

Geotechnical
Engineering

Environmental
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Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation
Proposed Multi-Storey Building
340 Huntmar Drive
Ottawa, Ontario

Prepared For

Kanata Microtel
10537616 Canada Ltd.

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Kanata Microtel to conduct a geotechnical investigation for the proposed multi-storey building to be located at 340 Huntmar Drive in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were:

- ❑ to determine the subsurface soil and groundwater conditions by means of boreholes,
- ❑ to provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of a multi-storey hotel building of slab-on-grade construction. It is further understood that associated access lanes, parking and landscaped areas will occupy the remainder of the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A geotechnical investigation conducted on October 9 to 11, and 15, 2013 included a total of 5 boreholes completed at, or in the vicinity of, the subject site. The locations of the relevant test holes are shown on Drawing PG4544-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights and a 50 mm diameter split-spoon sampler. The soil from the auger flights and split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the auger flight and split-spoon samples were recovered from the boreholes are depicted as AU and SS, 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 conducted at regular intervals in cohesive soils and completed using an MTO field vane apparatus.

The thickness of the overburden was evaluated by dynamic cone penetration testing (DCPT) at boreholes BH 4 and BH 5. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the 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.

Groundwater

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

3.2 Field Survey

The test hole locations were selected by Paterson and located and surveyed in the field by Stantec Geomatics. The ground surface elevations at the test hole locations are understood to be referenced to a geodetic datum. The locations and ground surface elevations of the test holes are presented on Drawing PG4544-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was obtained during the above-noted geotechnical investigation and submitted for analytical purposes. This sample was obtained from a borehole located approximately 100 m to the north of the subject site, and 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 also submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and is bordered by Huntmar Drive to the west, adjacent undeveloped properties to the north and east, and the Feedmill Creek valley corridor to the south. The ground surface across the site is relatively level, varying from approximate geodetic elevation 99 to 101 m, and is generally covered with minor vegetated growth.

The adjacent section of Feedmill Creek meanders in a west to east direction toward the Carp River within the approximately 15 to 25 m wide valley corridor with a 2 to 2.5 m high valley wall. It was noted that the watercourse is approximately 0.3 to 0.6 m deep, 2 to 3 m wide, and is located along the toe of the south valley wall.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consisted of a silty clay deposit encountered at the ground surface, or underlying an approximate 0.6 m thickness of fill. The silty clay deposit was observed to consist of a hard to stiff, brown silty clay crust extending to depths of 2.5 to 3.5 m, overlying a stiff to firm, grey silty clay.

Practical refusal to the DCPTs were encountered at depths of 15.8 m and 12.1 m in boreholes BH 4 and BH 5, respectively. 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 with drift thicknesses ranging from 10 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the standpipes on October 21, 2013 for boreholes completed as part of the previous investigation. The results of our groundwater readings from existing boreholes are presented in Table 1. It should be noted that surface water can become trapped within the backfilled borehole, which can lead to higher than normal groundwater level readings. The long term groundwater level can also be estimated based on the recovered soil sample's moisture level and consistency. Based on these observations, the long term groundwater table is anticipated to be at a 2.5 to 4 m depth. It should be further noted that the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Water Level		Date
		Depth (m)	Elevation (m)	
BH 4	99.35	2.23	97.12	October 21, 2013
BH 5	98.99	2.36	96.63	October 21, 2013
BH 6	98.90	Damaged	-	October 21, 2013
BH 7	97.75	Damaged	-	October 21, 2013
BH 21	98.55	0.91	97.64	October 21, 2013

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It is expected that the proposed multi-storey building will be founded by conventional shallow footings placed on an undisturbed, hard to stiff silty clay bearing surface.

Due to the presence of the silty clay deposit at the site, the proposed development will be subjected to a permissible grade raise restriction. If the grade raise restriction is exceeded, several options are available such as a preload/surcharge program or the placement of lightweight fill below the proposed building.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

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. 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 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 and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, 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, the material should be compacted in thin lifts to a minimum density of 95% of the respective SPMDD.

Backfill against foundation walls should consist of free-draining, non frost susceptible granular materials. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against the foundation walls, unless used in conjunction with a composite drainage blanket connected to the perimeter foundation drainage system.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed on an undisturbed, very stiff to stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

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

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.

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 hard to 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.

Permissible Grade Raise Recommendations

A permissible grade raise restriction of **2 m** is recommended for grading within 5 m of the proposed buildings. A permissible grade raise restriction of **3 m** is recommended in the parking areas and access lanes. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class D** as defined in Table 4.1.8.4.A of the Ontario Building Code 2012 for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and fill, containing deleterious or organic materials, the native soil will be considered to be an acceptable subgrade surface on which to commence backfilling for slab on grade construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

5.6 Pavement Structure

For design purposes, the pavement structures presented in the following tables shall be used for the design of car only parking areas, heavy truck parking areas and access lanes.

It is anticipated that the proposed pavement structures will be placed over either a hard to stiff silty clay or engineered fill subgrade.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil	

Table 3 - Recommended Pavement Structure Heavy Truck Parking Areas and Access Lanes	
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
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil	

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

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

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

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 the load bearing capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. 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 structure as an outlet for perched water below the sidewalks anticipated to surround the building. Perched water below the sidewalks can lead to heaved sidewalks due to freeze/thaw cycles. The system should consist of a 100 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. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 blanket, such as Miradrain G100N or Delta Drain 6000.

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.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, or an equivalent 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 be either 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 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

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

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

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the 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. 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 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 groundwater infiltration into the excavations should be 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.

A temporary MOECC permit to take water (PTTW) may be required 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 MOECC.

