

**Geotechnical
Engineering**

**Environmental
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Hydrogeology

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Engineering**

Materials Testing

Building Science

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Geotechnical Investigation

Proposed Multi-Storey Building
260 MacLaren Street
Ottawa, Ontario

Prepared For

Claridge Homes

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Report PG2232-1

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1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 260 MacLaren Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 PROPOSED PROJECT

It is understood that the proposed development will consist of a nine (9) storey residential building with three (3) levels of underground parking covering the majority of the subject site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on October 5 and 6, 2010. At that time, three (3) boreholes were advanced to a sampling depth of 14 m and followed by a Dynamic Cone Penetration Test (DCPT). The test hole locations were selected in a manner to provide general coverage of the proposed development. The borehole locations are shown on Drawing PG2232-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted power auger drill rig, operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from our geotechnical division. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory. The depths at which the split spoon, Shelby tubes and auger samples were recovered from the boreholes are shown as SS, TW and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

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

A dynamic cone penetration test (DCPT) was completed at all three (3) boreholes. The DCPT consists in driving a steel rod equipped with a cone at its tip using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increments.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of the manhole located along MacLaren Street, near the southwest corner of the intersection with Metcalfe Street. An assumed elevation of 100 m was assigned to the TBM. The location and ground surface elevations at borehole locations are presented on Drawing PG2232-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 **OBSERVATIONS**

4.1 **Surface Conditions**

The subject site is currently an asphaltic concrete paved parking lot, the ground surface at the subject site is relatively flat and at grade with surrounding streets. The site is bordered to the east and west by a multi-storey building and a residential dwelling, respectively, which are both located in close proximity to the adjacent property line, a multi-storey building and an asphalt concrete paved parking lot are located to the south of the subject site.

4.2 **Subsurface Profile**

The subsurface profile encountered at the borehole locations, consists of a pavement structure underlain by a silty sand fill mixed with brick, concrete chunks and wood, which in turn is underlain by a deep stiff silty clay deposit. Practical refusal to DCPT was encountered at depths ranging between 22.8 to 24.7 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

4.3 **Groundwater**

Groundwater levels were measured in the standpipes on October 26, 2010 and the results are presented in Table 1. Based on colour and consistency of the recovered soil samples, the groundwater table is expected between 3 to 4 m depth. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels			
Test Hole Number	Ground Surface Elevation (m)	Water Level	
		Depth (m)	Elevation (m)
BH 1	100.34	Dry to 5.5 m	N/A
BH 2	100.38	Dry to 5.2 m	N/A
BH 3	100.34	3.28	97.10

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building. It is anticipated that the proposed building will be founded on either end bearing piles or a raft foundation.

It is expected that temporary shoring will be required for the excavation of the underground levels. It is further expected that lateral support for the east neighbouring building will have to be replaced due to the anticipated shoring location.

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

5.2 Site Grading and Preparation

Stripping Depth

It is expected that all the surficial soil will be removed within the footprint of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building, should consist 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 225 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of its standard Proctor maximum dry density.

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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to a minimum density of 95% of its standard Proctor maximum dry density. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls.

5.3 Foundation Design

Conventional Shallow Footings

Footings placed on an undisturbed, stiff silty clay bearing surface at a depth of approximately 10 m below the existing grade (3 levels of underground parking), can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Footings designed using the abovenoted bearing resistance value at SLS can expect 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.

Raft Foundation

Consideration can be given to a raft foundation to found the proposed building. The factored bearing resistance (contact pressure) at ultimate limit states (ULS) can be taken to be **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The design should allow for the use of no more than 80% of the overconsolidation of the silty clay and account for a potential 0.5 m post-development groundwater lowering. A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **150 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

The modulus of subgrade reaction was estimated to be **1.5 MPa/m** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, sensitive silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus. This value can be re-evaluated once detail of the structural design becomes available.

Based on the above assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively. The base of the raft foundation (approximately 1 m thick) is to be located at a depth of approximately 10 m below the original ground surface. The raft slab is relatively impervious and the basement walls below the long-term groundwater table will be adequately waterproofed.

Piled Foundation

Consideration can be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data					
Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class D for the shallow foundations considered at this site. Reference should be made to the latest revision of the 2006 Ontario Building Code for a full discussion of the earthquake design requirements. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m³.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

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

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

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

H = height of the wall (m)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375a_c \gamma H^2/g$ where:

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

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

H = height of the wall (m)

g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.42g according to OBC 2006. Note that the vertical seismic coefficient is assumed to be zero.

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

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

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2006.

5.6 Basement Slab

It is expected that the basement area will be mostly parking and a pavement structure is anticipated. However, if storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the construction and foundation option selected, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

A waterproofing system should be applied below the foundation raft slab (if raft foundation option is selected) and to the exterior of the building foundation walls below the groundwater table in order to prevent water infiltration from the surrounding soils.

Depending on the type of waterproofing membrane used, it can be applied directly to the shoring, or may require a substrate to be applied first. A one-sided prefabricated drainage layer, such as Miradrain G100N, can then be applied continuously over the waterproofing membrane, with the joints between sheets properly lapped, with all geotextile (including laps) against the waterproofing, or directly against the shoring above the waterproofing level.

