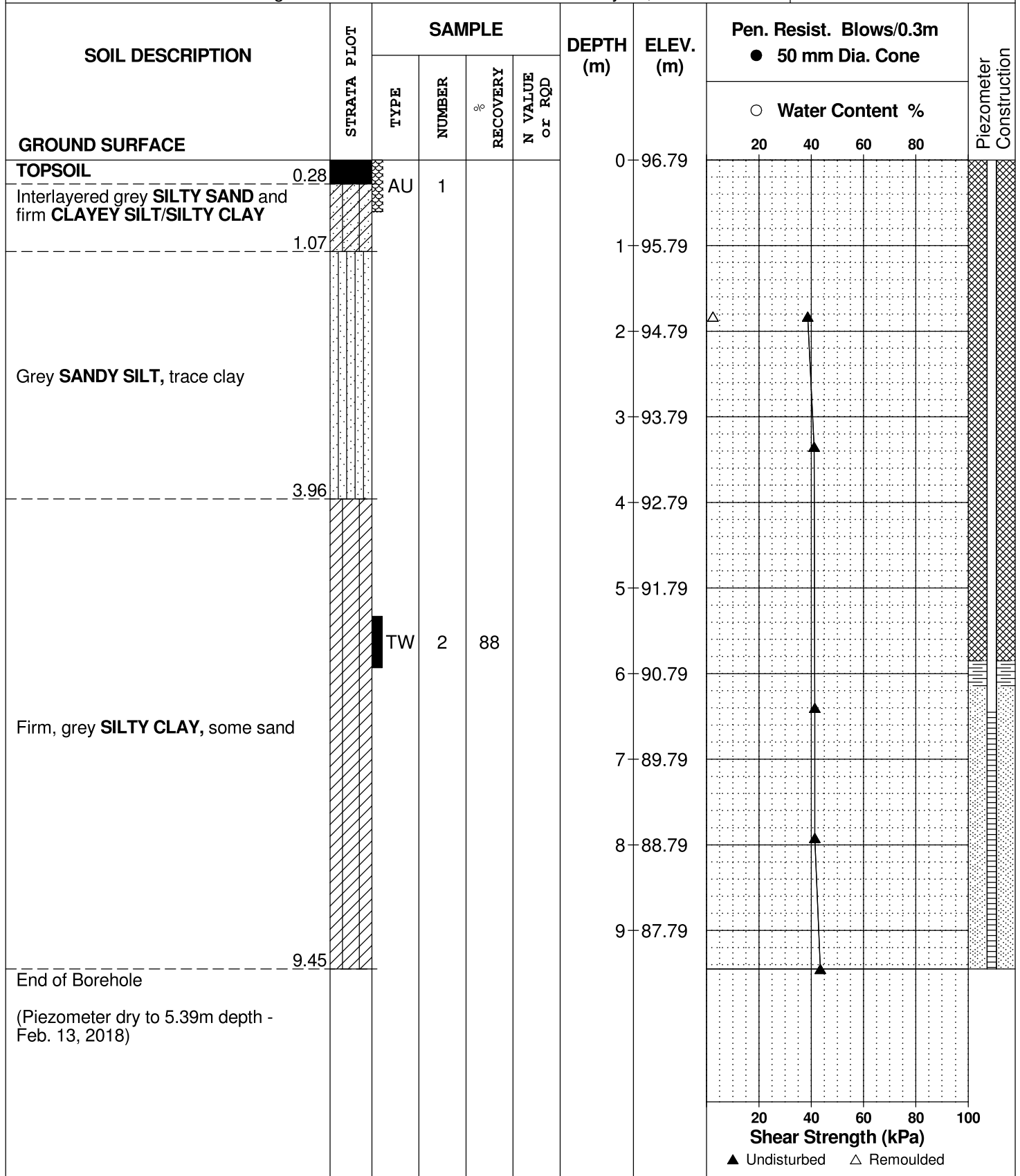
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2018

FILE NO. **PG4411**

HOLE NO. **BH 2-18**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

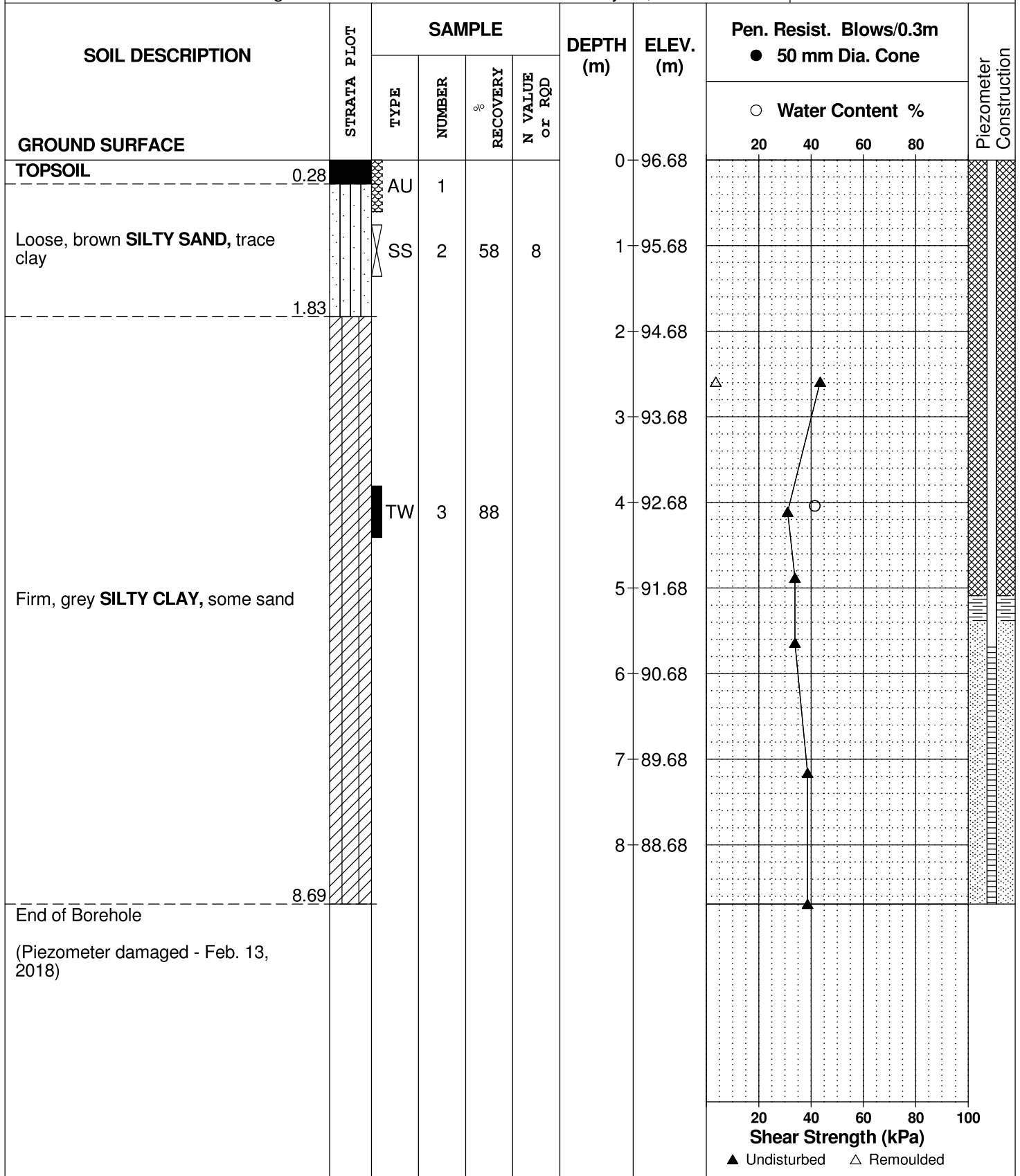
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2018

