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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Residential Development
146 Mountshannon Drive
Ottawa, Ontario

Prepared For

Richcraft Homes

March 17, 2010

Report No. PG0267-2

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1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Richcraft Homes (Richcraft) to conduct a geotechnical investigation for the proposed residential development located at 146 Mountshannon Drive in the City of Ottawa (Nepean), Ontario (refer to Figure 1 - Key Plan).

The objectives of the current investigation were to determine the subsoil and groundwater conditions at this site by means of boreholes and to provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation, therefore, the present report does not address environmental issues.

Paterson conducted a Phase I - Environmental Site Assessment (ESA) at this site previous to the current geotechnical investigation. The findings and recommendations of the Phase I ESA are presented under separate cover.

2.0 PROPOSED DEVELOPMENT

The subject site is located in the northeast quadrant of the intersection of Mountshannon Drive and Daventry Crescent. For ease of description, the project north is considered to be parallel to Mountshannon Drive from Daventry Crescent towards Sutcliffe Terrace.

The site is bordered to the north by park areas, to the east by a school and townhouses, to the south by townhouses along Daventry Crescent, and to the west by Mountshannon Drive.

It is our understanding that the proposed residential development will include the construction of twelve (12) blocks of townhouses, with a total of 210 units, including access lanes and parking areas.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the investigation was carried out on June 14, 2004. At that time, six (6) boreholes (BH 1 to BH 6) were advanced to depths ranging from 5.9 to 8.0 m below existing grade. The borehole locations were distributed across the site to provide general coverage of the proposed development. The locations of the boreholes are shown on Drawing PG0267-2 - Test Hole Location Plan included in Appendix 2.

The test holes were put down using a track-mounted power auger drill rig, operated by a crew of two. All fieldwork was conducted under the full-time supervision of a technical staff member 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 and regularly sampling the overburden.

Sampling and In Situ Testing

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

Undrained shear strength testing, using a vane apparatus, was carried out in cohesive soils.

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.

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 the boreholes to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected and determined in the field by Paterson personnel along the proposed lane way to provide general coverage of the site. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), which consists of the top of a spindle of a local fire hydrant located on the east side of Mountshannon Drive, at the southwest corner of the subject site. An arbitrary elevation of 100 m was assigned to the TBM.

The locations of the boreholes and of the TBM as well as the ground surface elevation at each borehole are presented on Drawing PG0267-2 - 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 by a geotechnical engineer to review the results of the field logging.

3.4 Analytical Testing

Two (2) soil samples were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were analysed to determine the concentration of sulphate and chloride, the resistivity and the pH of the samples. The results are shown in Appendix 1 and are discussed further in subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is vacant and generally grass covered. The terrain is slightly undulating. A few small mounds of soil were observed at the southwest, northwest and northeast corner of the subject site. A steel wire fence was also observed in the centre of the property from north to south. There is a drainage ditch along the east property line while fill, consisting of topsoil, sod and construction debris (wood, bricks etc.) was observed along the west property line.

A 0.6 m grade difference was measured between the ground surface elevations at the borehole locations.

4.2 Subsurface Profile

The soil profile at this site consists primarily of topsoil overlying silty clay and/or clayey silt. Glacial till was encountered beneath the above layers at depths varying between 5.5 and 6.2 m. The inferred bedrock surface, based on practical refusal to augering, was encountered at a depth of 8.0 m below the existing ground surface at BH 3. 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.

Topsoil

Topsoil was encountered at ground surface at all borehole locations. The thickness of the topsoil ranges from 150 to 250 mm.

Silty Clay

Brown silty clay was encountered below the topsoil at all borehole locations. The upper portion of the silty clay has been weathered to a brown stiff to very stiff crust. The weathered crust extends to depths varying between 2.2 and 3.5 m at the borehole locations.

In situ shear vane testing carried out in the lower portion of the crust at BH 1 yielded an undrained shear strength value of 115 kPa. This value is indicative of a very stiff consistency.

Grey silty clay exists below the weathered crust at all borehole locations and extends to depths varying between 4.5 and 5.5 m.

In situ shear vane tests carried out in the grey silty clay yielded undrained shear strength values ranging from approximately 35 to 60 kPa. These values are indicative of a firm to stiff consistency. In general, this layer is firm.

Clayey Silt

Clayey silt was encountered below the grey silty clay at all boreholes except at BH 2. The thickness of the clayey silt varies between 1.0 and 1.7 m.

In situ shear vane testing carried out in the clayey silt yielded undrained shear strength values ranging from about 30 to 45 kPa. These values are indicative of a firm consistency.

Glacial Till

Glacial till was encountered below the silty clay or clayey silt at all borehole locations, at depths varying between 5.5 and 6.2 m. The glacial till was not fully penetrated at any of the borehole locations, although practical refusal to augering was encountered in BH 3.

Glacial till consists of a fine soil matrix mixed with gravel, cobbles and boulders. Generally, the fine soil matrix consists of sandy silt. A silty sand matrix was encountered at depth of 6.0 m at BH 3 and 5.5 m at BH 6.

Based on the results of the SPTs, which yield N values ranging from 13 to more than 50 blows per 300 mm penetration, the state of compactness of this layer varies between compact and very dense.