A waterproofing system should also be applied below the foundation raft slab to prevent water infiltration from the underlying clay. However, it should be noted that this system still has the potential to allow some seepage. Therefore, drainage above the raft slab and below the basement floor will be required to control potential seepage through the foundation raft slab. For design purposes, it is recommended to use 100 or 150 mm diameter perforated pipes, placed at 4 to 6 m centres, connected to a positive outlet (pumped sump pit). The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

The drainage geocomposite material requires positive outlets near the base of the foundation. The geocomposite should start at the footing level and sleeves should be placed through the footing to positively connect the drainage (exterior) side of the membrane to the inside of the footing where a collector drainage pipe should be installed to intercept the flow from the sleeves. It is recommended that 100 to 150 mm diameter sleeves be provided at 3 m maximum spacing.

A perimeter collector pipe is required to intercept the drainage from the sleeves through the footing. This collector pipe can be positively connected to the sleeves, or if a clear stone trench is used around a perforated collector pipe, the drain can be located adjacent to the sleeves, and will collect and remove the drainage water to the outlet.

6.2 Protection of Footings Against Frost Action

Perimeter footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

Other exterior unheated footings, pile caps or grade beams, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Temporary Shoring

Temporary Shoring Requirements

The temporary shoring requirements will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services.

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

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The design of the rock anchors can be based on an allowable grout to rock bond stress of 700 kPa at this site. It is recommended that the upper 2 m of the bedrock be disregarded. A minimum grout strength of 40 MPa is recommended.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 3 - Soil Parameters	
Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m ³	20
Submerged Unit Weight(γ), kN/m ³	13

Soldier Pile and Lagging System

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

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

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

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

If a soldier pile and lagging system is used, the piles could be socketed in the bedrock in pre-augered holes. The augered holes should be advanced at least 2 m into the bedrock and at least 2 m below the bottom of the excavation.

A minimum factor of safety of 1.5 should be used.

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 should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

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

6.6 Winter Construction

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. 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 carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The 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 an aggressive to very aggressive corrosive environment.

7.0 RECOMMENDATIONS

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ☐ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 STATEMENT OF LIMITATIONS

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

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

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

Paterson Group Inc.

David J. Gilbert, P.Eng.



Carlos P. Da Silva, P.Eng.



Report Distribution:

- ☐ Claridge Homes (5 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

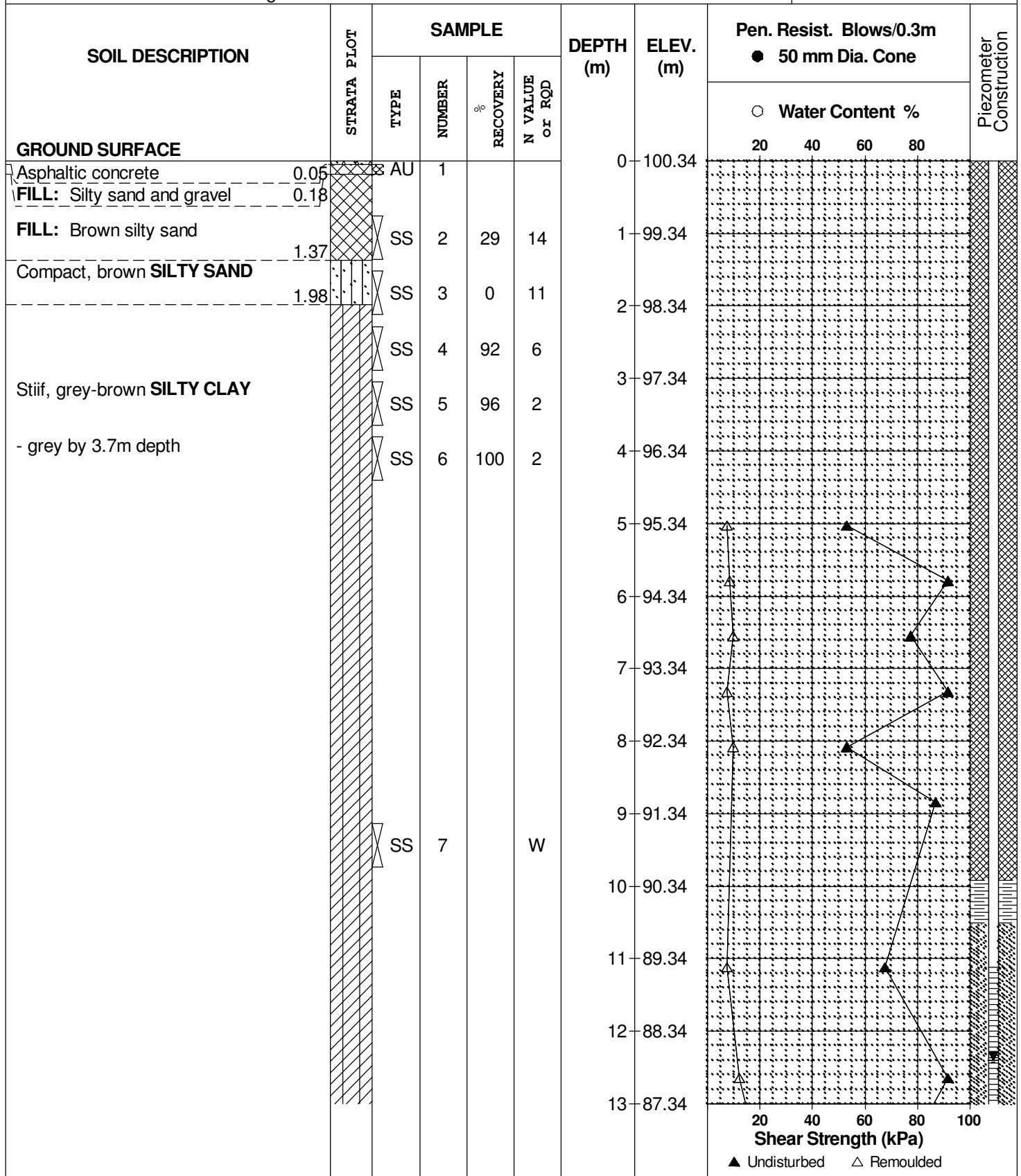
REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 October 2010