FILE NO. **PG4411**

HOLE NO. **BH 3-18**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

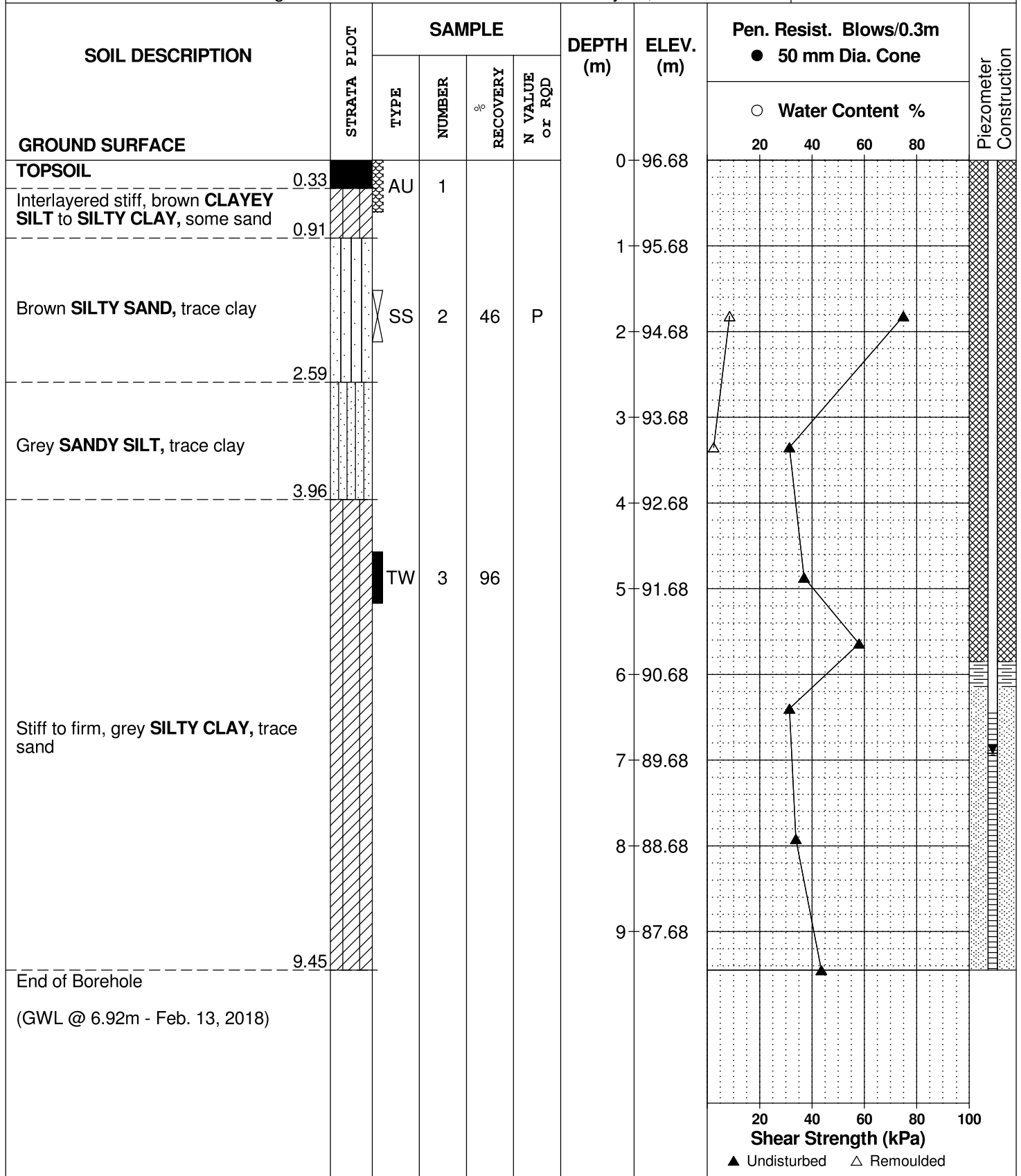
FILE NO. **PG4411**

REMARKS

HOLE NO. **BH 4-18**

BORINGS BY CME 55 Power Auger

DATE January 16, 2018



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

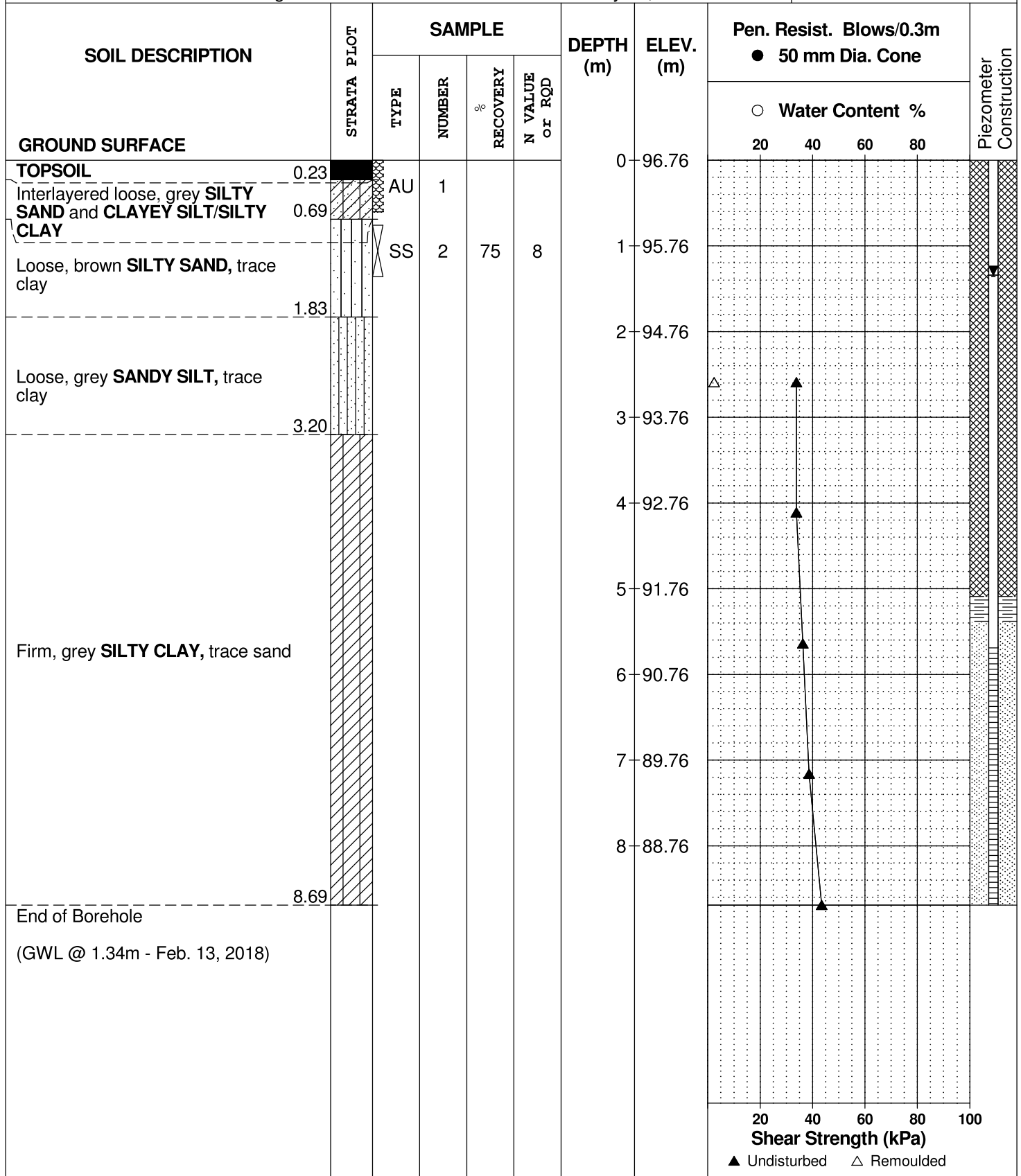
FILE NO. **PG4411**

REMARKS

HOLE NO. **BH 5-18**

BORINGS BY CME 55 Power Auger

DATE January 16, 2018



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

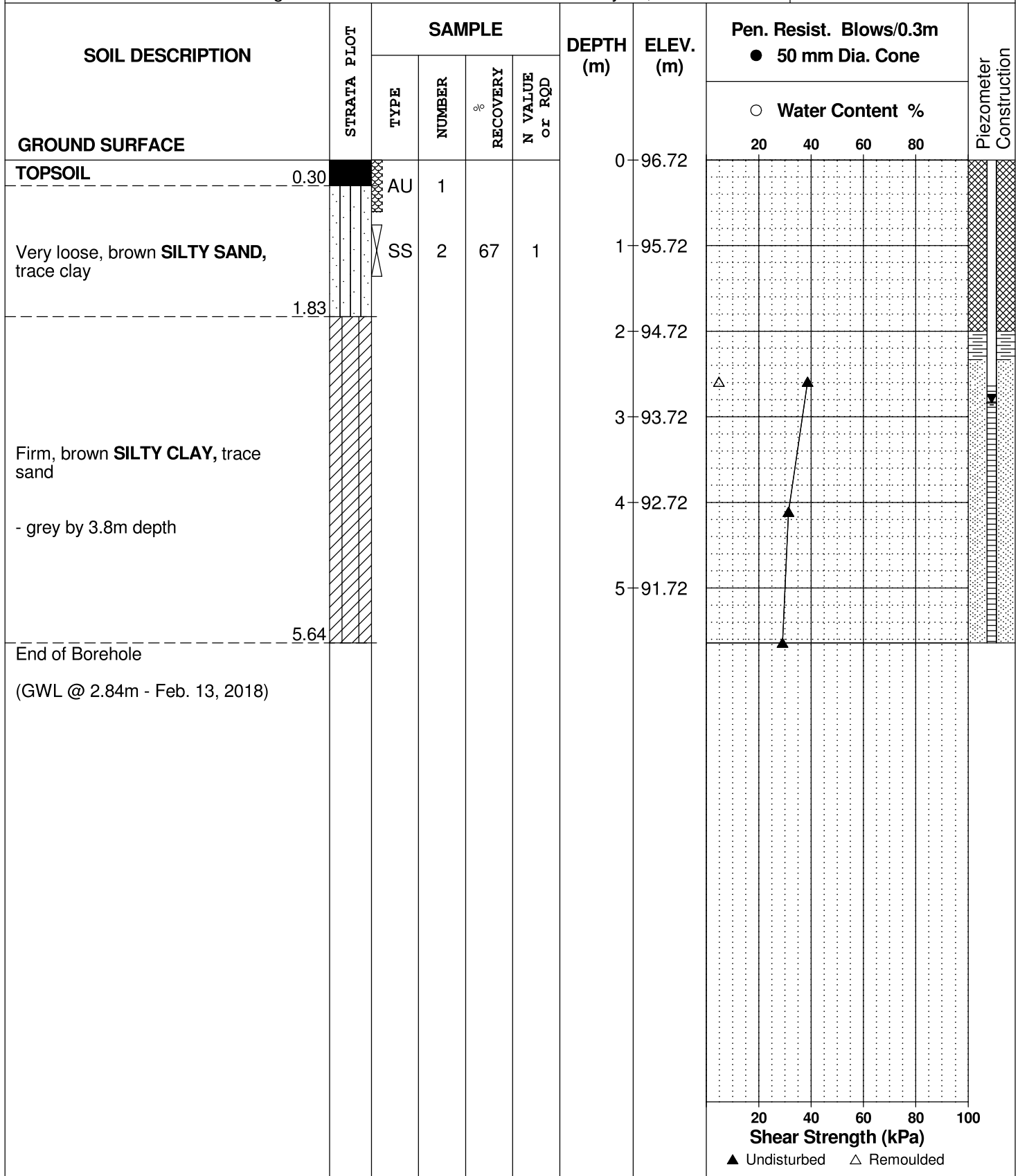
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 16, 2018

