patersongroup

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Part 6:
Geotechnical Investigation

Westboro Collection
319 and 320 McRae Avenue
Ottawa, Ontario

Prepared For

914168 Ontario Inc.

Paterson Group Inc.
Consulting Engineers
28 Concourse Gate - Unit 1
Ottawa (Nepean), Ontario
Canada K2E 7T7

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

November 20, 2008

Report: PG1769-1

TABLE OF CONTENTS

APPENDICES

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG1769-1 - Test Hole Location Plan

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by 914168 Ontario Inc. to conduct a geotechnical investigation for proposed developments to be located at 319 and 320 McRae Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current 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 developments as they are understood at the time of writing this report.

Paterson has completed a limited environmental program for the subject site. The findings and recommendations of the environmental program are presented under separate cover.

2.0 PROPOSED PROJECT

Details of the proposed developments were unknown at the time of this investigation. It is understood that residential and/or commercial facilities are proposed to be constructed at the subject site. For preliminary purposes, it is expected that a combination of mixed use buildings, townhouses and a high rise building will be constructed at the subject site.

In general residential and commercial properties border the subject sites. A hydro easement border the westerly boundary of 310 McRae Avenue.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the investigation was carried out on October 27 and 28, 2008. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The approximate locations of the boreholes are shown on Drawing PG1769 -1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and 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 samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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.

Rock samples were recovered from BHs 1, 4, 5, 6 and 7 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to our laboratory. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Four (4) flexible polyethylene standpipes and two (2) monitoring wells were installed at borehole locations, except BHs 3 and 8, to permit monitoring of the 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 determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to the top spindle of the fire hydrant located at the south west corner of McRae Avenue and Scott Street. An elevation of 64.445 m was indicated for the top spindle on survey plan provided by the client. The location and ground surface elevations of the boreholes and the benchmark are presented on Drawing PG1769 -1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and rock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. The testing results are presented in Subsection 4.3.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

Several building structures and garages are currently occupying the subject site with corresponding gravel and asphalt parking areas. The site is relatively flat and at grade with surrounding streets and properties. A hydro easement is located along the west property boundary of 310 McRae Avenue.

4.2 Subsurface Profile

Generally, the soil profile of the site consists of crushed stone fill or asphaltic concrete pavement structure at surface. The abovenoted layers are underlain by a native glacial till and/or bedrock. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Bedrock, consisting of grey limestone, was cored at BHs 1, 4, 5, 6 and 7 to a maximum depth of 7.8 m. The recovery values and RQD values for the bedrock cores were calculated. The recovery values range from 80 to 100%, while the RQD values vary between 27 and 100%. Based on these results the quality of the bedrock poor to excellent.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone and dolostone of the Gull River Formation.

4.3 Groundwater

Four (4) flexible polyethylene standpipes and two (2) monitoring wells were installed at borehole locations, except BHs 3 and 8. The groundwater (GWL) readings are presented in Table 1. It should be noted that the groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	64.61	2.48	62.13	November 3, 2008
BH 2	64.34	1.76	62.58	November 3, 2008
BH 4	63.42	2.95	60.47	November 3, 2008
BH 5	64.27	Damaged	N/A	November 3, 2008
BH 6	63.16	4.85	58.31	November 3, 2008
BH 7	63.61	3.72	59.89	November 3, 2008

Note: The ground surface elevations at the borehole locations were referenced to a benchmark (BM), consisting of the top spindle of the fire hydrant located on the south west corner of McRae Avenue and Scott Street. An elevation of 64.445 m was given based on “Registered Plan No. 263" provided by the client.

5.0 DISCUSSION

5.1 Geotechnical Assessment

It is understood that residential and/or commercial facilities are proposed to be constructed at the subject site and may consist of mixed use buildings, townhouses and a high rise building. No further details were known at the time of this investigations.

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated that the proposed facilities will be founded on conventional spread footings placed on bedrock or dense glacial till. The preference will be to found the footings on bedrock. However, lighter structures can be founded on a combination of glacial till and bedrock.

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 perimeter. The excavation will be extended to the native soils or bedrock.