Practical Refusal to Augering

Practical refusal to augering was encountered at BH 3, at a depth of 8.0 m below existing grade. The refusal may be bedrock or a large boulder in the glacial till.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of quartz sandstone, sandy dolostone and dolostone of the March formation, with an overburden thickness at depths varying between 5 and 10 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in all the boreholes on June 28, 2004. The measured groundwater levels ranged from 2.1 to 3.2 m below existing ground surface. The measured GWL readings are presented in Table 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Levels		Date
		Depth (m)	Elevation (m)	
BH 1	99.36	2.82	96.54	June 28, 2004
BH 2	98.78	3.24	95.54	June 28, 2004
BH 3	98.8	2.1	96.7	June 28, 2004
BH 4	98.73	3.1	95.63	June 28, 2004
BH 5	98.86	2.28	96.58	June 28, 2004
BH 6	99.06	3.15	95.91	June 28, 2004

Note: The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), which consists of the top of a spindle of a local fire hydrant located on the east side of Mountshannon Drive, at the southwest corner of the subject site. An arbitrary elevation of 100.00 m was assigned to the TBM.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Geotechnical Assessment

The proposed development will consist of twelve (12) townhouse buildings, a paved private roadway and parking areas. From a geotechnical viewpoint, the subject site is suitable for the proposed development. It is anticipated that the residential dwellings will be founded on shallow footings placed in the upper brown weathered silty clay crust below any fill or topsoil.

Due to the presence of a silty clay deposit, grade raises should be limited to a maximum of 1.0 m at the buildings. Otherwise, an analysis based on consolidation testing should be carried out. The grading plan, once available, should be reviewed by the geotechnical engineer.

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

5.2 Site Grading and Preparation

Stripping Depth

All fill and topsoil should be removed from within the building perimeters. Topsoil and deleterious fill such as those containing organic material should be stripped from under any paved areas and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the buildings and under paved areas (below the base and subbase materials) should consist of clean imported granular fill, such as Ontario Provincial Standard Specification (OPSS) Granular A (crushed stone), Granular B Type I or Type II. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to a minimum of 98% of the material's 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. 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 their respective SPMDD values.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a complete drainage system (such as Miradrain G100N) connected to a perimeter drainage system is provided.

5.3 Foundation Design

Based on the soil profile encountered at this site, it is anticipated that the brown silty clay will be encountered at the expected founding level of the lightly loaded residential dwellings, which is expected to be about 1.5 to no more than 2.0 m below final grade. The silty clay is a suitable bearing strata upon which to found footings for the support of lightly loaded structures.

Bearing Resistance Values

Founding depth of 1.5 m below existing ground surface, or shallower

Using continuously applied loads, strip and pad footings placed on an undisturbed, stiff silty clay bearing surface can be designed using the bearing resistance values at serviceability limit states (SLS) of **100 kPa** for strip footings (up to 1 m wide) and pad footings (up to 1.5 m square) and factored bearing resistance values at ultimate limit states (ULS) of **150 kPa** for strip footings and pad footings. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Founding depth of 2.0 m below existing ground surface

Using continuously applied loads, strip and pad footings placed on an undisturbed, stiff silty clay bearing surface can be designed using the bearing resistance values at SLS of **80 kPa** for strip footings (up to 1 m wide) and pad footings (up to 1.5 m square) and factored bearing resistance values ULS of **125 kPa** for strip footings and pad footings. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values 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.

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 stiff to very stiff silty clay bearing medium 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 soil.

Settlement/Grade Raise

Footings designed using the allowable bearing pressures provided herein will be subjected to potential post construction total and differential settlements of 25 and 15 mm, respectively. A permissible grade raise restriction of **1.0 m** is applicable for fill to be placed on the development site based on the results of the undrained shear tests and subsoil conditions noted at the borehole locations. It should be noted that building on silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking as compared to unreinforced foundations.

If proposed grade raises are above our permissible grade raise recommendations and/or higher loading conditions are required, post construction settlements can be reduced by several methods, such as preloading/surcharge settlement programs or the use of lightweight fill. Once the required grade raises are established, the above options could be further discussed along with further recommendations on specific requirements.

5.4 Design for Earthquakes

The proposed site can be taken as seismic site response **Class D** as defined in the Ontario Building Code 2006 (OBC 2006; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction. A higher site class, such as Class C could be applicable for this site, but would have to be determined based on site-specific shear wave velocity testing.

5.5 Basement Wall

The basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K \cdot \gamma \cdot H$ where:

K = 0.33 - where slight movement is permissible
K = 0.5 - where no movement is permissible (normal basement wall)
γ = 22 kN/m^3, unit weight of the fill
H = height of the basement wall, m

An additional pressure having a magnitude equal to $K \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q, that may be placed at ground surface adjacent to the wall.

5.6 Basement Slab

With the removal of all topsoil and fill containing organic matter within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type I or II are recommended for backfilling below the floor slab (outside the zones of influence of the footings, if applicable). 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 buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.7 Pavement

Car only parking areas and access roadways are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure
Access Roadways

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

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

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

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains at each catch basin. These drains should 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 structures. 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. 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 a drainage blanket (such as Miradrain G100N) connected to a perimeter drainage system is provided. Imported granular materials, such as clean sand or OPSS Granular B Type I material, should be used for this purpose.

A dampproofing system and/or composite drainage system should be applied to the exterior of the building foundation walls in order to reduce the risk of moisture infiltration from the backfill materials.

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent foundation insulation) should be provided for other exterior unheated footings, such as those for isolated piers.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to minimize the trench width, the excavation can be carried out within the confines of a fully braced steel trench box or other acceptable shoring systems.