FILE NO. **PG4411**

HOLE NO. **BH 6-18**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

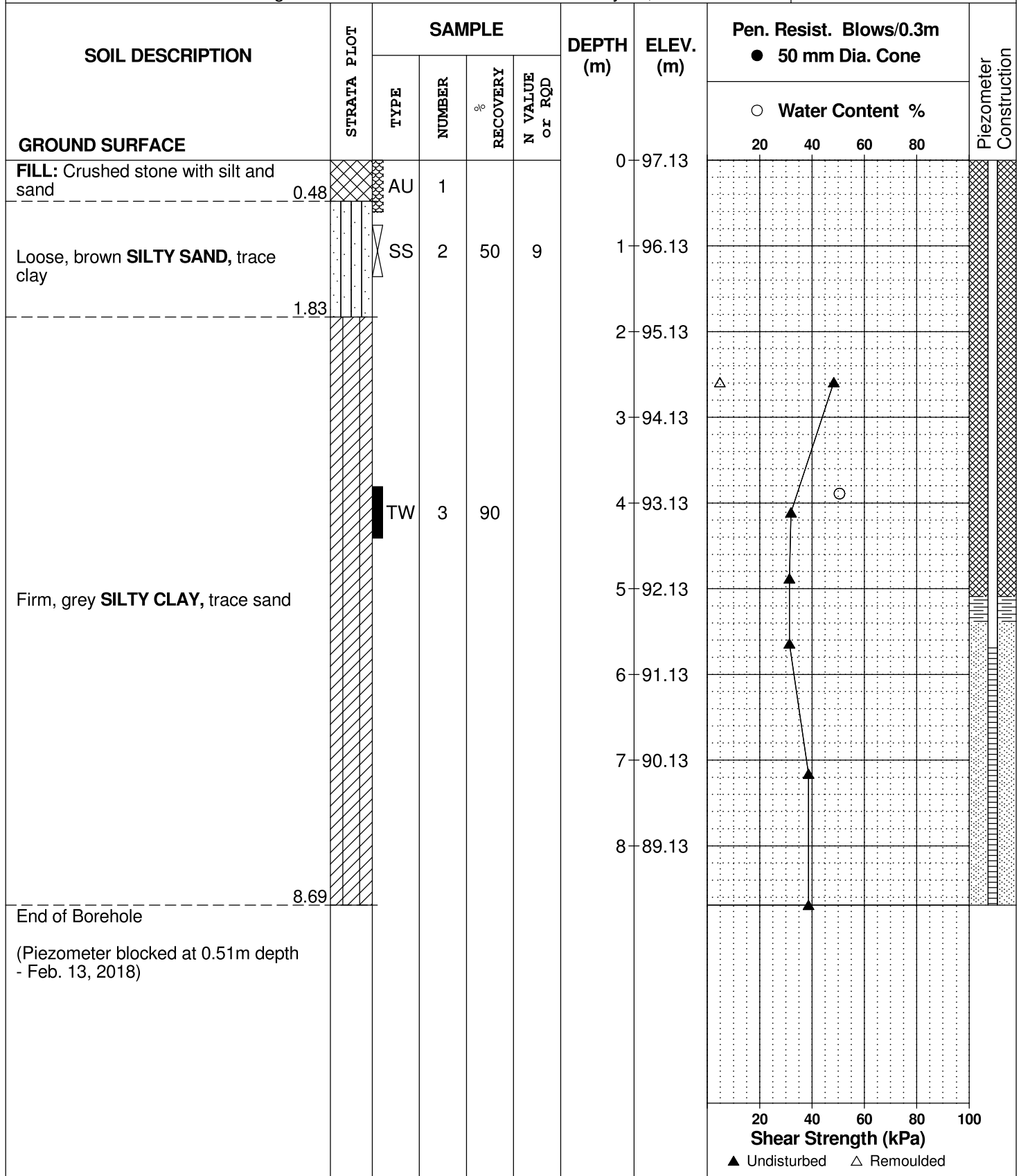
REMARKS

BORINGS BY CME 55 Power Auger

DATE January 16, 2018

FILE NO. **PG4411**

HOLE NO. **BH 7-18**



DATUM Geodetic elevations provided by Stantec Geomatics Limited.

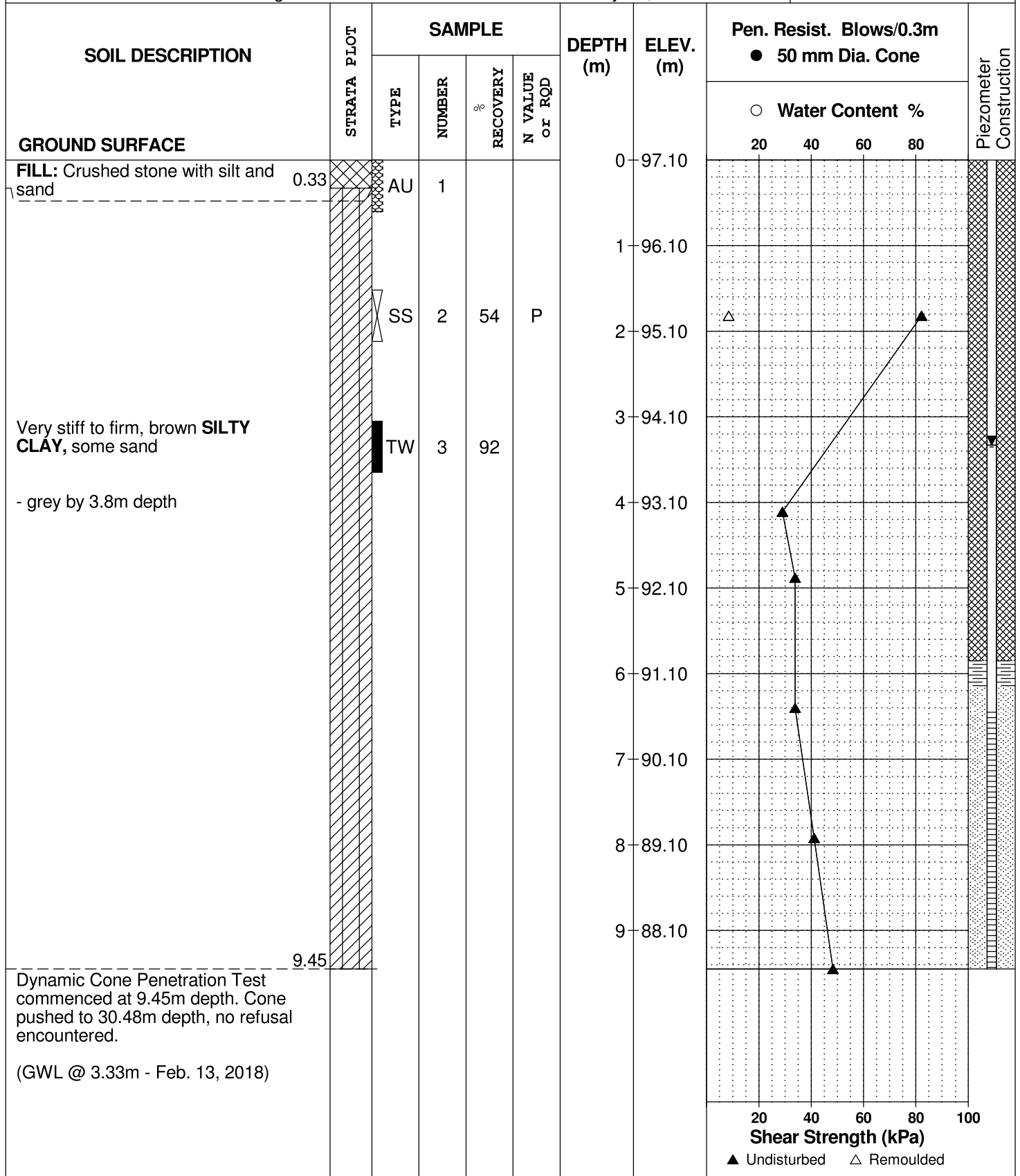
REMARKS

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DATE January 17, 2018

FILE NO. **PG4411**

HOLE NO. **BH 8-18**



DATUM

FILE NO.