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

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.

6.7 Corrosion Potential and Sulphate

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

6.8 Tree Planting Restrictions

Given that the multi-storey building is proposed to have a finished floor elevation of 99.86 m, the underside of footing is expected at approximately elevation 98.4 m. Based on the subsurface conditions encountered in the boreholes, the hard to stiff silty clay crust extends to approximate geodetic elevations of 95.5 to 95 m. As such, the silty clay which extends 3 to 3.5 m below design footing level should be considered low to medium sensitivity clay and should not be considered a sensitive marine clay.

Based on the above discussion, it is recommended that trees placed within 4.5 m of the foundation wall consist of street trees with shallow root systems that extend less than 1.5 m below ground surface. Trees placed greater than 4.5 m from the foundation wall may consist of moderate water demanding trees with roots extending to a maximum 2 m depth. It should be noted that shrubs and other small plantings are permitted within the 4.5 m setback area.

It is documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils which 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 should not be considered in the landscaping design.

6.9 Infiltration System Recommendations

It is understood that the proposed development will include a stormwater infiltration system. Based on our review of published values for silty clay and the results of a previous hydrogeological investigation completed in the general area, the recommended percolation rate for the stormwater infiltration system at this site is 35 to 50 minutes/cm.

It is also recommended that the bottom of the stormwater infiltration system be located a minimum of 1 m above the long term groundwater table, in accordance with the Ministry of Environment (MOE) Stormwater Design Manual.

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.

- Review final 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 granular fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Kanata Microtel or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



Scott S. Dennis, P.E.



David J. Gilbert, P.Eng.

Report Distribution:

- Kanata Microtel (e-mail copy)
- Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

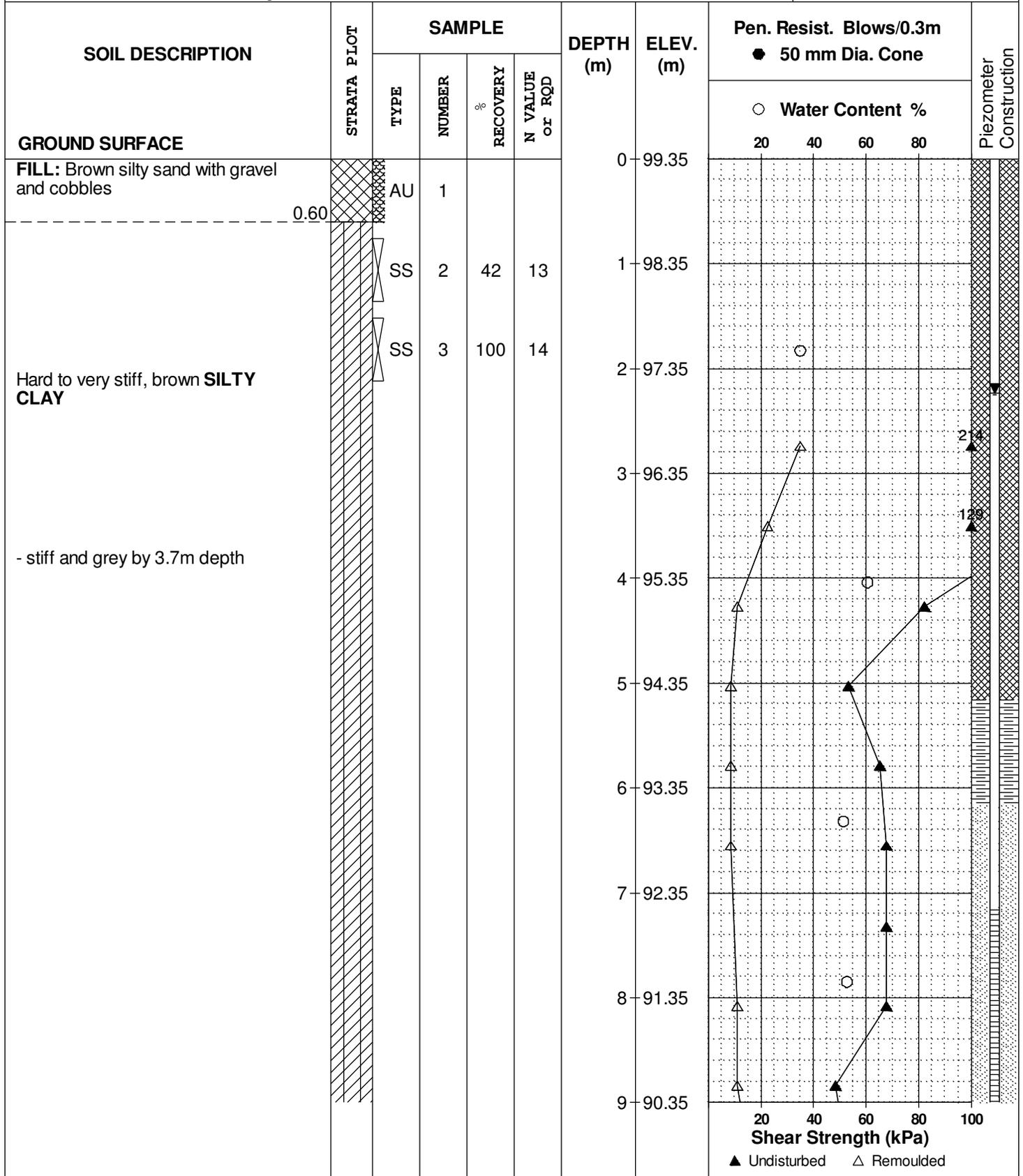
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REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 10 October 2013