Fill Placement

Fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, it is expected that limestone bedrock or glacial till will be encountered at the founding levels.

Footings placed on a clean, surface sounded limestone/dolostone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **3,000 kPa** could be used if founded on limestone/dolostone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

For footings placed on an undisturbed, compact to dense glacial till, a factored bearing resistance value at ULS of **225 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at SLS of **150 kPa** can be used.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements. Foundations on undisturbed, glacial till may yield settlements up to 25 and 20 mm total and differential settlements, respectively.

5.4 Design for Earthquakes

The proposed site can be taken as seismic site response Class C 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 A could be applicable for this site and would require confirmation using site specific seismic testing.

5.5 Basement Slab

The glacial till and bedrock is an acceptable founding medium for the basement floor slab. It is expected that the basement area for the building will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used, 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 and the anticipated founding level, 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.

5.6 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 19 kN/m^3. The applicable effective unit weight of the material is 13 kN/m^3, where applicable.

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether it is a yielding or an unyielding structure. A basement wall is considered to be an unyielding structure.

Unyielding walls, such as the basement walls of the proposed structure, are considered to be subjected to “at rest” earth pressures will not deflect enough to allow for the development of “active” earth pressures. It is recommended that the at-rest earth pressure case be used for basement walls.

The total earth force includes both the static earth force component and the seismic component.

Static Earth Pressures

Under static conditions, 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 $K_o\gamma H$ where:

K_o = At-rest earth pressure coefficient, where no movement is permissible = 0.50 (normal basement wall)
γ = unit weight of the fill = 19 kN/m^3
H = height of the basement wall (m)

The static (horizontal) earth force can be taken as equal to $K_o\gamma H^2/2$. The static component is a conventional triangular shaped pressure distribution with the resultant located H/3 up from the wall base.

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

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

Seismic loading conditions influence the earth pressures that will act on earth retaining structures during seismic events. In Ottawa, the peak ground acceleration (PGA) is 0.42g for the OBC 2006.

The magnitude of seismic earth pressures acting on a structure are dependent upon the relative flexibility of the structure. Basement foundation walls braced at both top and bottom are generally considered to be “unyielding”, and do not move sufficiently to mobilize the shear strength of the backfill soil. As a result, the minimum active or maximum passive earth pressures cannot be developed.

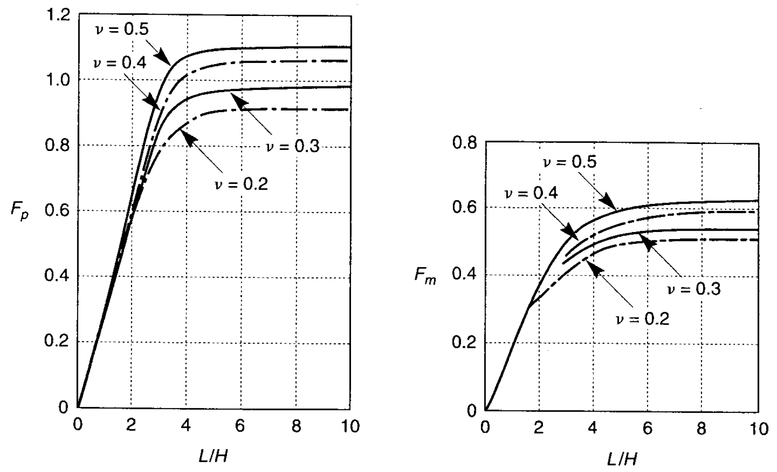


Figure 1 - Dimensionless Thrust and Moment Factors for Seismic Earth Pressures - Wood (1973)

For the case of constant horizontal acceleration, the dynamic thrust (ΔP_{eq}) and dynamic overturning moment (ΔM_{eq}) (about the base of the wall) are defined as:

$$\Delta P_{eq} = \gamma H^2 (a_h/g) F_p$$

$$\Delta M_{eq} = \gamma H^3 (a_h/g) F_m$$

where: γ = unit weight of the fill = 19 kN/m^3
H = height of the basement wall (m)
a_h = Peak ground acceleration, 0.42g (for Ottawa area)
g = Gravity constant, 9.81 m/s^2
F_p = Dimensionless Thrust Factor, defined in Figure 1 below.
F_m= Dimensionless Moment Factor, defined in Figure 1 below.
L = 4
ν = Poisson's ratio, use 0.3 for sand or sound, limestone bedrock and 0.5 for fully saturated, silty clay.