PG0812

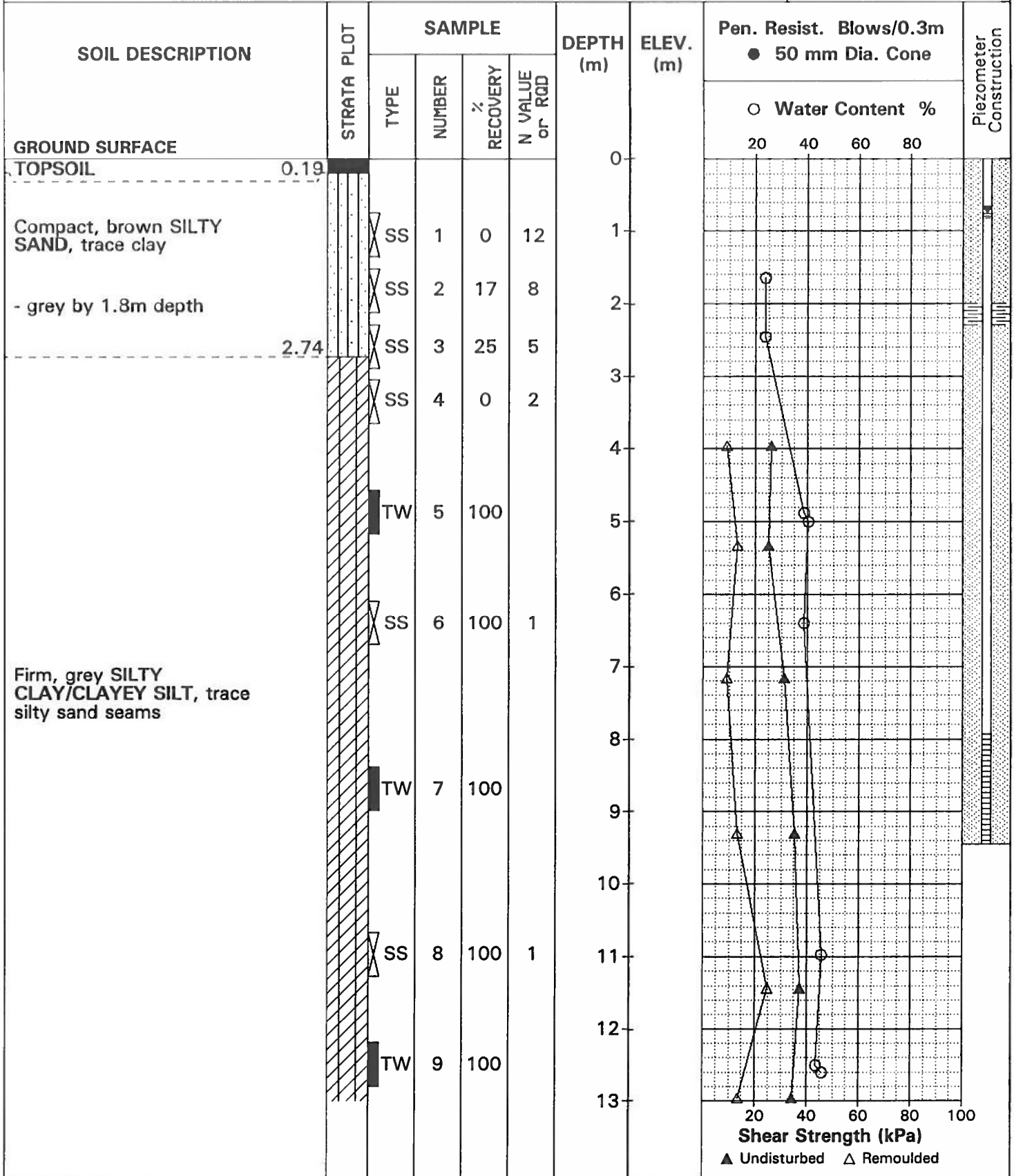
REMARKS

HOLE NO.

BH 2

BORINGS BY CME 75 Power Auger

DATE 1 MAY 06



DATUM

FILE NO.

REMARKS

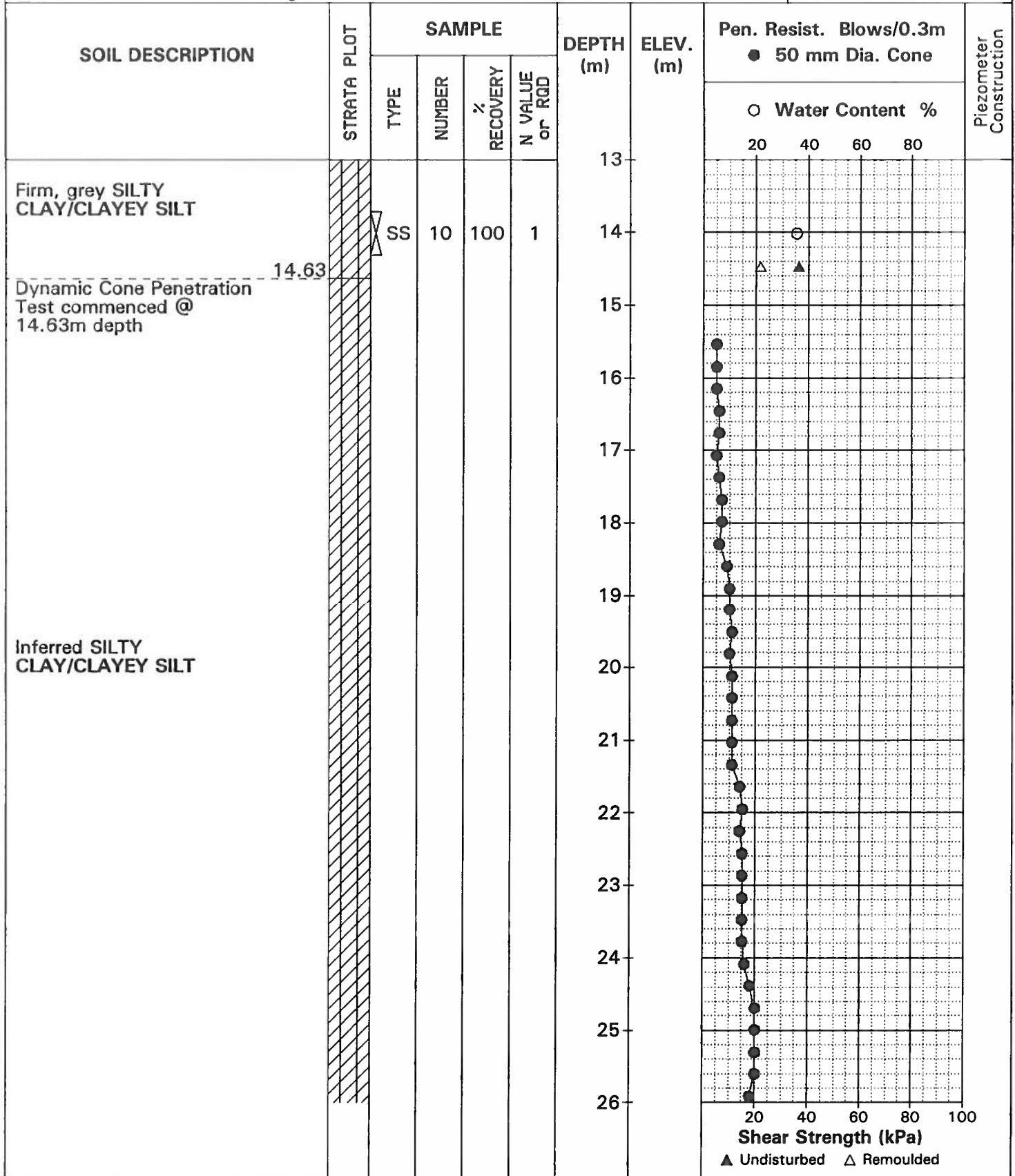
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HOLE NO.