FILE NO. PG2232

HOLE NO. BH 1



DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

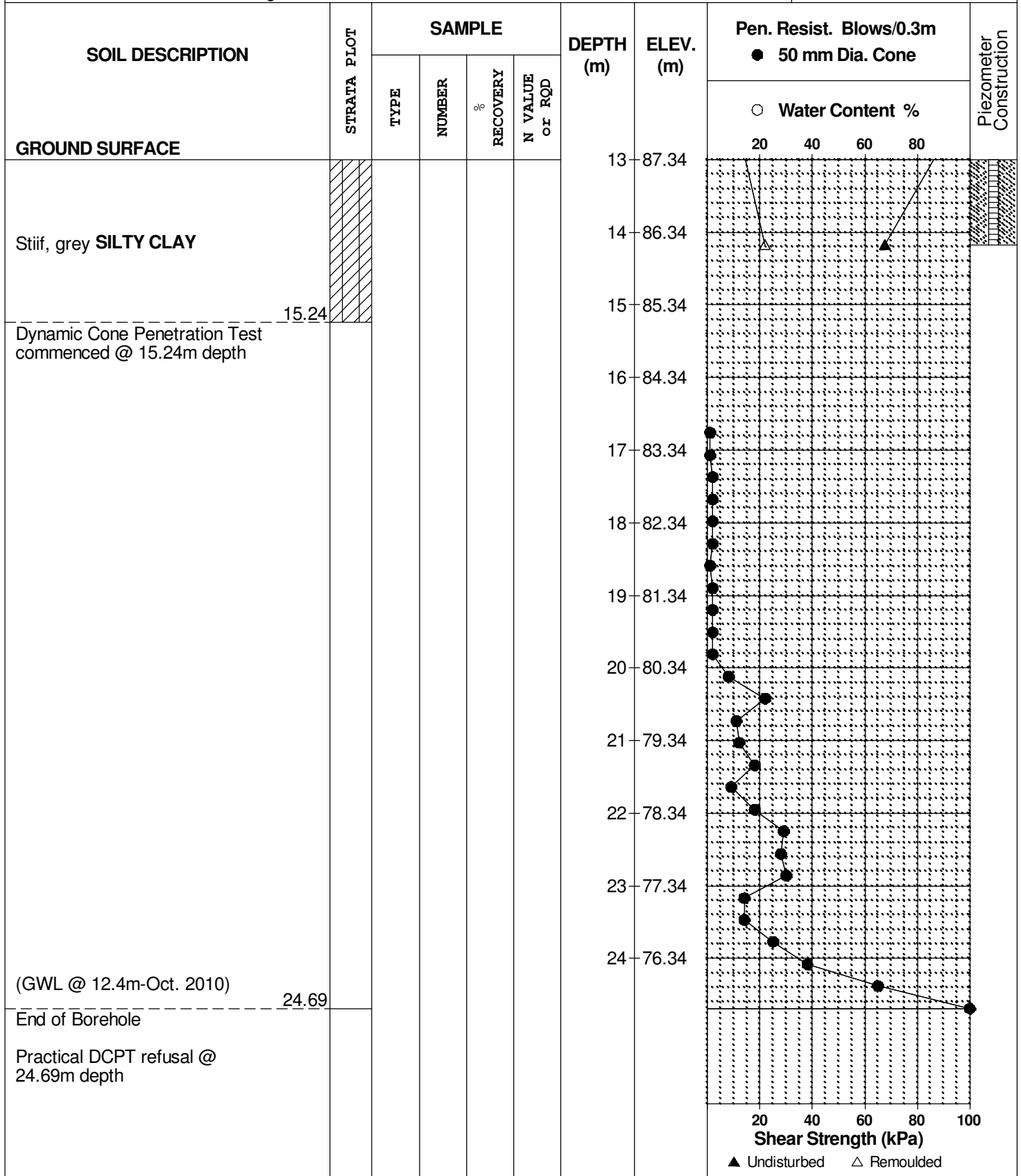
REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 October 2010