The subsoil at this site is considered to be mainly a Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

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

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

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. Where the invert of the excavation is below the stiff crust and into the grey silty clay the thickness of the bedding should be increased to 300 mm. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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 reduce the potential 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 clay seals should be as per Standard Drawing No. S8 of the Department of Transportation, Utilities & Public Works Infrastructure Services Branch (TUPWISB) of the City of Ottawa. 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 compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations in the service trenches, at spacings no greater than 60 m.

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 flow of groundwater into the excavation through the overburden materials is expected to be controllable using properly sized pumps and sumps.

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 the presence of water and freezing conditions, ice could form within the soil mass. Heaving upon freezing 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 buildings and/or the footings are protected with sufficient soil cover to prevent freezing at the founding level.

The excavation and backfilling operations should be carried out in such a manner as to avoid the introduction of frozen material into these areas. Additional information can be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content varies between 0.01 and 0.02%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for the subject site. The results of the chloride content, pH and resistivity indicate the presence of a slightly aggressive environment for exposed ferrous metals at this site.

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.

- ❏ 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 Richcraft 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.

Stephen Dunlop, M.Sc.

Carlos P. Da Silva, P. Eng.

Report Distribution:

- ❑ Richcraft Homes (3 copies)
- ❑ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

FILE NO. PG0267

REMARKS

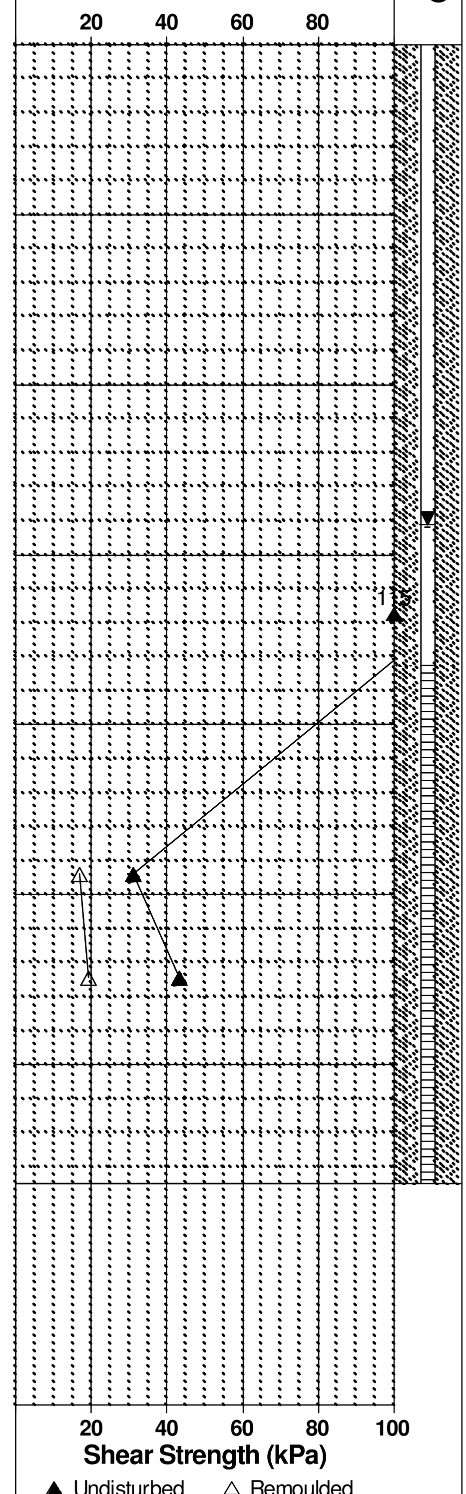
HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.36
TOPSOIL	0.20						
		SS	1	17	15	1	98.36
		SS	2	75	11	2	97.36
Very stiff, brown **SILTY CLAY**		SS	3	100	3	3	96.36
- grey by 3.5m depth							
	4.50	SS	4	71	3	4	95.36
Firm, grey **CLAYEY SILT**		SS	5	83	3	5	94.36
	6.20					6	93.36
GLACIAL TILL: Compact, grey sandy silt with gravel, cobbles and boulders	6.70	SS	6	38	13		
End of Borehole							
(GWL @ 2.82m-June 28/04)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %

Piezometer Construction



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

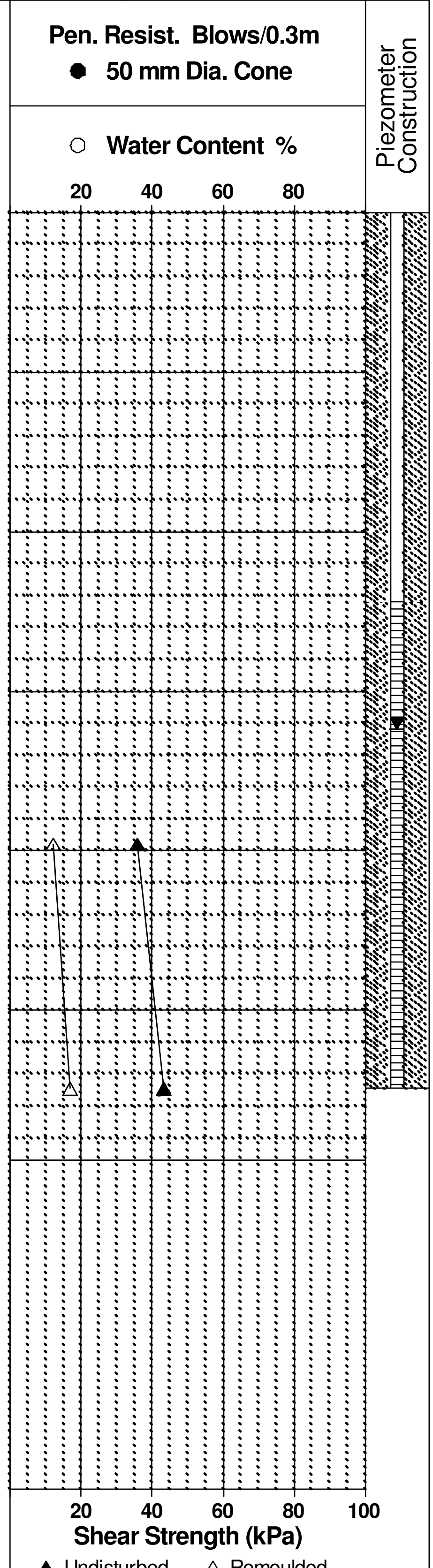
FILE NO. PG0267

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.78
TOPSOIL (0.20)							
		SS	1	67	14	1	97.78
Stiff, brown **SILTY CLAY**							
		SS	2	71	7	2	96.78
- firm and grey by 2.3m depth							
		SS	3	100	2	3	95.78
		SS	4	100	1	4	94.78
		SS	5	88	1	5	93.78
(5.50)							
GLACIAL TILL: Compact, grey sandy silt with gravel, cobbles and boulders (5.94)		SS	6	13	17		
End of Borehole							
(GWL @ 3.24m-June 28/04)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

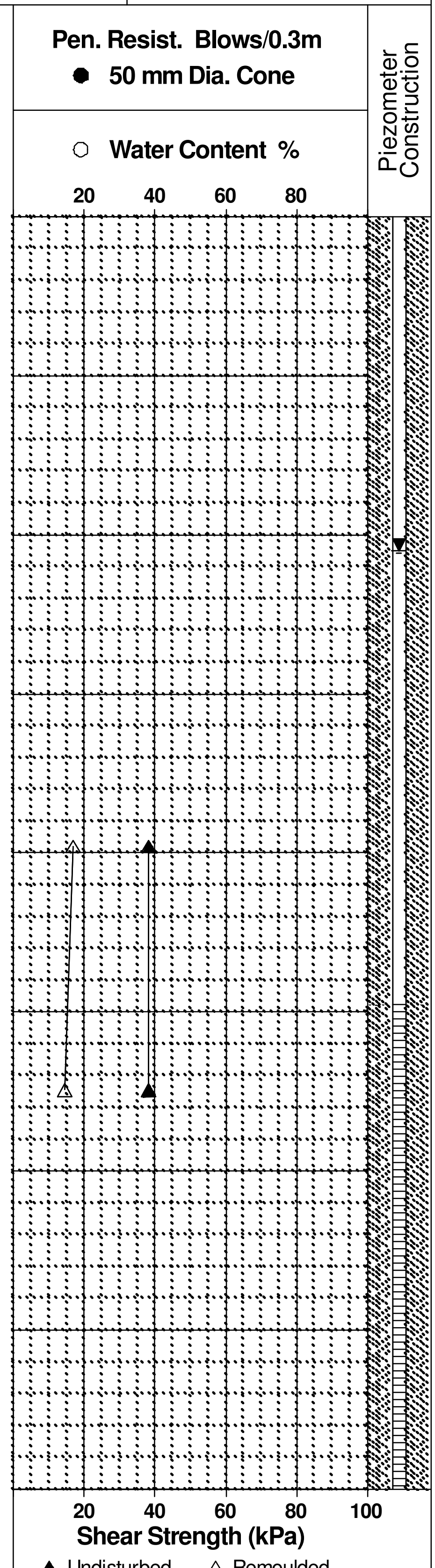
FILE NO. PG0267

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.80
TOPSOIL	0.23						
Stiff, brown **SILTY CLAY** - brown-grey by 2.2m depth - firm and grey by 3.0m depth		SS	1	71	13	1	97.80
		SS	2	83	6	2	96.80
		SS	3	92	1	3	95.80
	4.50	SS	4	100	1	4	94.80
Firm, grey **CLAYEY SILT**	5.54	SS	5	75	1	5	93.80
GLACIAL TILL: Compact, grey sandy silt with gravel, cobbles and boulders	6.00	SS	6	100	3	6	92.80
GLACIAL TILL: Compact, grey silty fine sand, occasional gravel and cobbles		SS	7	79	26	7	91.80
	8.00	SS	8	100	59+	8	90.80



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

FILE NO. PG0267

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
End of Borehole Practical refusal to augering @ 8.00m depth (GWL @ 2.10m-June 28/04)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