BH 2



DATUM

REMARKS

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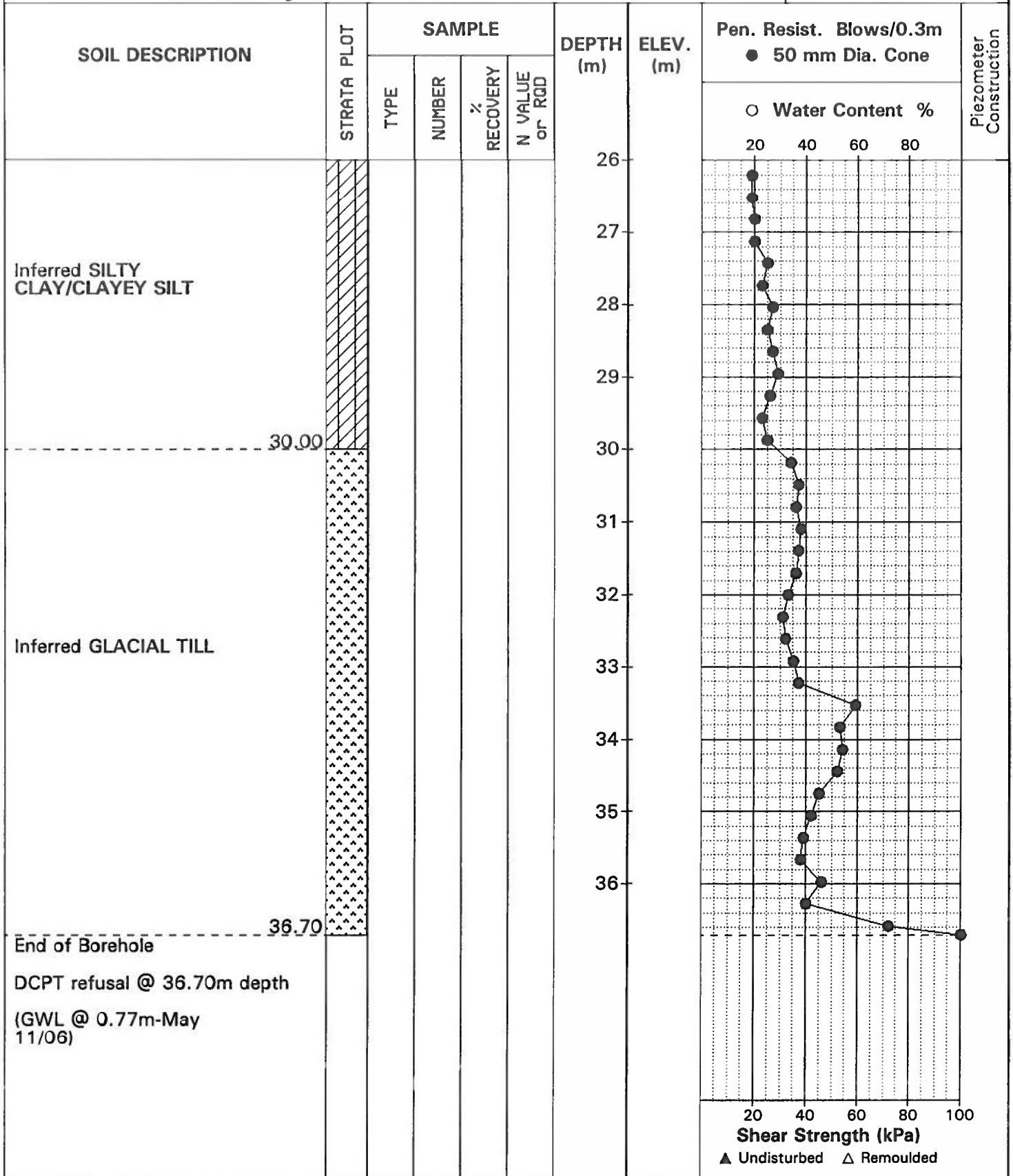
DATE 1 MAY 06

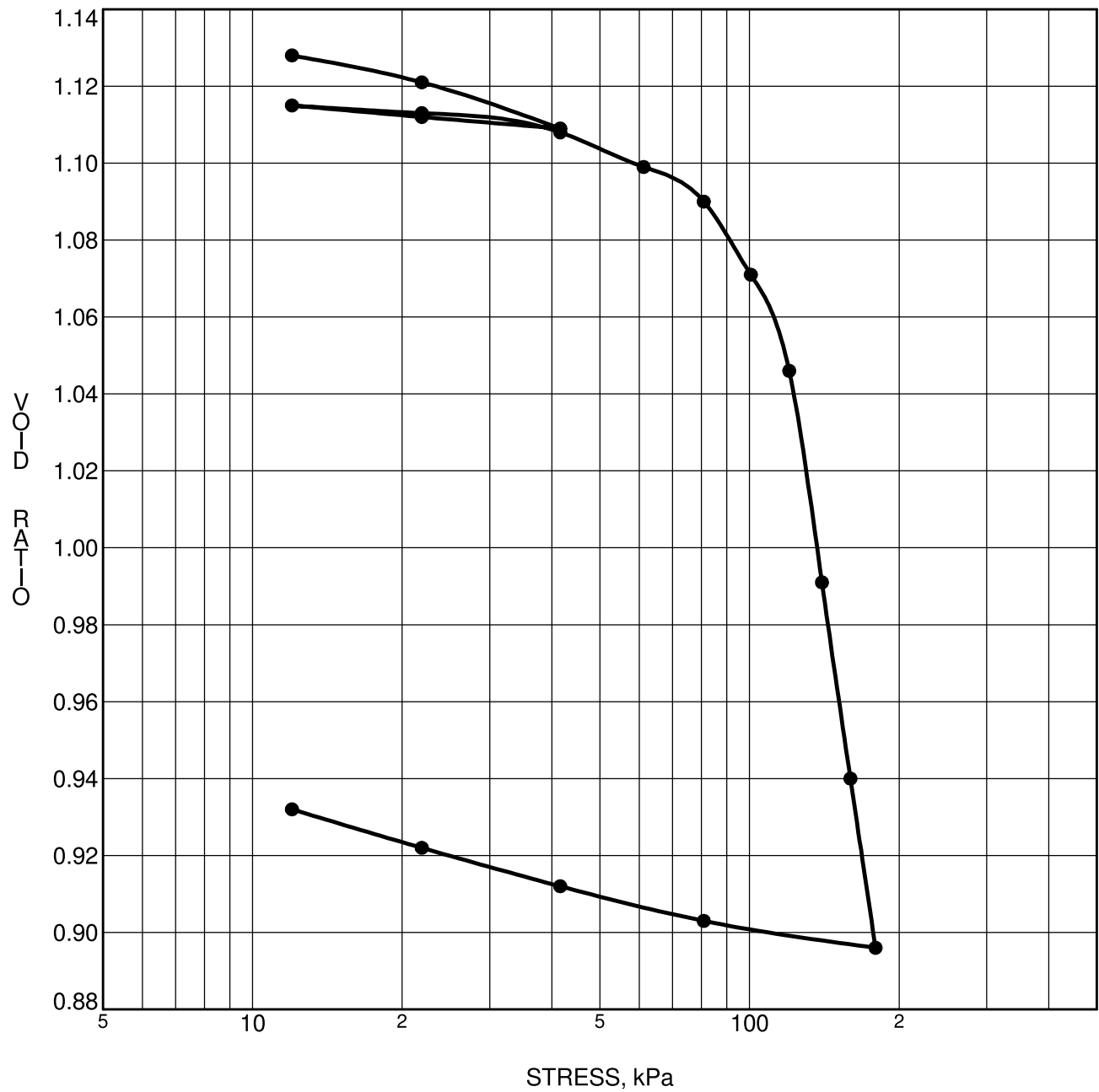
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HOLE NO.

BH 2





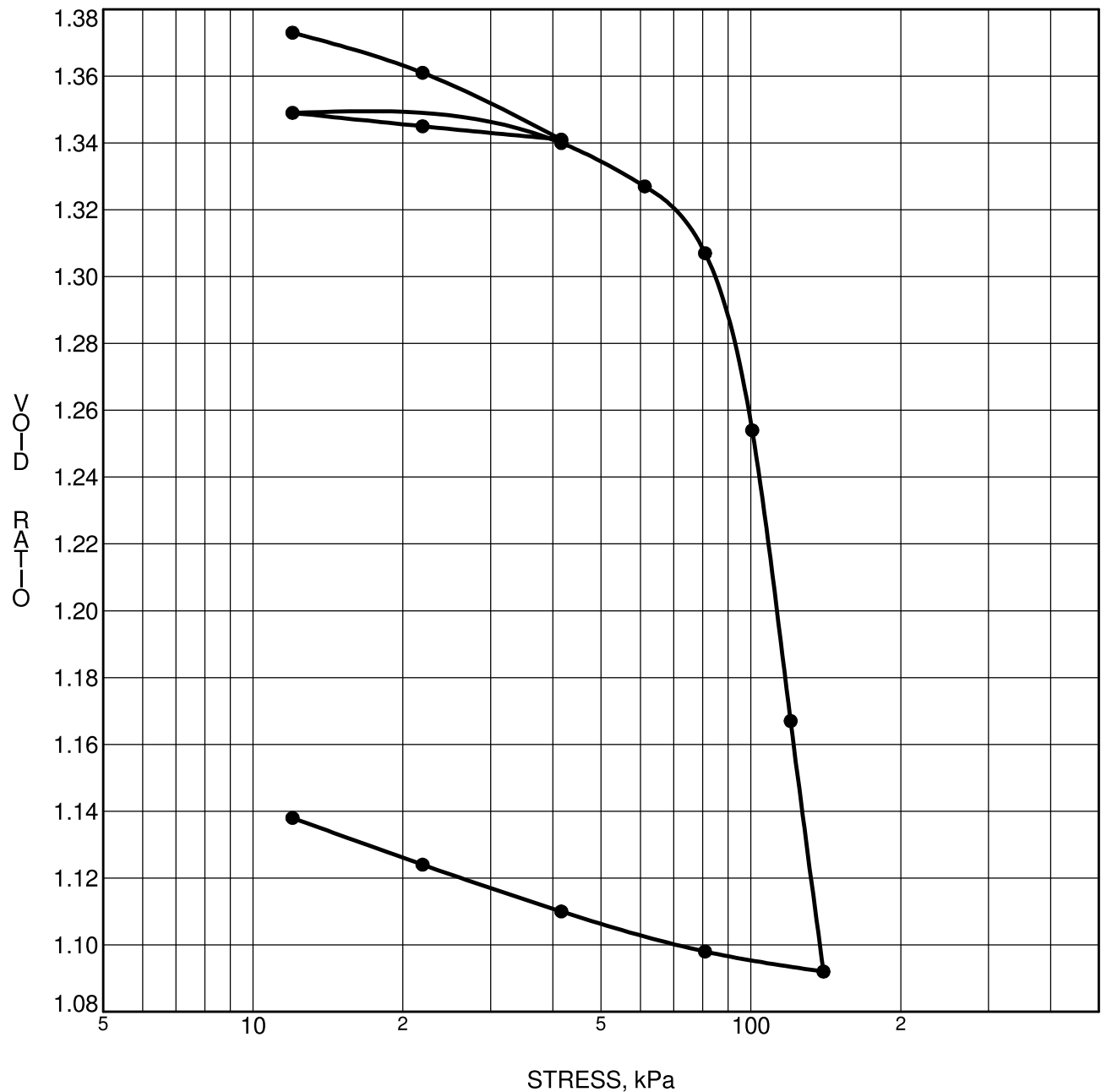
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 3-18	p'_o	41.54 kPa	C_{cr}	0.013
Sample No.	TW 3	p'_c	110.66 kPa	C_c	0.898
Sample Depth	4.04 m	OC Ratio	2.7	W_o	41.4 %
Sample Elev.	92.64 m	Void Ratio	1.139	Unit Wt.	17.8 kN/m ³