The point of application of the dynamic thrust is at height, $h_{eq} = \Delta P_{eq}/\Delta M_{eq}$, above the base of the wall.

5.7 Pavement Structure

Pavement Design

Car only parking areas and access lanes 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 Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 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.

If the existing pavement structure is to be reinstated after the construction of the proposed commercial building, the following guidelines should be adhered to during pavement reinstatement.

As a general guideline, the pavement structure should be reinstated by matching the new pavement layers to the existing ones. Stepped joints should be provided in the asphaltic concrete layers to provide more resistance to reflective cracking at the joint. Care should also be taken to reinstate the subgrade by matching the existing subgrade to minimize the potential for differential frost heaving.

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.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

For the shallow foundations expected at the site with no basement levels foundation drainage will not be required.

For consideration of foundations with one basement level it is recommended that the composite drainage system (such as Miradrain G100N or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures to be placed on glacial till or other frost susceptible soils are required to be insulated against the deleterious effects 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.

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

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For preliminary design purposes it is expected that sufficient room will be available to permit excavations to be undertaken by open-cut methods (i.e. unsupported excavations). Exception may apply depending on the buildings layout and coverage.

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 Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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 (5th edition, March 31, 2006). Trench details should be as per Drawing Nos. W17, S6 and S7.

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

At least 150 mm of OPSS Granular A should be used for bedding for water pipes. The bedding material, which should extend to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular M. 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.

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 for the type of soil encountered at the 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 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOE.

6.6 Winter Construction

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 corrosive environment.

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 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 and follow-up 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 recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 914168 Ontario Inc. 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.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Nov.20/08
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng.

Report Distribution:

- ☐ 914168 Ontario Inc. (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 27 Oct 08

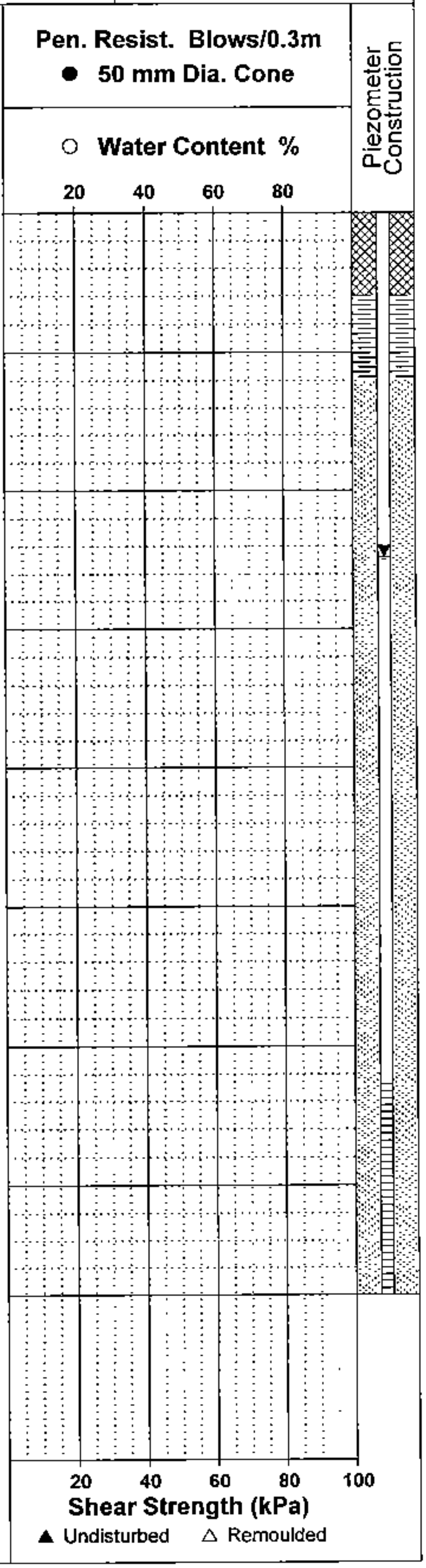
FILE NO. PG1769

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone	0.20
FILL: Sand and gravel	0.91
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders	1.97
BEDROCK: Grey limestone	7.80
End of Borehole	
(GWL @ 2.48m-Nov. 3/08)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	3		
SS	1	27	45
SS	2	33	50+
RC	1	100	0
RC	2	100	27
RC	3	95	85
RC	4	99	96
RC	5	100	100