FILE NO. **PG0267**

REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

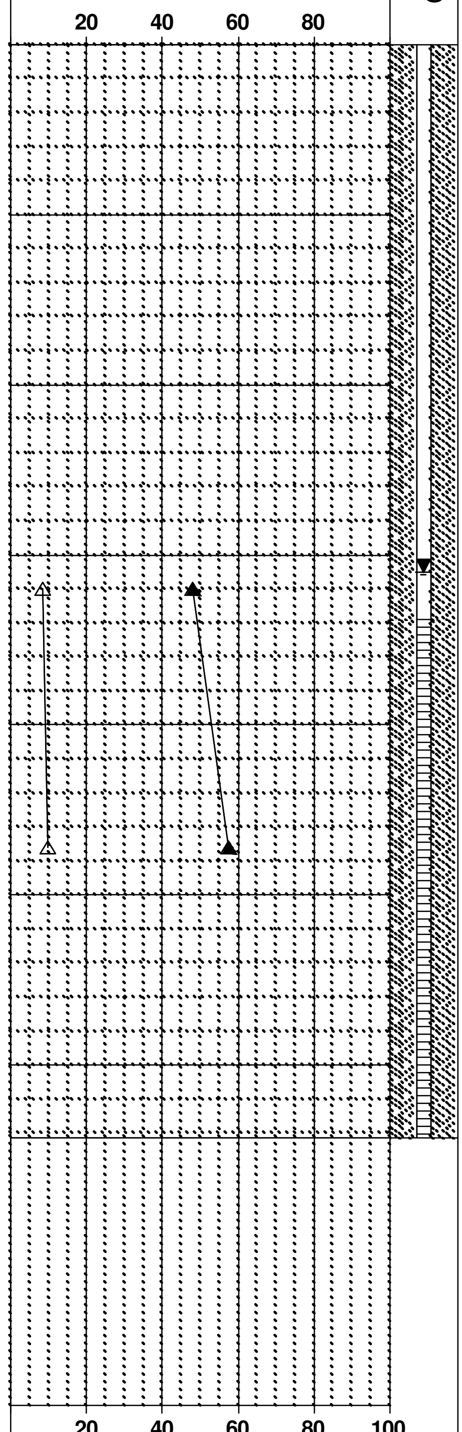
SOIL DESCRIPTION	Depth of layer (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.73
TOPSOIL	0.15						
Stiff, brown **SILTY CLAY**		SS	1	62	19	1	97.73
		SS	2	75	7	2	96.73
		SS	3	100	3	3	95.73
- firm and grey by 3.2m depth		SS	4	62	1	4	94.73
	5.00					5	93.73
Firm to stiff, grey **CLAYEY SILT**		SS	5	75	5	6	92.73
	6.10						
GLACIAL TILL: Loose, grey sandy silt with gravel, cobbles and boulders	6.43	SS	6	31	50+		

End of Borehole

(GWL @ 3.10m-June 28/04)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

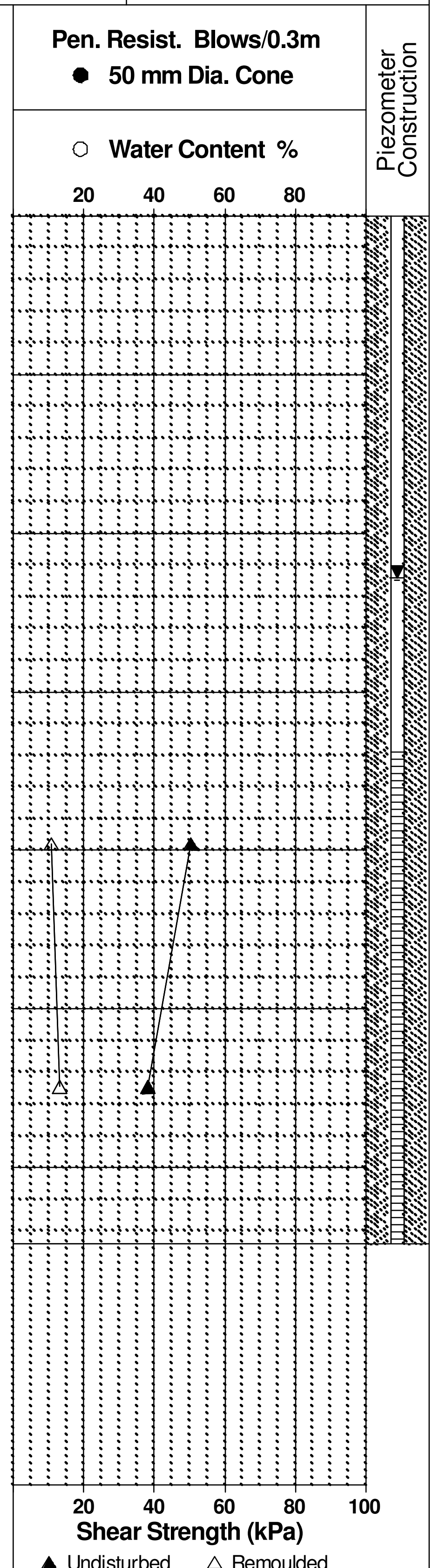
DATE 14 Jun 04

FILE NO. PG0267

HOLE NO. BH 5

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.18				
Stiff, brown **SILTY CLAY** - brown-grey by 3.0m depth - firm and grey by 3.8m depth	4.50	SS	1	50	11
		SS	2	71	8
		SS	3	100	3
		SS	4	100	1
Firm, grey **CLAYEY SILT**	6.10	SS	5	75	2
GLACIAL TILL: Very loose, grey sandy silt with gravel, cobbles and boulders	6.48	SS	6	47	52+
End of Borehole (GWL @ 2.28m-June 28/04)					

DEPTH (m)	ELEV. (m)
0	98.86
1	97.86
2	96.86
3	95.86
4	94.86
5	93.86
6	92.86



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development, Mountshannon Drive
Ottawa (Nepean), Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Assumed elevation = 100.00m.