FILE NO. PG2232

HOLE NO. BH 1



DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

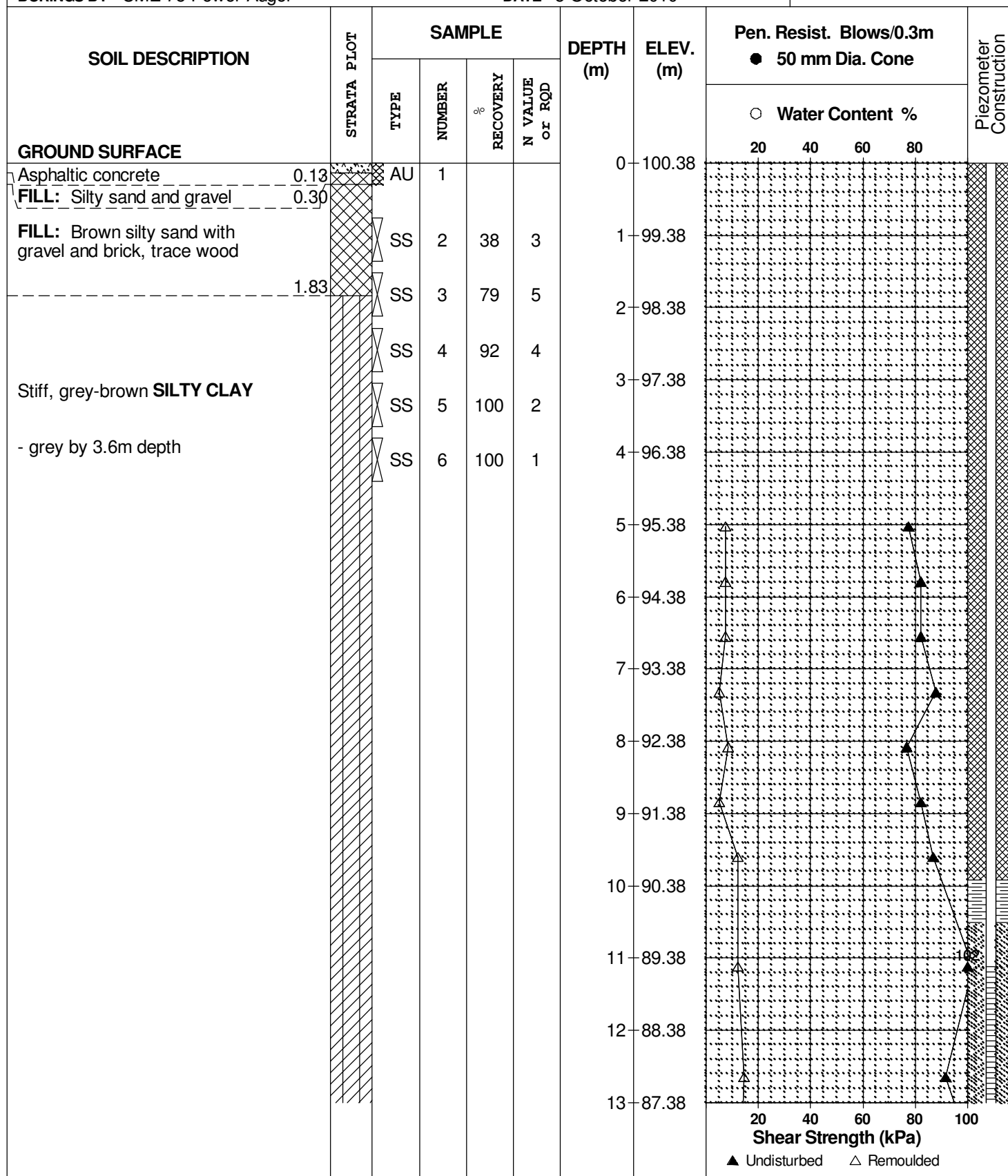
REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 October 2010