DEPTH (m)	ELEV. (m)
0	64.61
1	63.61
2	62.61
3	61.61
4	60.61
5	59.61
6	58.61
7	57.61



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 27 Oct 08

FILE NO. PG1769

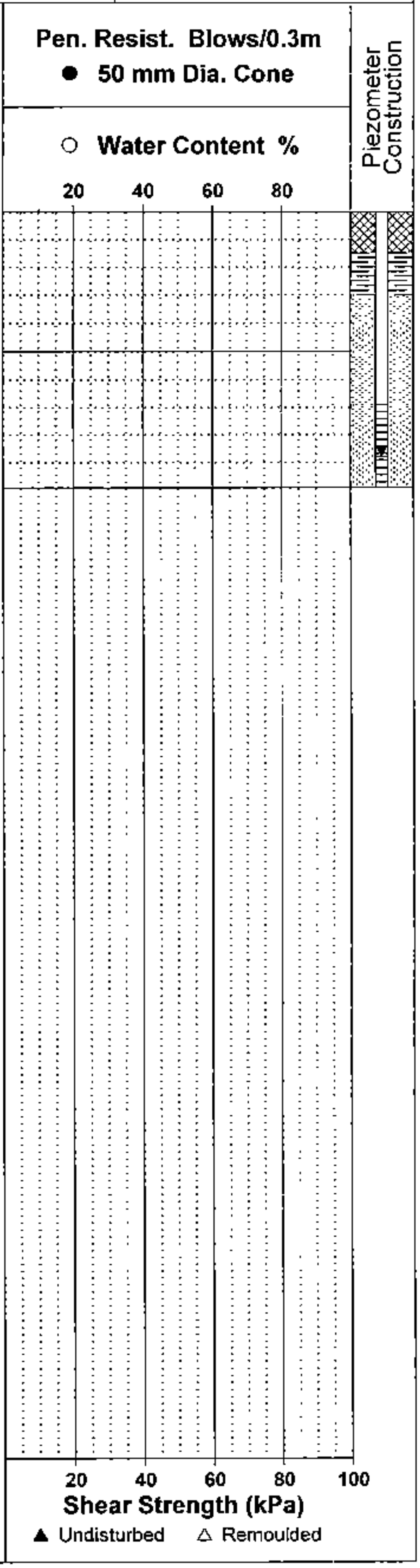
HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	64.34
FILL: Sand and gravel 0.08							
FILL: Crushed stone 0.20		AU	1				
FILL: Sand and gravel 0.91							
GLACIAL TILL: Grey silty sand with gravel and rock fragments		SS	2	33	23	1	63.34
1.98		SS	3	25	50+		

End of Borehole

Practical refusal to augering @ 1.98m depth

(GWL @ 1.76m-Nov. 3/08)