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

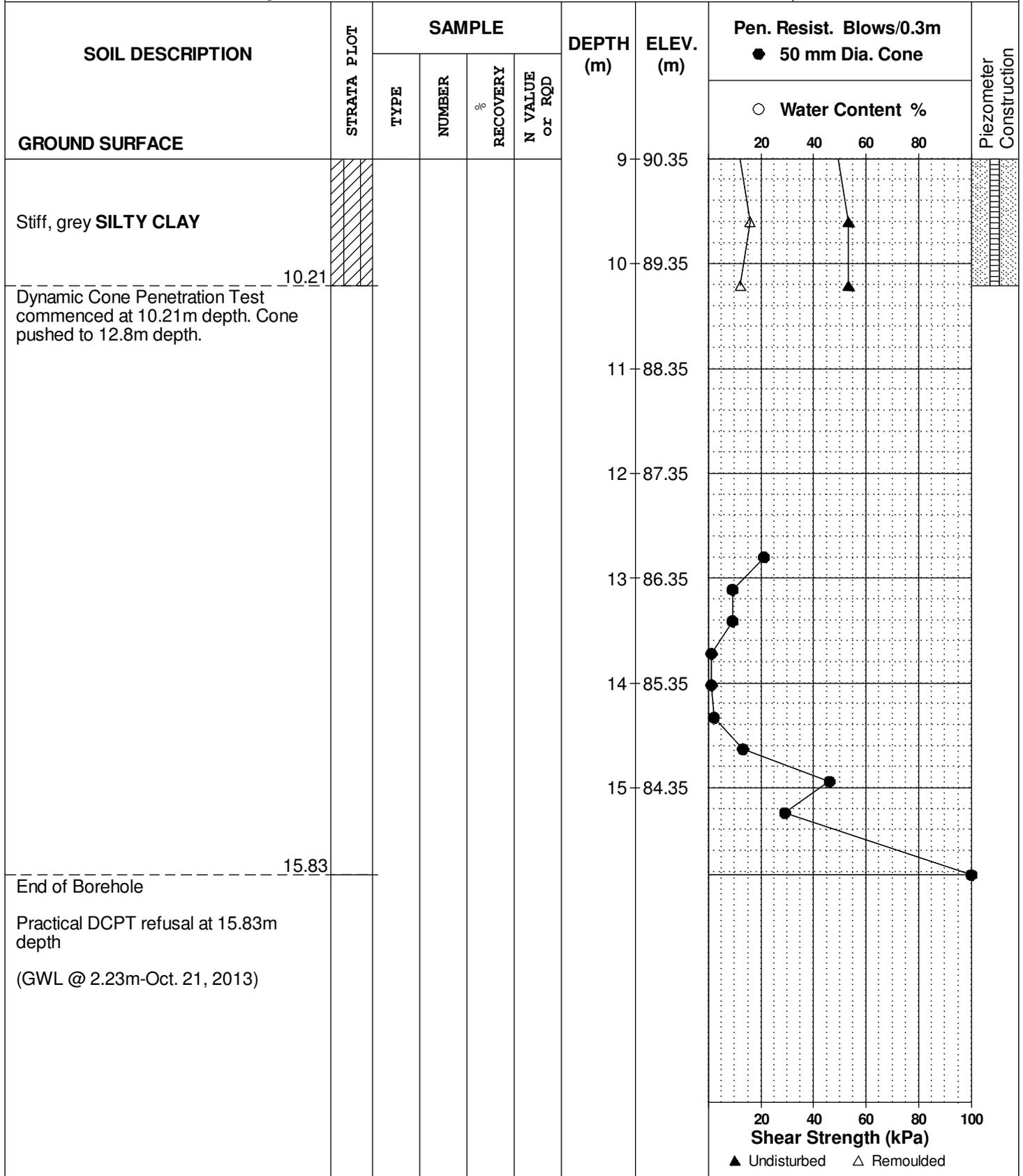
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REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 10 October 2013