FILE NO.
PG2232

HOLE NO.
BH 2



DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

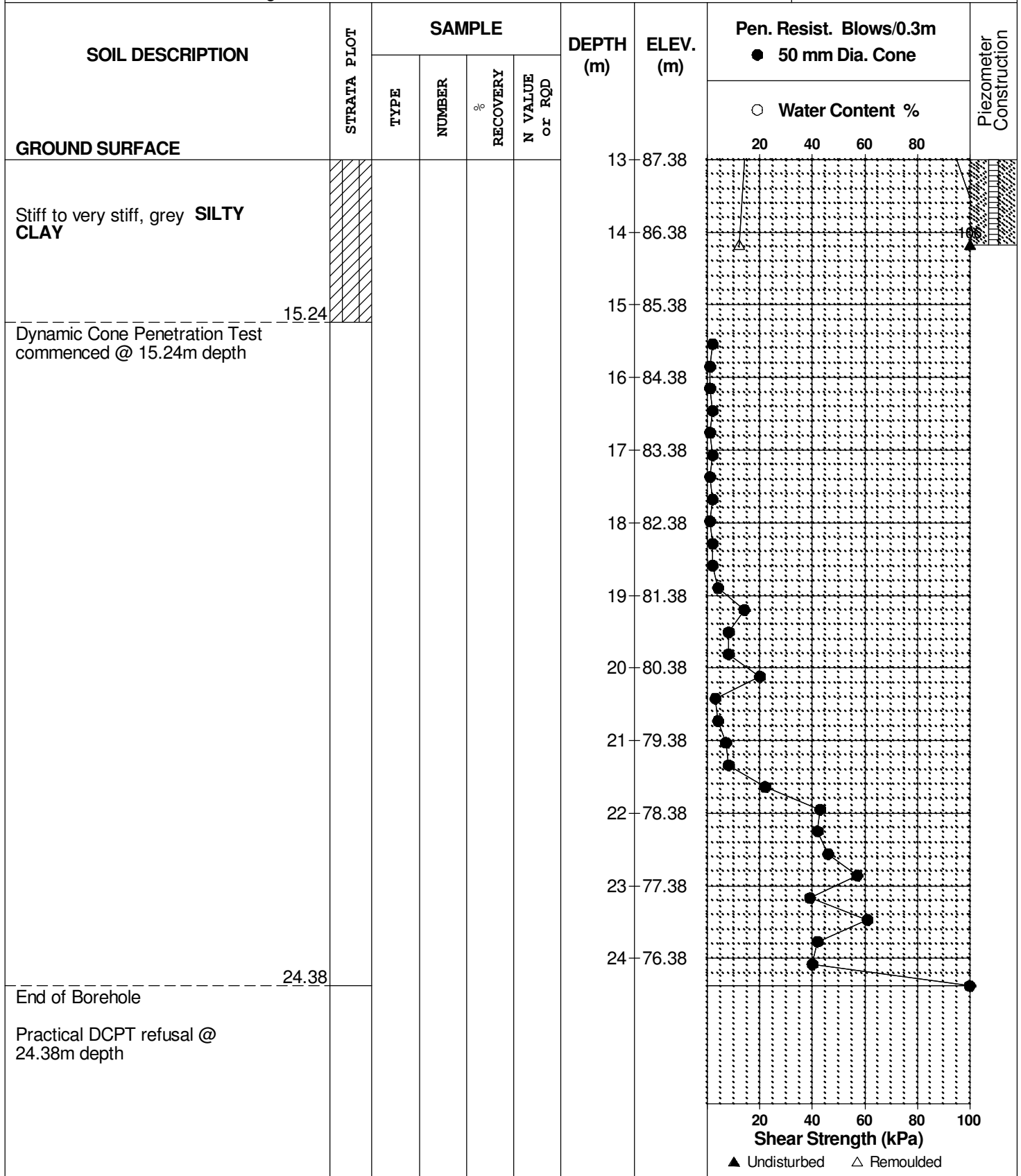
REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 October 2010