CLIENT Street Properties
 PROJECT Geotechnical Investigation - Prop. Commercial
Development - Terry Fox Drive

FILE NO. PG4411
 DATE 03/02/2018

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

CONSOLIDATION TEST



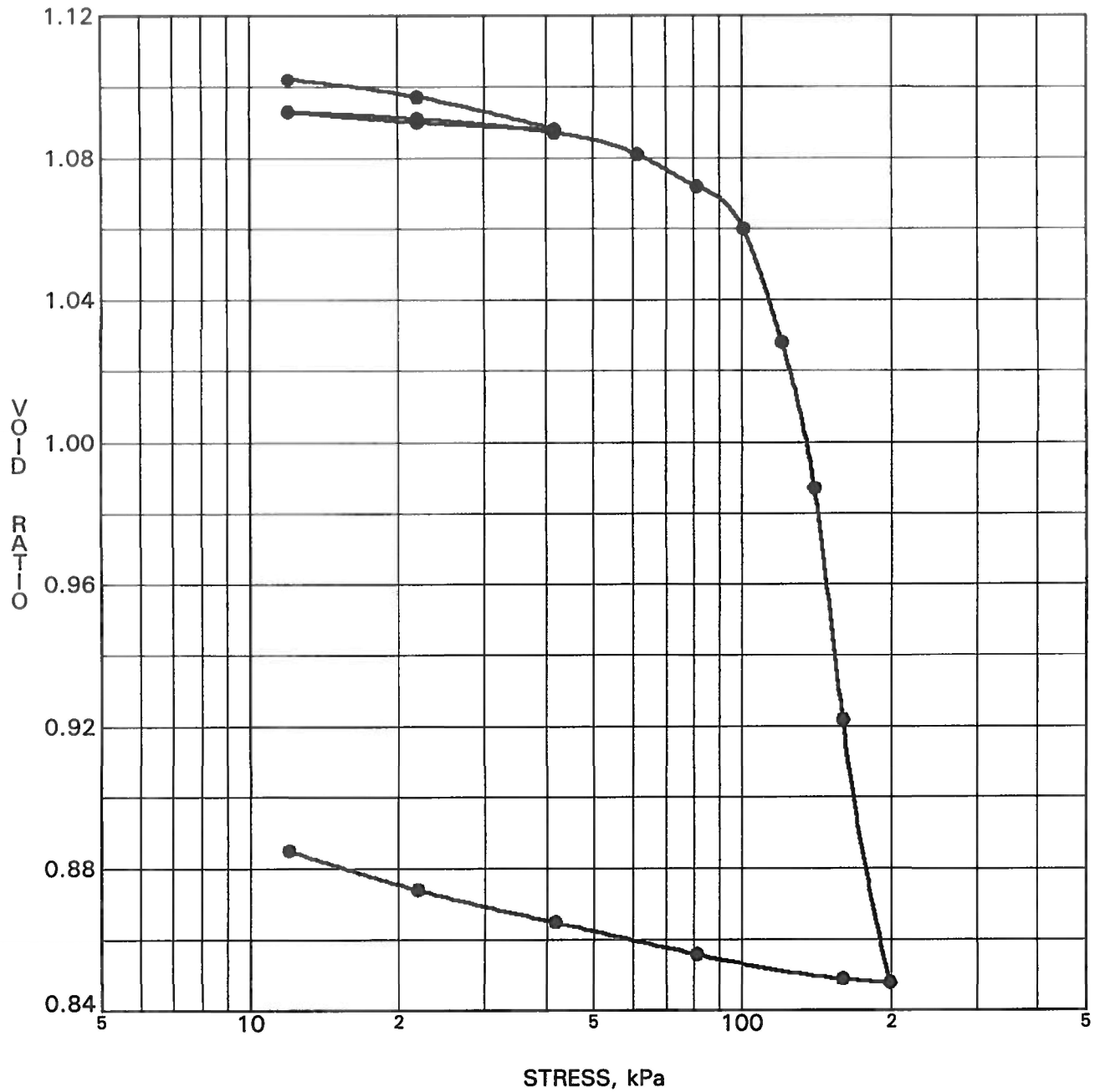
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 7-18	p'_o	40.6 kPa	C_{cr}	0.021
Sample No.	TW 3	p'_c	90 kPa	C_c	1.190
Sample Depth	3.89 m	OC Ratio	2.2	W_o	50.5 %
Sample Elev.	93.24 m	Void Ratio	1.389	Unit Wt.	17.0 kN/m³

CLIENT Street Properties
 PROJECT Geotechnical Investigation - Prop. Commercial
Development - Terry Fox Drive

FILE NO. PG4411
 DATE 05/02/2018

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

CONSOLIDATION TEST



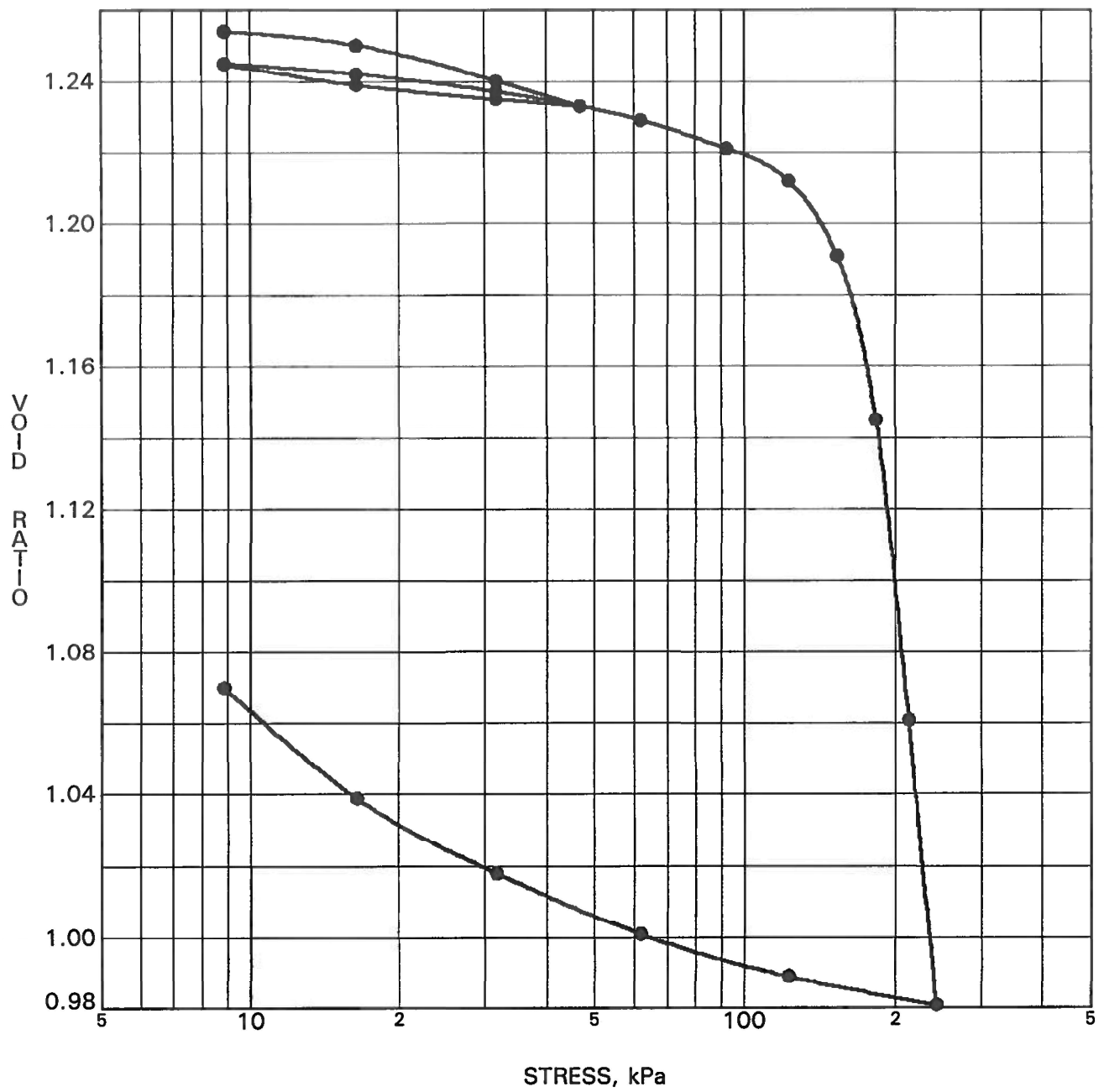
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 2	p'_o	52 kPa	C_{cr}	0.011
Sample No.	TW 5	p'_c	115 kPa	C_c	0.984
Sample Depth	5.00 m	OC Ratio	2.2	W_o	40.3 %
Sample Elev.	m	Void Ratio	1.107	Unit Wt.	18.0 kN/m ³