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

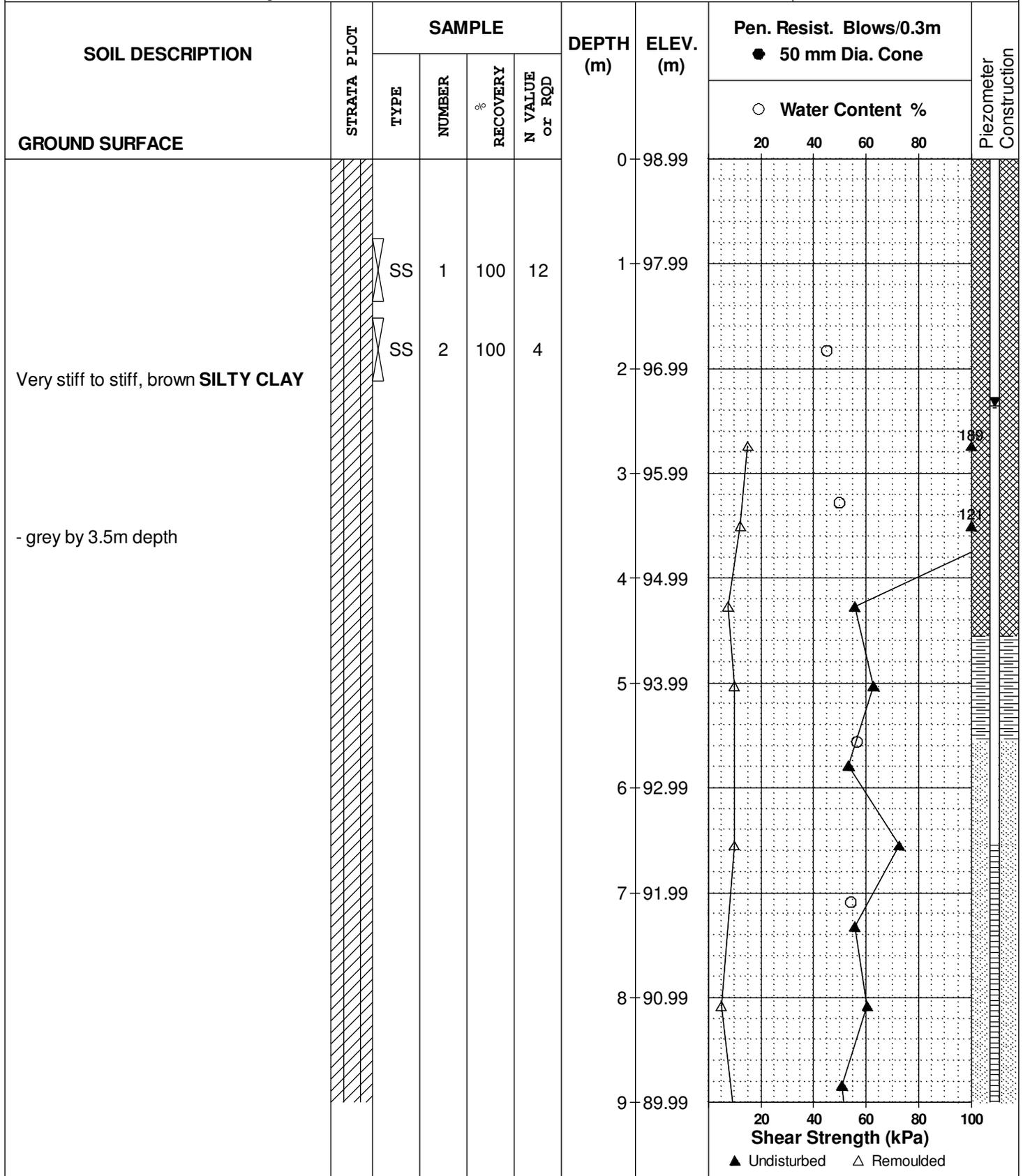
DATE 10 October 2013

FILE NO.

PG4544

HOLE NO.

BH 5



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

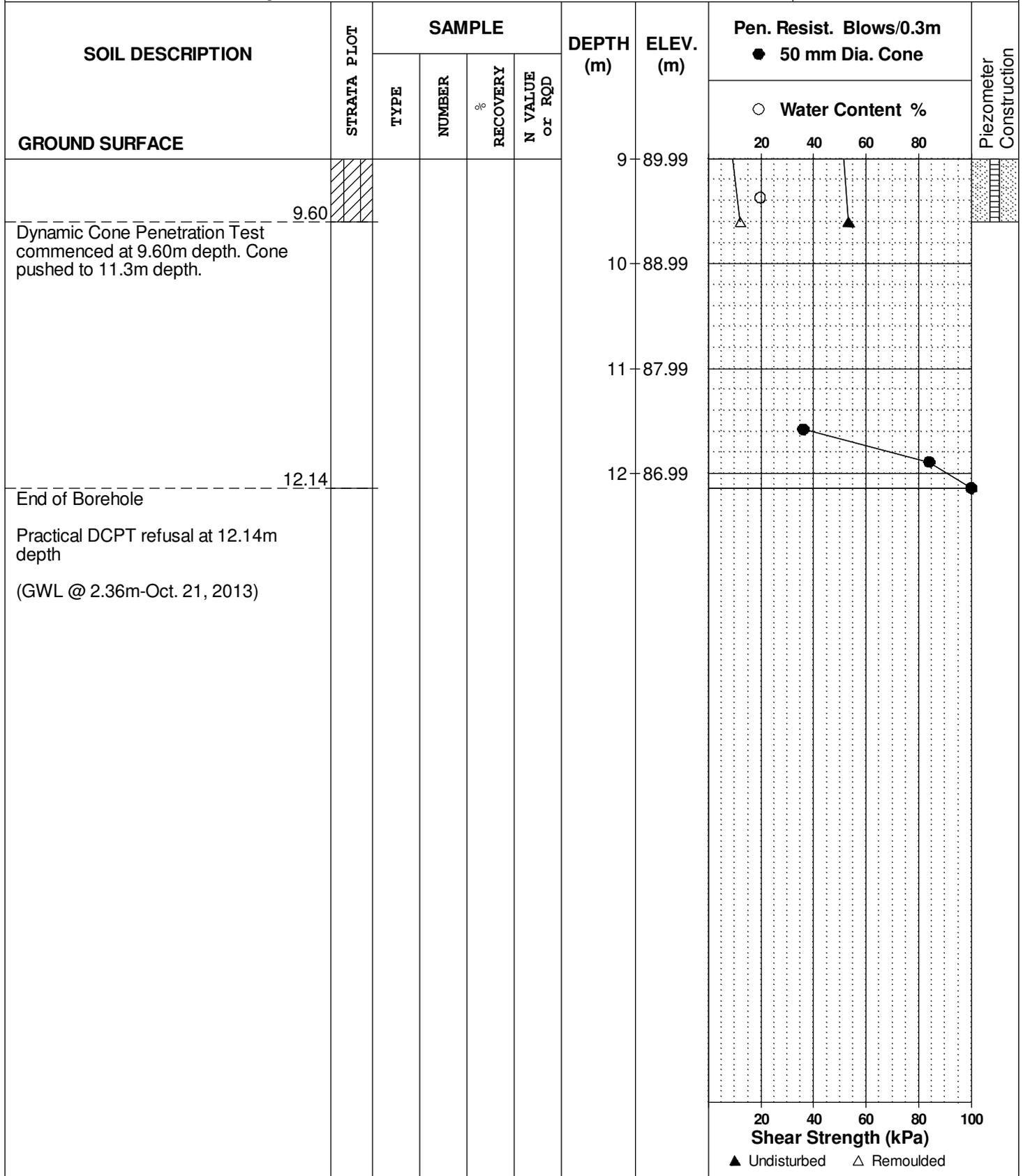
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REMARKS