FILE NO. PG0267

REMARKS

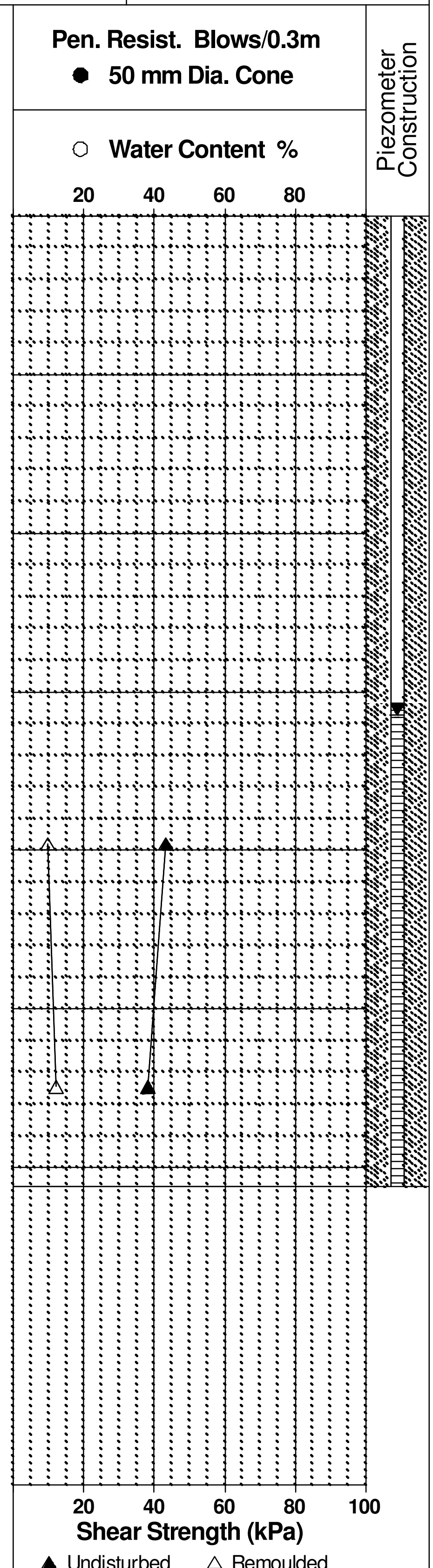
HOLE NO. BH 6

BORINGS BY CME 55 Power Auger **DATE** 14 Jun 04

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.25
Stiff, grey-brown **SILTY CLAY** - brown-grey by 3.0m depth - firm and grey by 4.0m depth	4.50
Firm, grey **CLAYEY SILT**	5.51
GLACIAL TILL: Loose, grey silty fine sand with gravel, cobbles and boulders	6.12
End of Borehole (GWL @ 3.15m-June 28/04)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	62	13
SS	2	54	6
SS	3	100	3
SS	4	100	2
SS	5	75	3
SS	6	29	53

DEPTH (m)	ELEV. (m)
0	99.06
1	98.06
2	97.06
3	96.06
4	95.06
5	94.06
6	93.06



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 = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

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

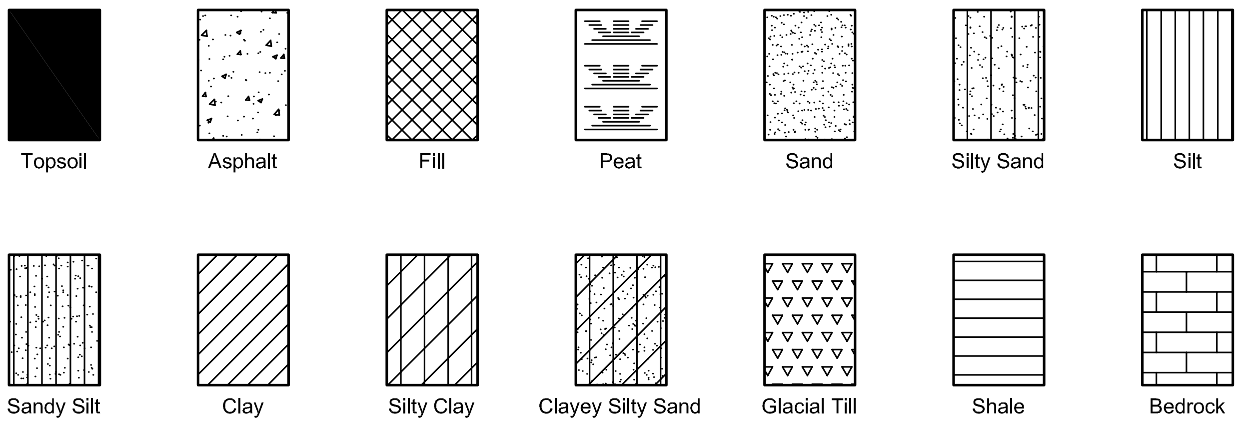
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

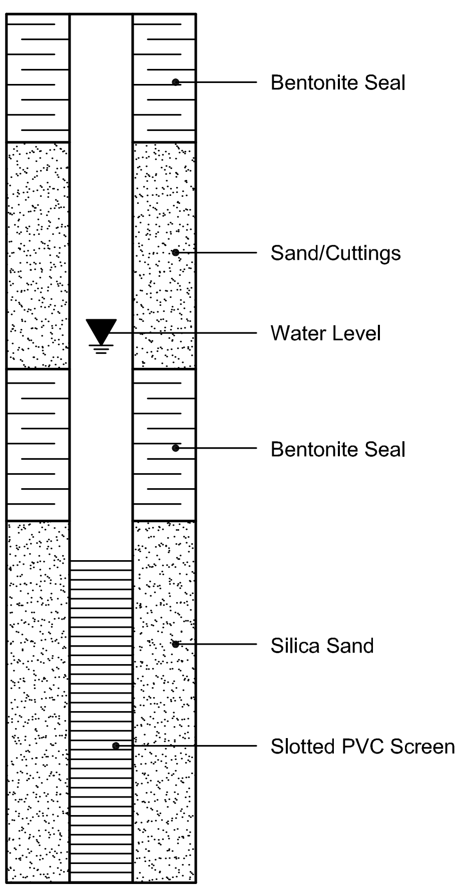
SYMBOLS AND TERMS (continued)