CLIENT John Van Gaal
 PROJECT Preliminary Geotechnical Investigation - Terry
Fox Drive at Fernbank Road

FILE NO. PG0812
 DATE 05/06/06

paterosongroup Consulting Engineers
 28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



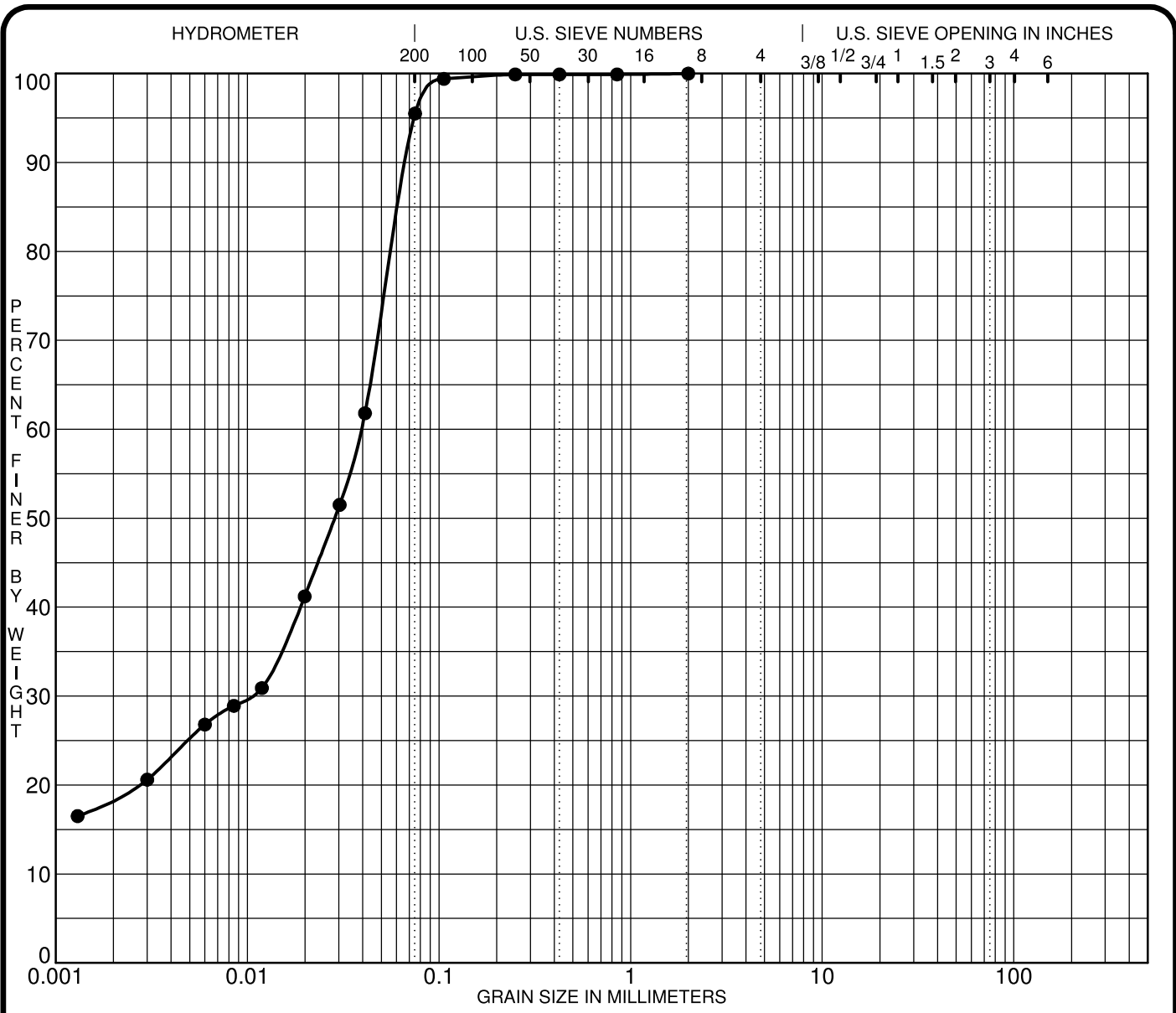
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 2	p'_o	114 kPa	C_{cr}	0.017
Sample No.	TW 9	p'_c	163 kPa	C_c	1.319
Sample Depth	12.60 m	OC Ratio	1.4	W_o	45.7 %
Sample Elev.	m	Void Ratio	1.256	Unit Wt.	17.4 kN/m ³

CLIENT John Van Gaal
 PROJECT Preliminary Geotechnical Investigation - Terry
Fox Drive at Fernbank Road

FILE NO. PG0812
 DATE 05/06/06

patersongroup Consulting Engineers
 28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● BH 1-21 SS3							

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● BH 1-21 SS3	2.00	0.04	0.010		0.0	4.5	95.5	

CLIENT Claridge Homes

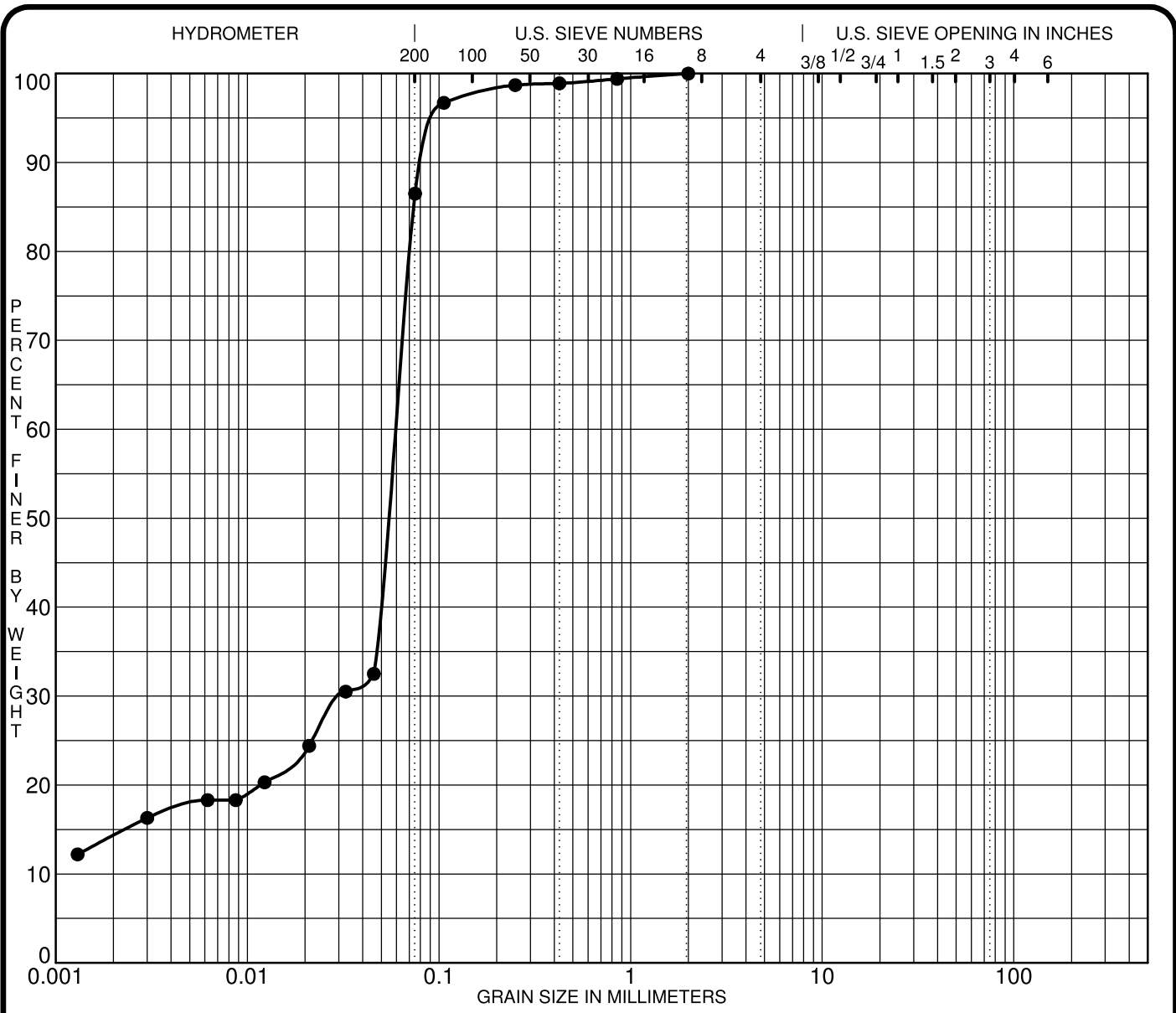
PROJECT Geotechnical Investigation - Proposed Residential Development

FILE NO. PG5683

DATE 9 Feb 21

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

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● BH 4-21 SS2							

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● BH 4-21 SS2	2.00	0.06	0.031		0.0	13.5	86.5	