HOLE NO. **BH 5**

BORINGS BY CME 55 Power Auger

DATE 10 October 2013



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

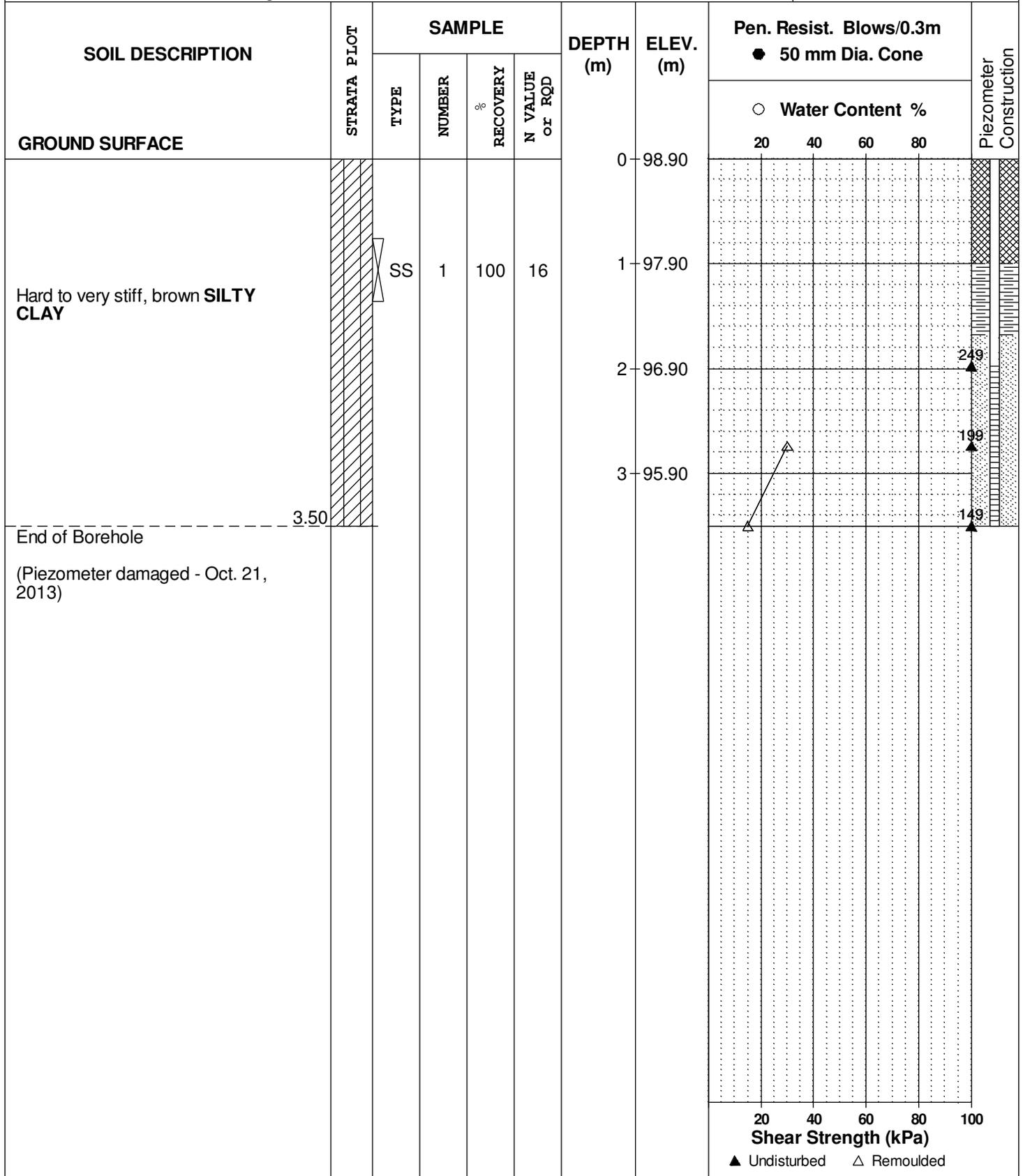
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REMARKS