STRATA PLOT

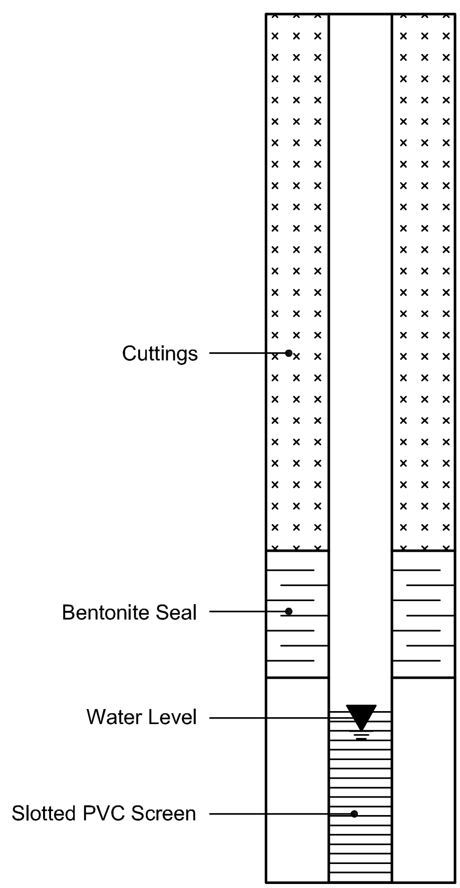


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



ACCUTEST LABORATORIES LTD

REPORT OF ANALYSIS

Client: Paterson Group
28 Concourse Gate, Unit 1
Nepean, ON
K2E 7T7
Attention: Glenn Collins

Report Number: 2411078
Date: 2004-06-23
Date Submitted: 2004-06-15

Project: PG0267

P.O. Number:
Matrix: Soil

		LAB ID:	324749	324750				GUIDELINE		
		Sample Date:	2004-06-14	2004-06-14						
		Sample ID:	BH2-SS2	BH4-SS2						
PARAMETER	UNITS	MDL						TYPE	LIMIT	UNITS
Chloride	%	0.001	0.005	0.004						
Electrical Conductivity	mS/cm	0.01	0.16	0.15						
pH			7.3	7.3						
Resistivity	ohm-cm	1	6250	6670						
Sulphate	%	0.01	0.02	0.01						

MDL = Method Detection Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

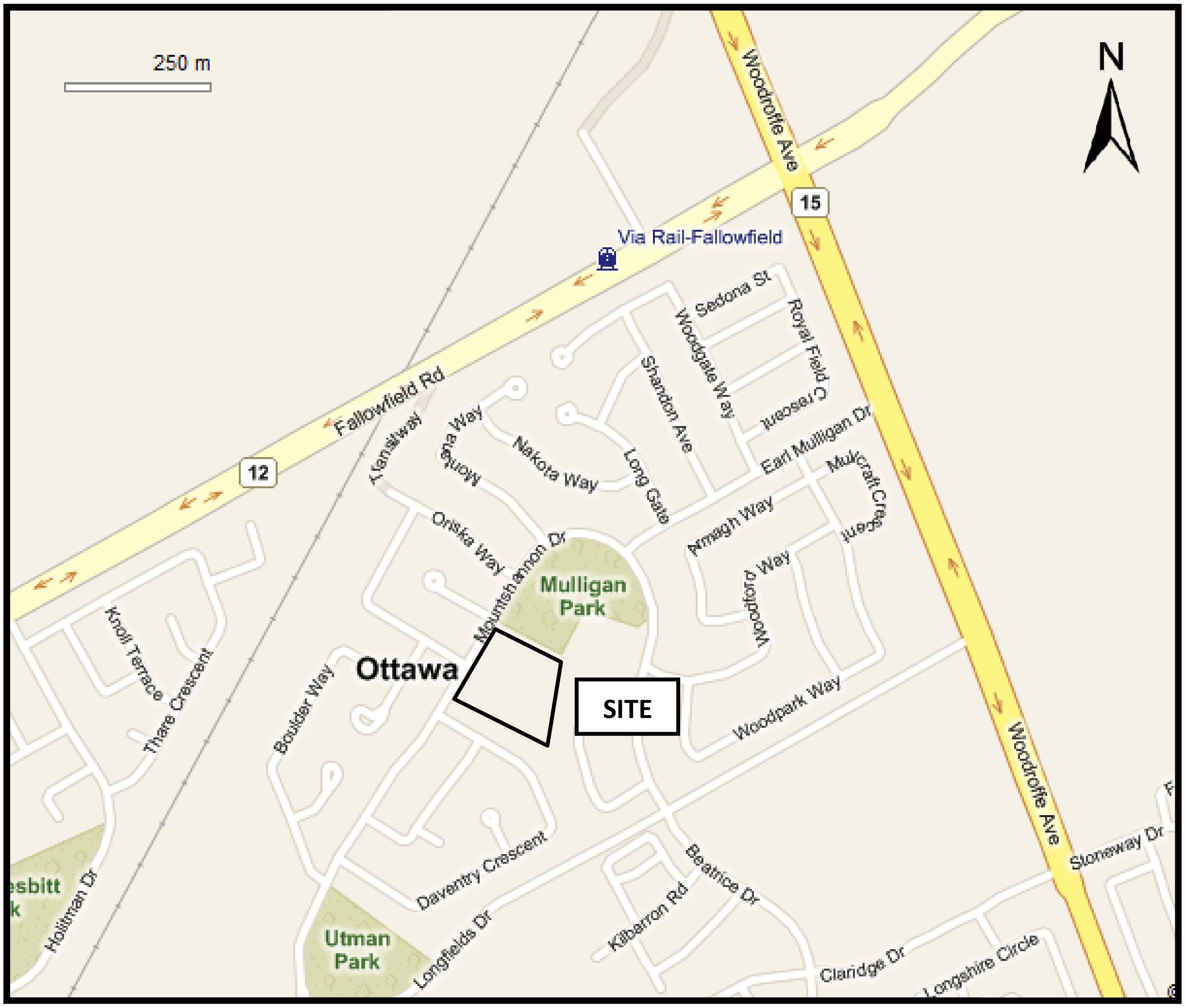
Comment:

APPROVAL:
Lorna Wilson
Agriculture Lab Supervisor

APPENDIX 2

FIGURE 1 - KEY PLAN

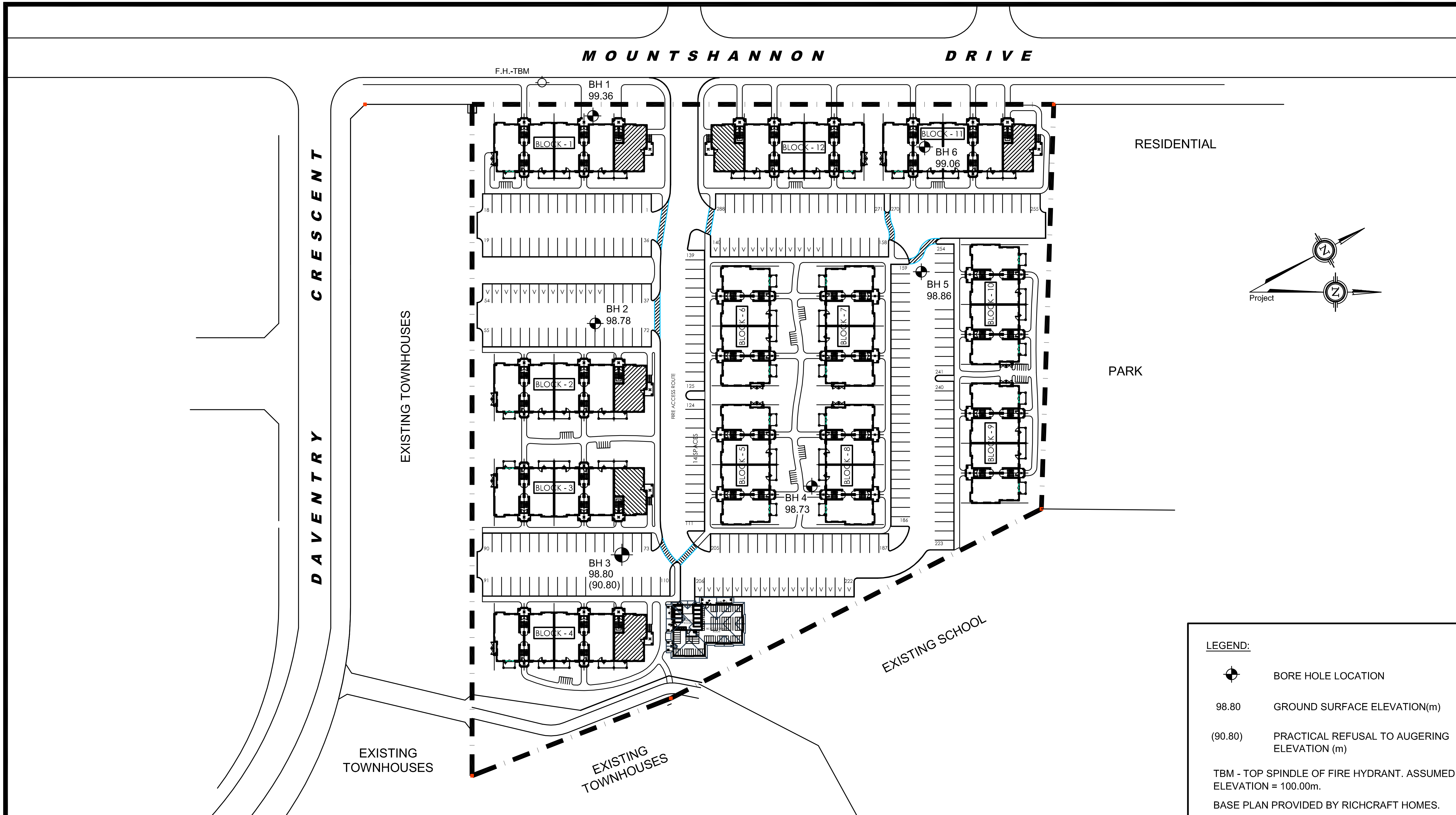
DRAWING PG0267-2 - TEST HOLE LOCATION PLAN



Source: Bing Maps

FIGURE 1

KEY PLAN



LEGEND:

(symbol)	BORE HOLE LOCATION
98.80	GROUND SURFACE ELEVATION(m)
(90.80)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT. ASSUMED ELEVATION = 100.00m.

BASE PLAN PROVIDED BY RICHCRAFT HOMES.

paterson group
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

Scale:	1:1000
Des.:	SD
Dwn:	MPG
Chkd:	CDS

RICHCRAFT HOMES

GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT, MOUNTSHANNON DRIVE

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG0267-2
Report No.:	PG0267-2
Date:	03/2010