CLIENT Claridge Homes
 PROJECT Geotechnical Investigation - Proposed Residential Development

FILE NO. PG5683
 DATE 9 Feb 21

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

GRAIN SIZE DISTRIBUTION

Certificate of Analysis
 Client: Paterson Group Consulting Engineers
 Client PO: 23169

Report Date: 12-Feb-2018

Order Date: 6-Feb-2018

Project Description: PG4411

Client ID:	BH3-18-SS2	-	-	-
Sample Date:	06-Feb-18	-	-	-
Sample ID:	1806239-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	81.1	-	-	-
----------	--------------	------	---	---	---

General Inorganics

pH	0.05 pH Units	7.65	-	-	-
Resistivity	0.10 Ohm.m	62.6	-	-	-

Anions

Chloride	5 ug/g dry	17	-	-	-
Sulphate	5 ug/g dry	41	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5683-1 - TEST HOLE LOCATION PLAN

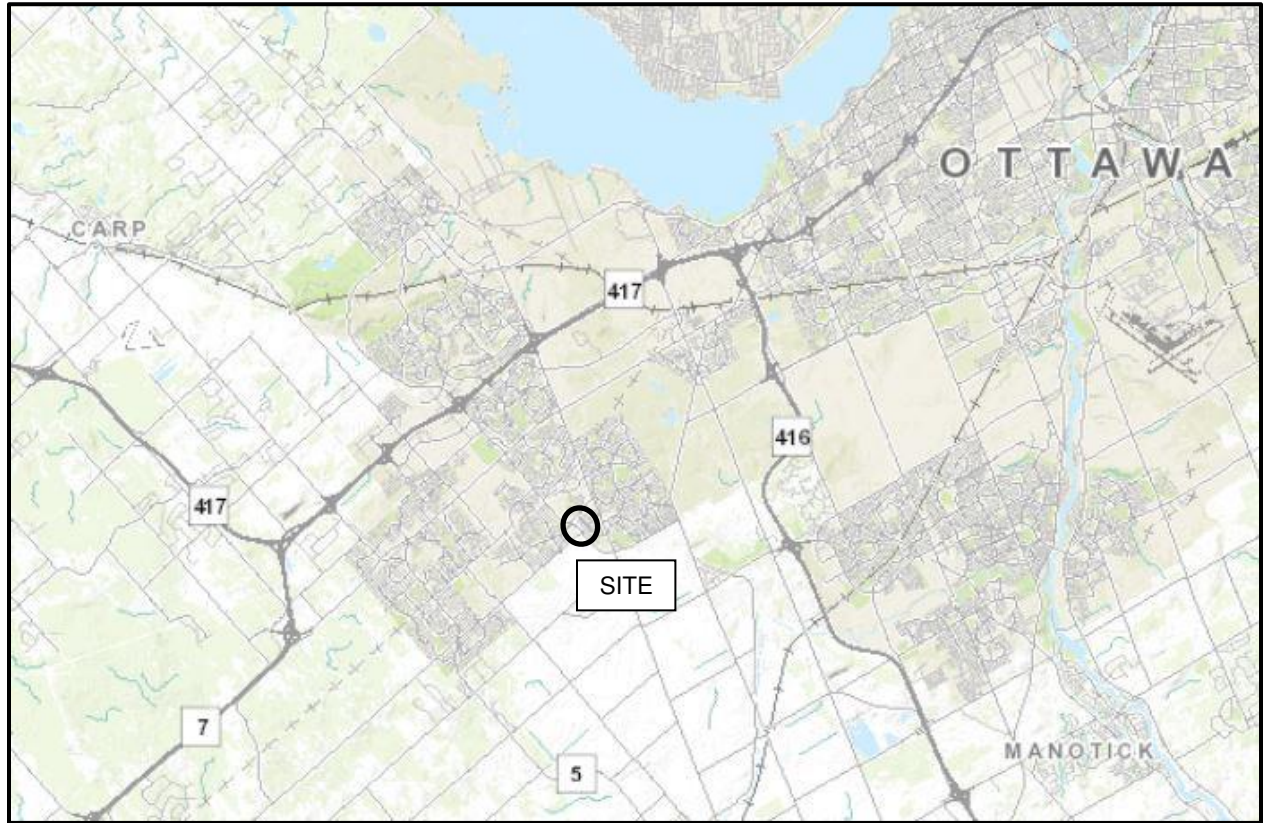
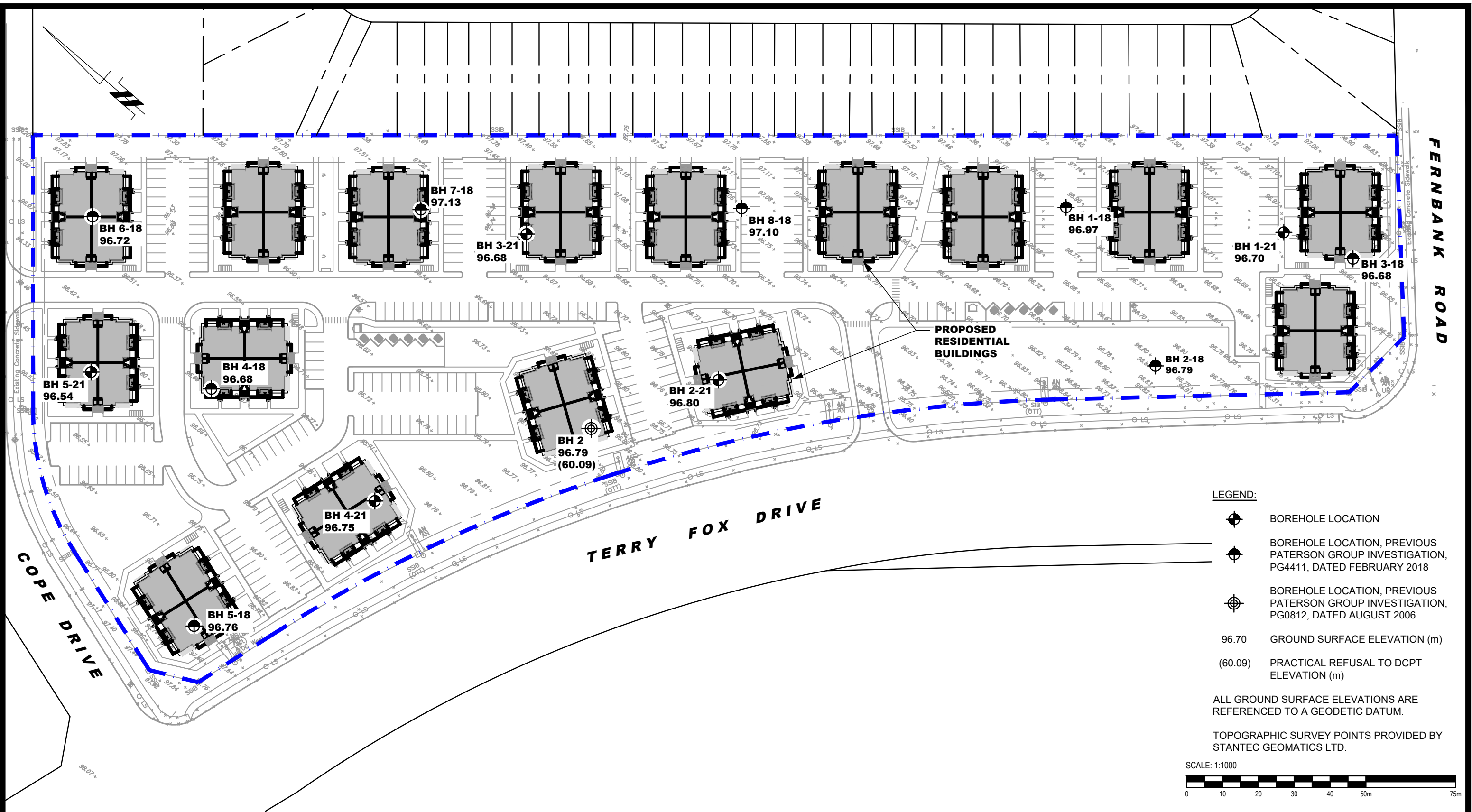


FIGURE 1

KEY PLAN

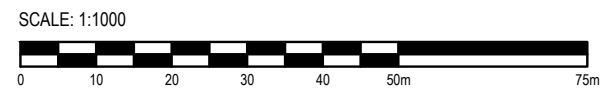


LEGEND:

- BOREHOLE LOCATION
- BOREHOLE LOCATION, PREVIOUS PATERSON GROUP INVESTIGATION, PG4411, DATED FEBRUARY 2018
- BOREHOLE LOCATION, PREVIOUS PATERSON GROUP INVESTIGATION, PG0812, DATED AUGUST 2006
- 96.70 GROUND SURFACE ELEVATION (m)
- (60.09) PRACTICAL REFUSAL TO DCPT ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

TOPOGRAPHIC SURVEY POINTS PROVIDED BY STANTEC GEOMATICS LTD.



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
5331 FERNBANK ROAD
ONTARIO

OTTAWA,
Title:

TEST HOLE LOCATION PLAN

Scale:	1:1000	Date:	02/2021
Drawn by:	NFRV	Report No.:	PG5683-1
Checked by:	VD	Dwg. No.:	PG5683-1
Approved by:	DJG	Revision No.:	

c:\users\nicholas\desktop\pg5683-1\hlp.dwg