FILE NO. PG2232

HOLE NO. BH 2



DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

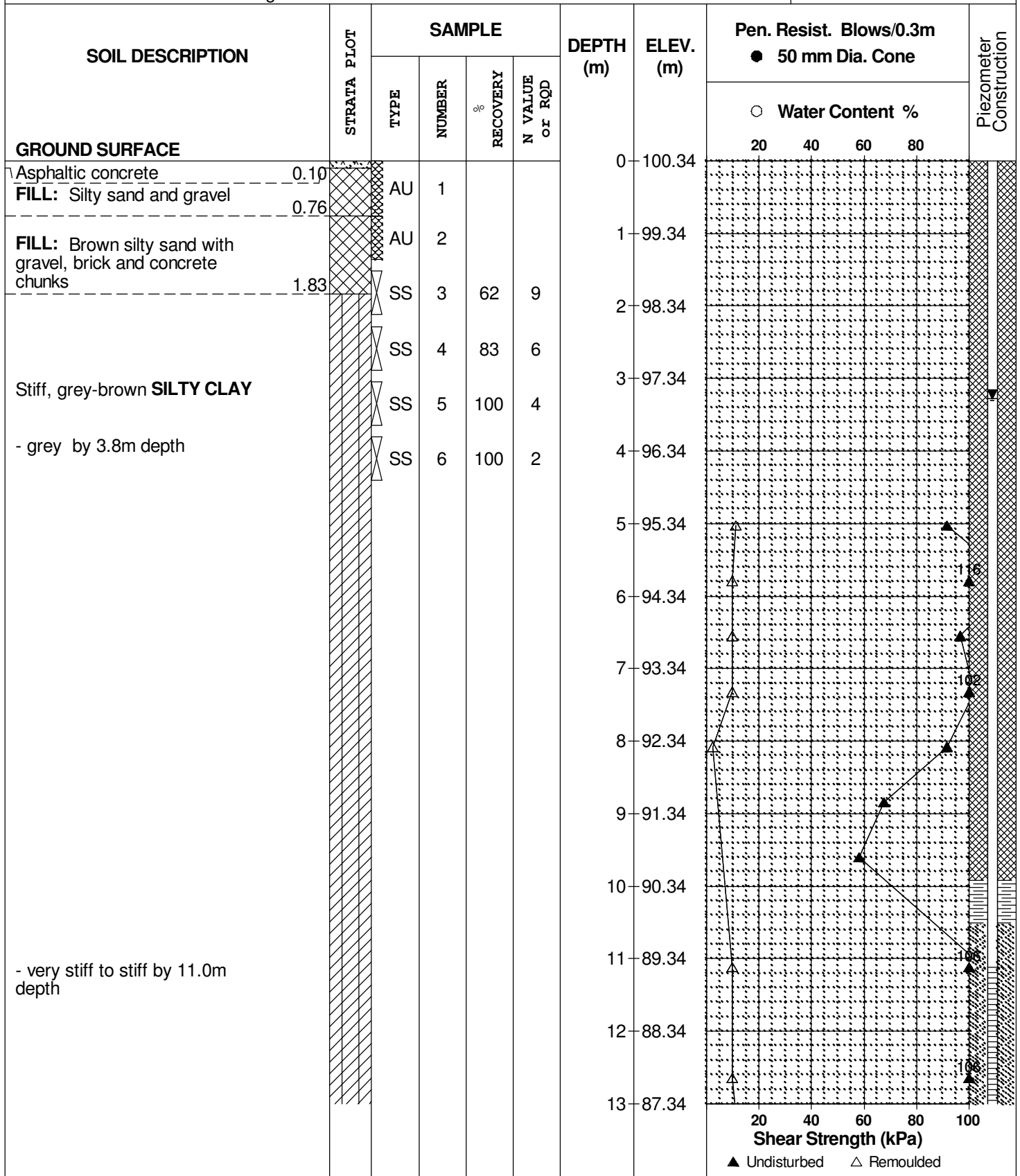
REMARKS

BORINGS BY CME 75 Power Auger

DATE 6 October 2010

FILE NO. PG2232

HOLE NO. BH 3



DATUM TBM - Top of manhole cover, southwest corner of Metcalfe Street and Maclaren Street. Assumed elevation = 100.00m.