HOLE NO. **BH 6**

BORINGS BY CME 55 Power Auger

DATE 15 October 2013



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

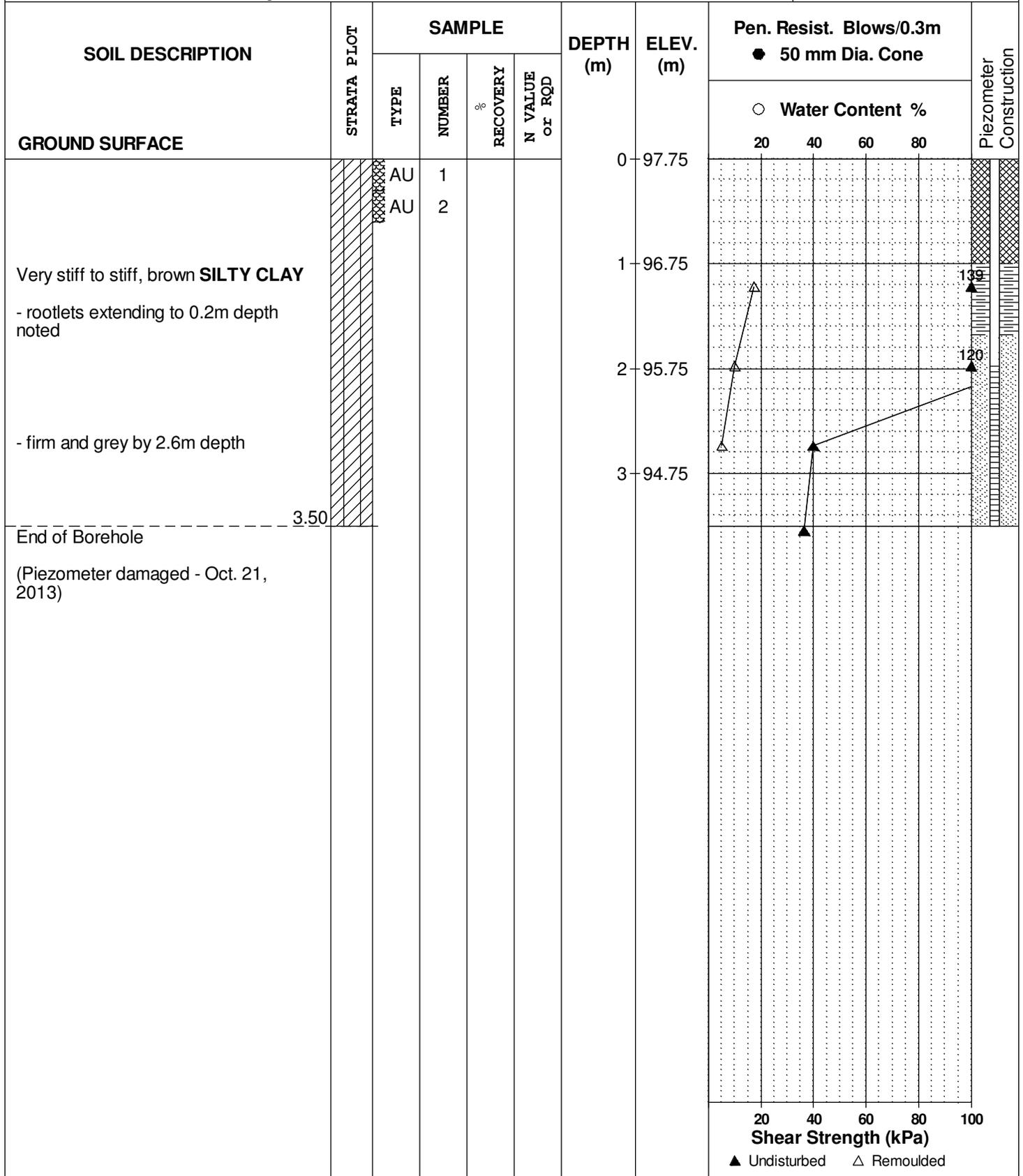
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REMARKS

HOLE NO. **BH 7**

BORINGS BY CME 55 Power Auger

DATE 9 October 2013



DATUM Ground surface elevations provided by Stantec Geomatic Ltd.