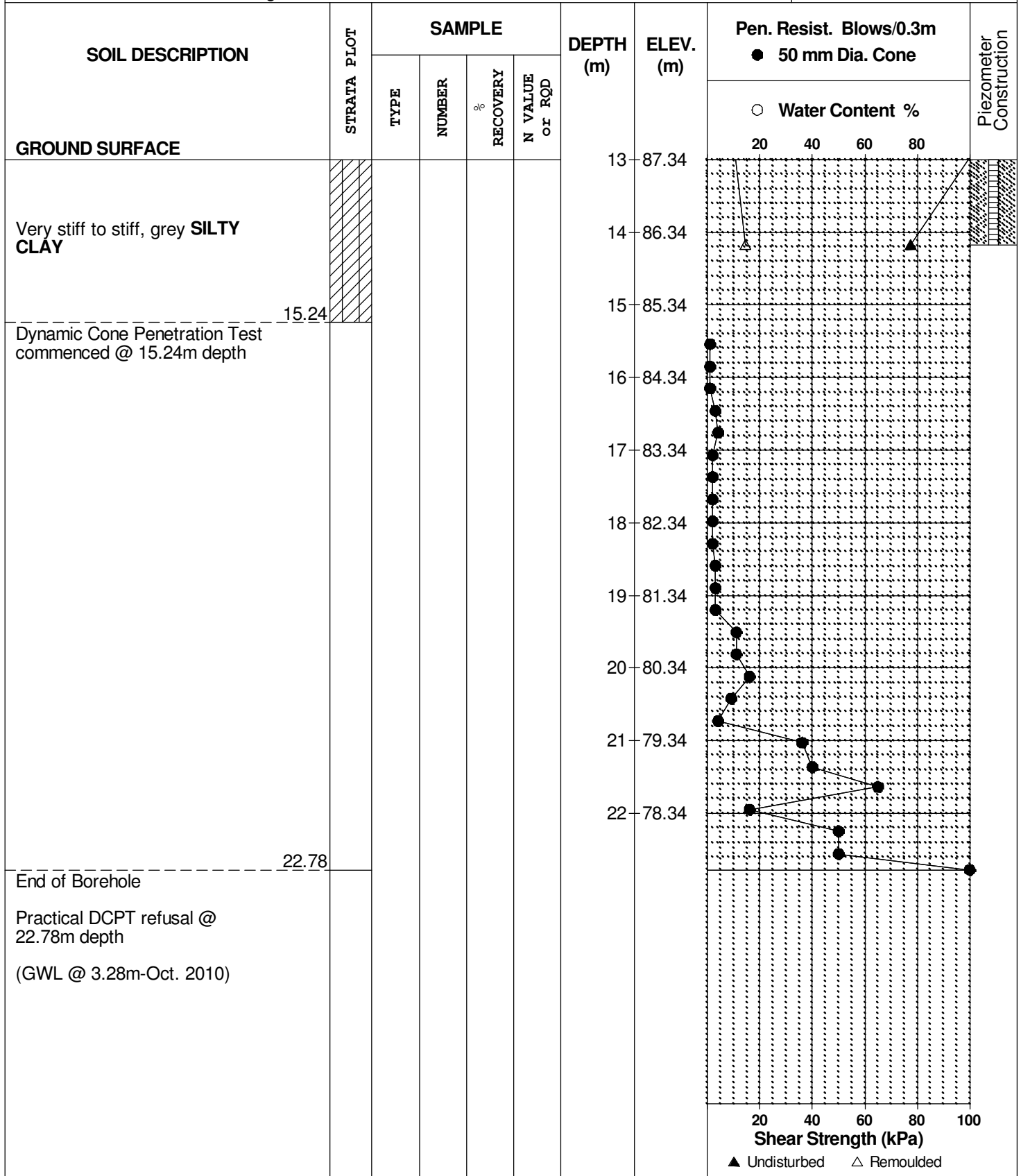
REMARKS

FILE NO. PG2232

HOLE NO. BH 3

BORINGS BY CME 75 Power Auger

DATE 6 October 2010



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

Cc and Cu are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$

Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$

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

Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

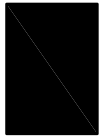
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.
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SYMBOLS AND TERMS (continued)

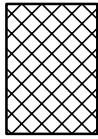
STRATA PLOT



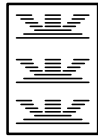
Topsoil



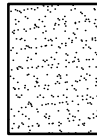
Asphalt



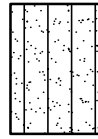
Fill



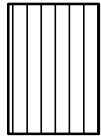
Peat



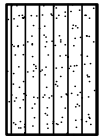
Sand



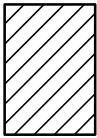
Silty Sand



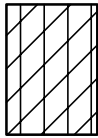
Silt



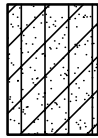
Sandy Silt



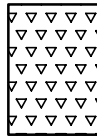
Clay



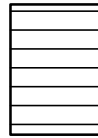
Silty Clay



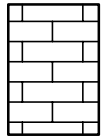
Clayey Silty Sand



Glacial Till



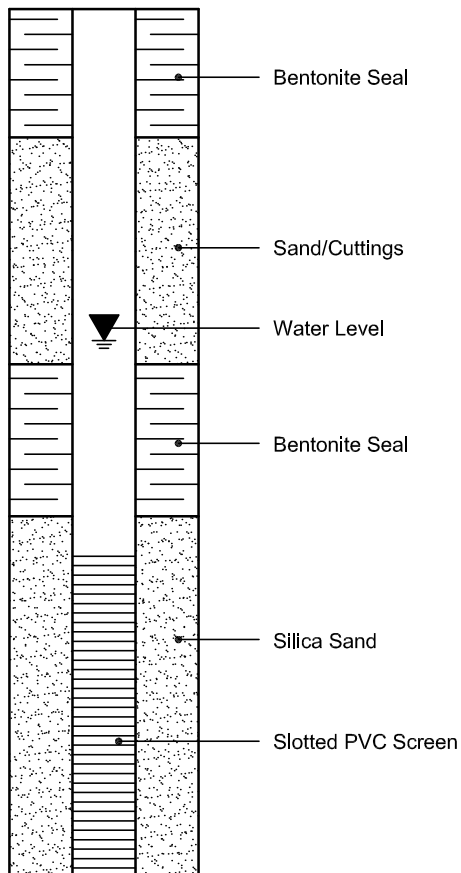
Shale



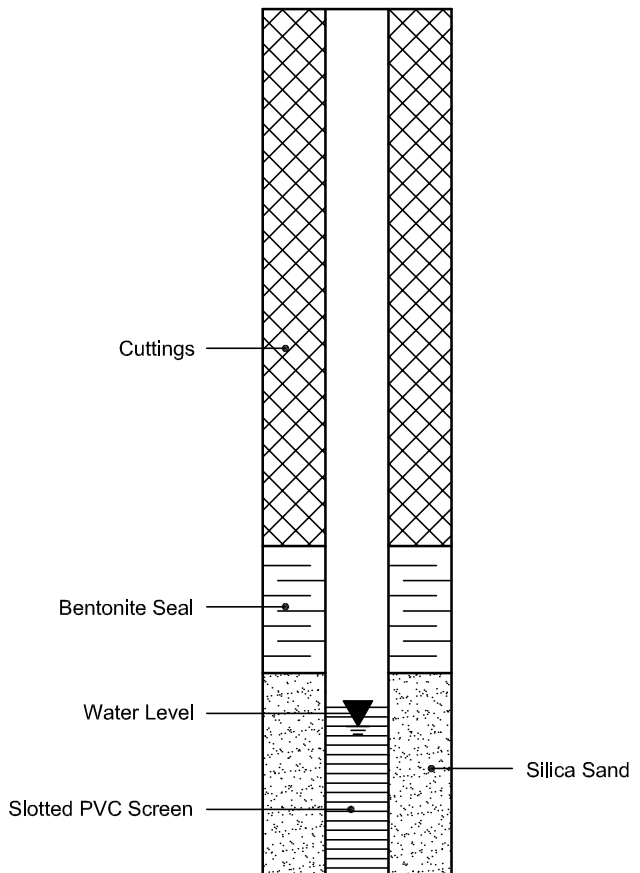
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Report Date: 18-Oct-2010

Client PO: 9465

Project Description: PG2232

Order Date: 12-Oct-2010

Client ID:	BH3-SS6	-	-	-
Sample Date:	06-Oct-10	-	-	-
Sample ID:	1042020-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	57.3	-	-	-
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General Inorganics

pH	0.05 pH Units	8.07	-	-	-
Resistivity	0.10 Ohm.m	8.06	-	-	-

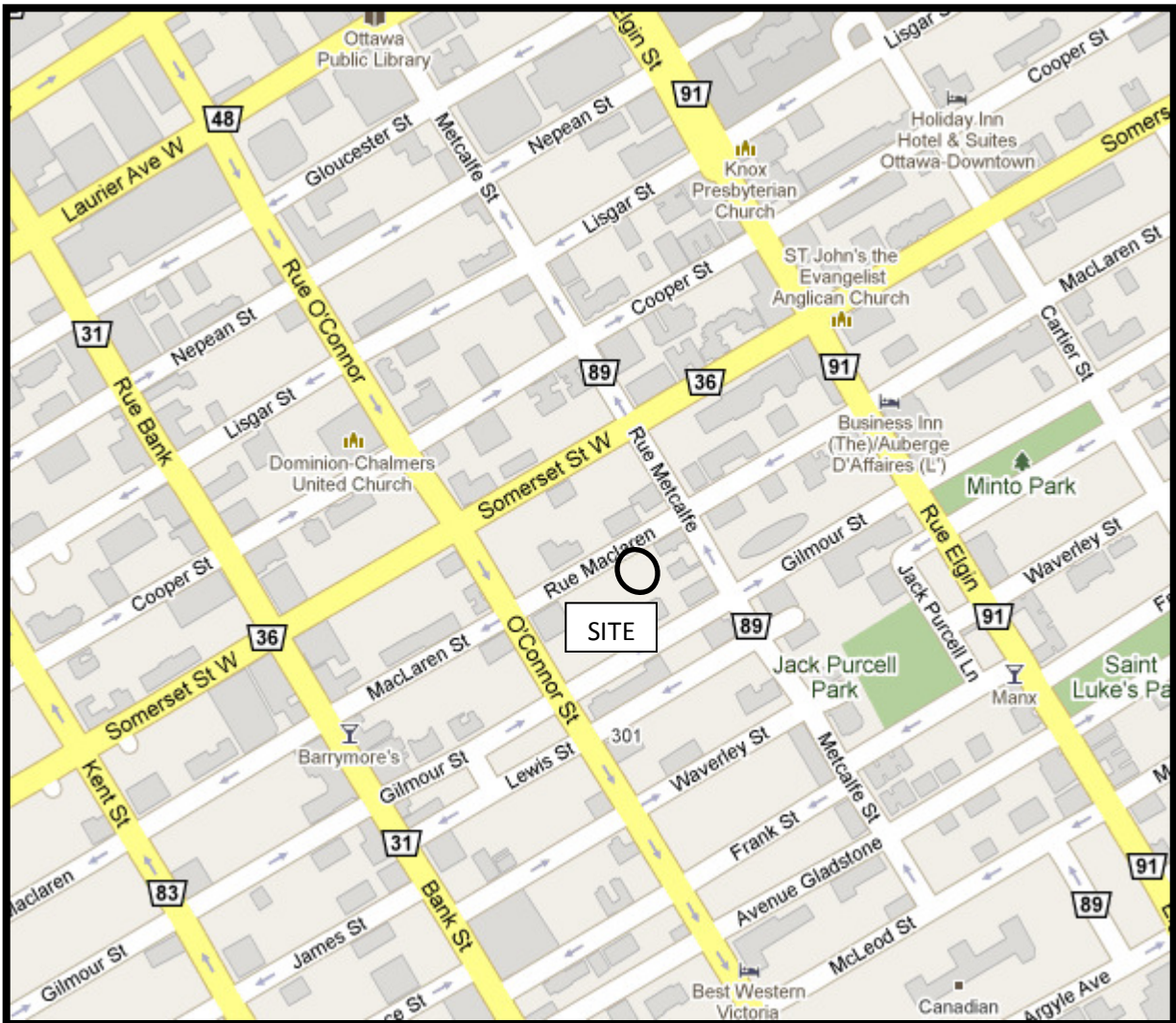
Anions

Chloride	5 ug/g dry	630	-	-	-
Sulphate	5 ug/g dry	1040	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2232-1 - TEST HOLE LOCATION PLAN



Source: Google Maps

FIGURE 1
KEY PLAN