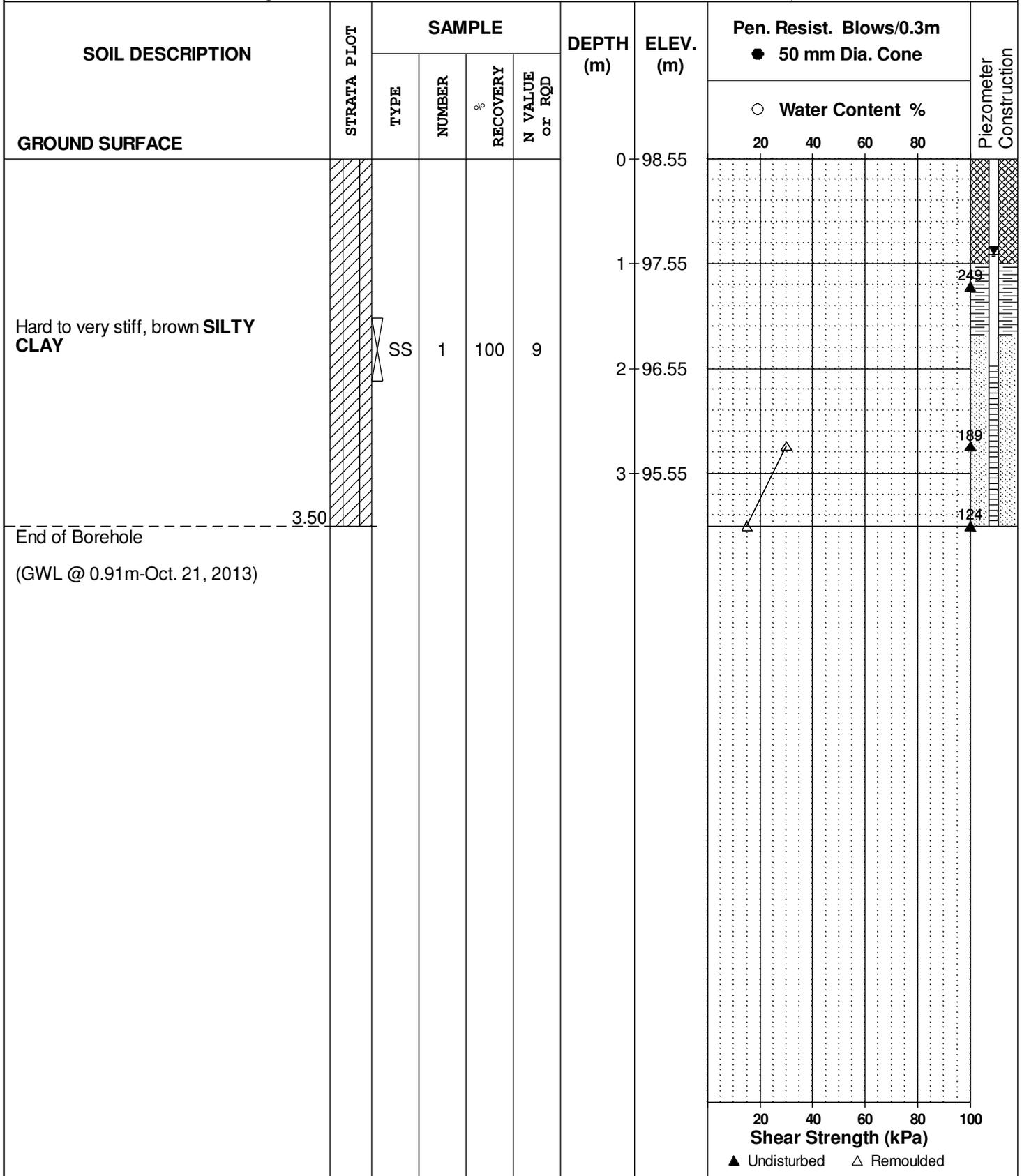
FILE NO. **PG4544**

REMARKS

HOLE NO. **BH21**

BORINGS BY CME 55 Power Auger

DATE 15 October 2013



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)

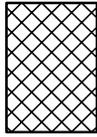
STRATA PLOT



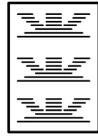
Topsoil



Asphalt



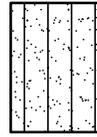
Fill



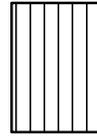
Peat



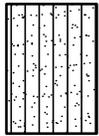
Sand



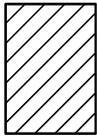
Silty Sand



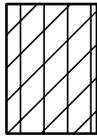
Silt



Sandy Silt



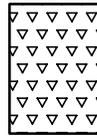
Clay



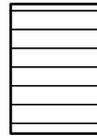
Silty Clay



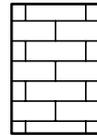
Clayey Silty Sand



Glacial Till



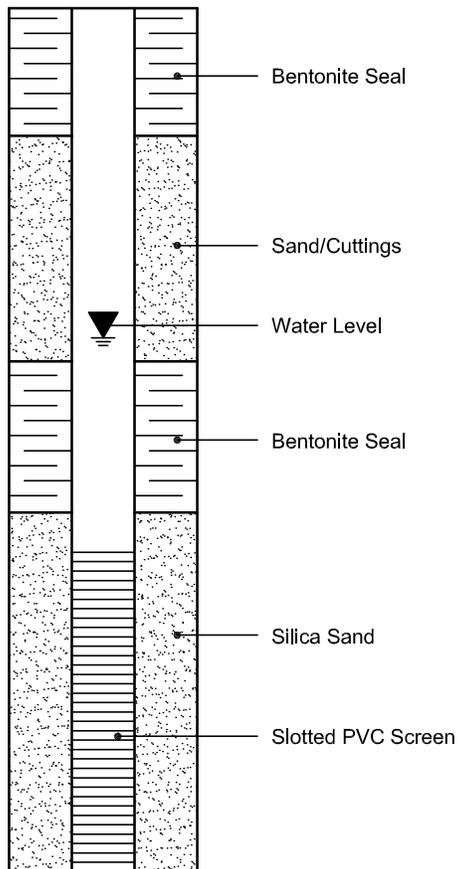
Shale



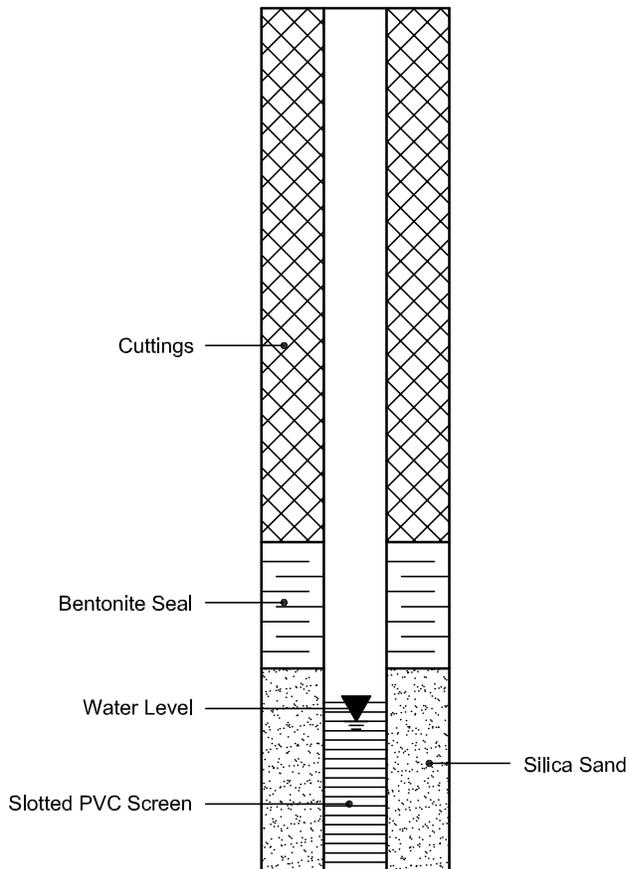
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 22-Oct-2013

Client: **Paterson Group Consulting Engineers**

Order Date: 16-Oct-2013

Client PO: 15096

Project Description: PG3045

Client ID:	BH12-SS1	-	-	-
Sample Date:	10-Oct-13	-	-	-
Sample ID:	1342113-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	68.0	-	-	-
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General Inorganics

pH	0.05 pH Units	7.61	-	-	-
Resistivity	0.10 Ohm.m	31.5	-	-	-

Anions

Chloride	5 ug/g dry	79	-	-	-
Sulphate	5 ug/g dry	47	-	-	-

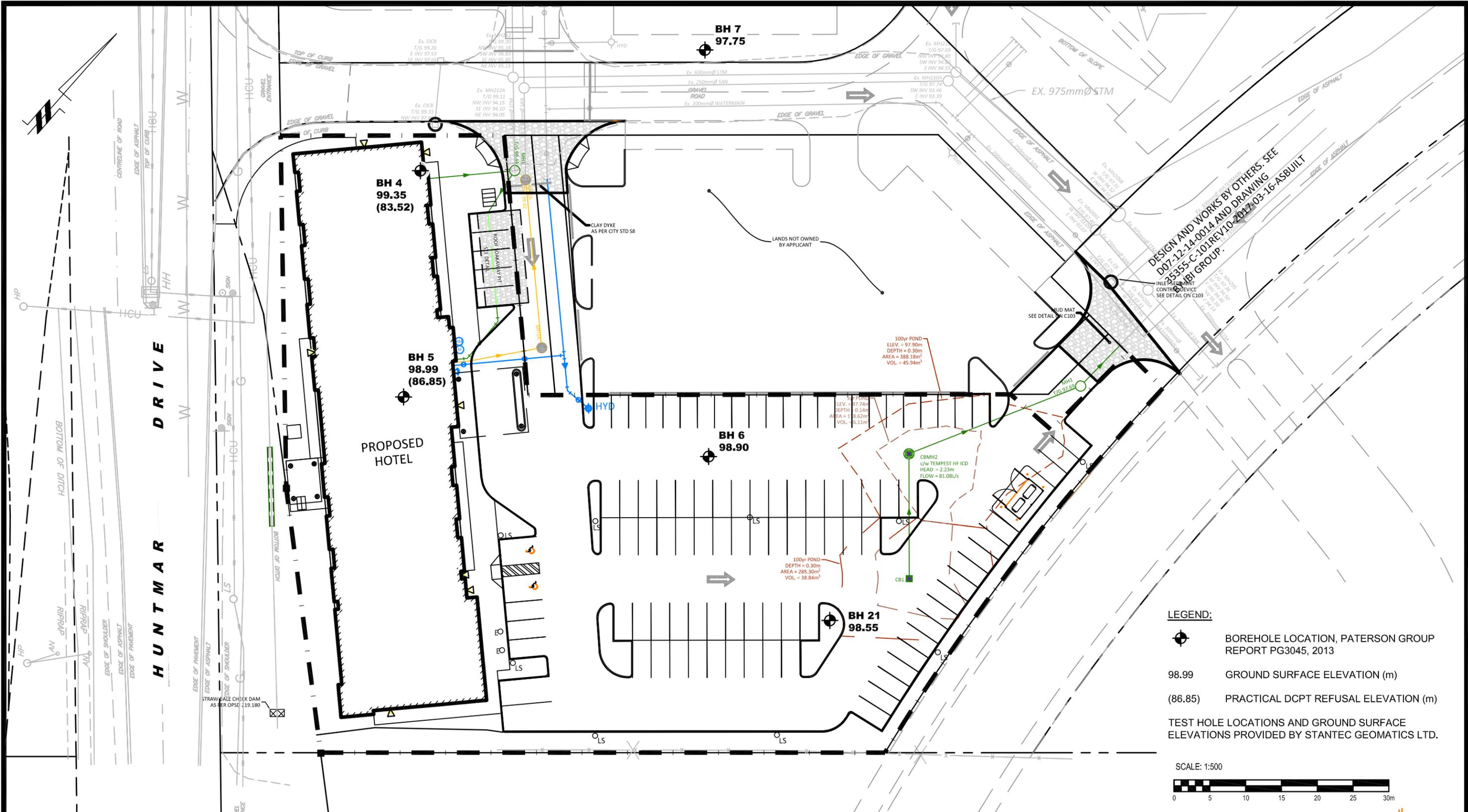
APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4544-1 - TEST HOLE LOCATION PLAN



FIGURE 1
KEY PLAN

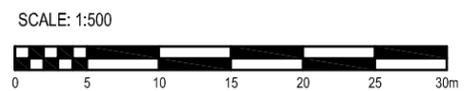


DESIGN AND WORKS BY OTHERS. SEE
 D07-12-14-0014 AND DRAWING
 25355-C-101REV10-2017-03-16-ASBUILT
 PATERSON GROUP

LEGEND:

- BOREHOLE LOCATION, PATERSON GROUP REPORT PG3045, 2013
- 98.99 GROUND SURFACE ELEVATION (m)
- (86.85) PRACTICAL DCPT REFUSAL ELEVATION (m)

TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS 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
0			

KANATA MICROTREL
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STORY BUILDING - 340 HUNTMAR DRIVE

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	06/2018
Drawn by:	MPG	Report No.:	PG4544-1
Checked by:	SD	Dwg. No.:	PG4544-1
Approved by:	SD	Revision No.:	0

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