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 27 Oct 08

FILE NO. PG1769

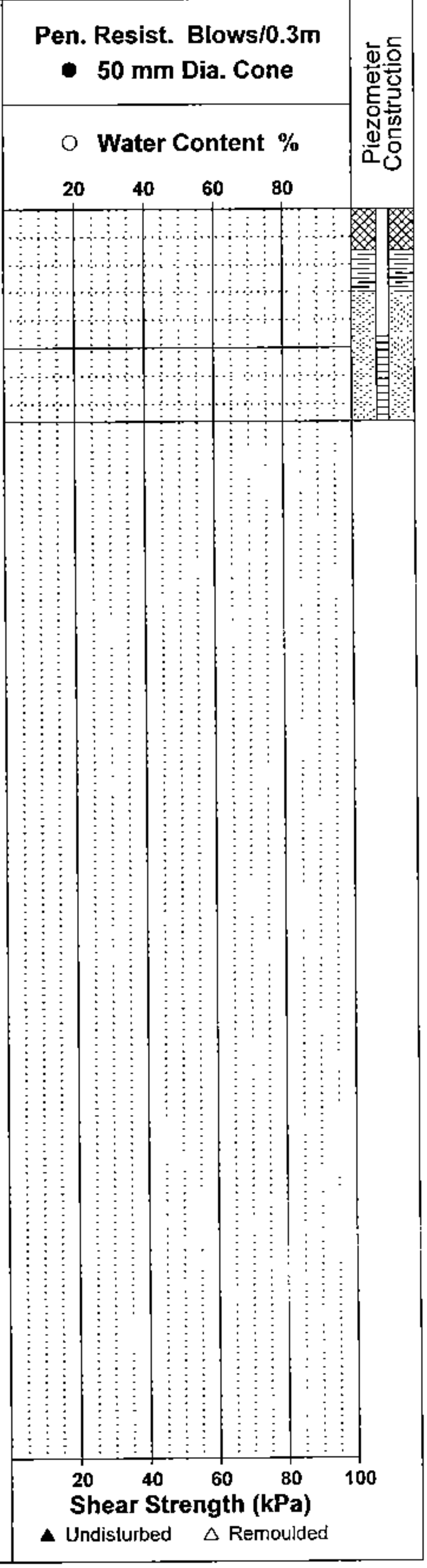
HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.42
FILL: Crushed stone 0.25							
FILL: Sand and gravel 0.91		AU	1				
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders 1.52		SS	2	50	63	1	63.42

End of Borehole

Practical refusal to augering @ 1.52m depth

(Piezometer damaged - Nov. 3/08)



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

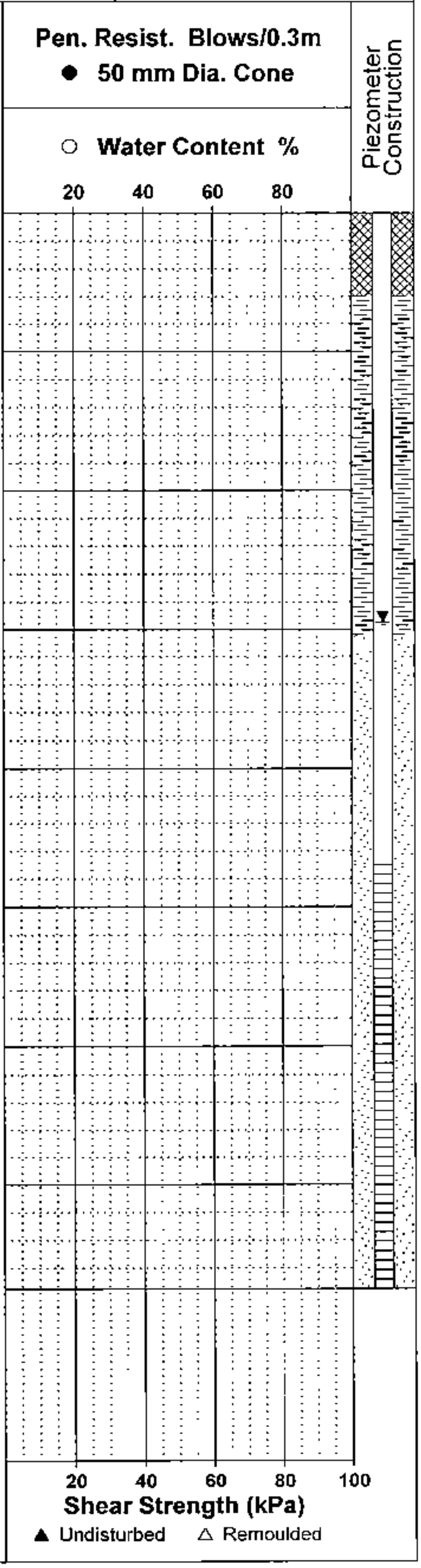
DATE 27 Oct 10

FILE NO. PG1769

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Sand and gravel	AU	1		
	SS	2	42	41
1.47				
BEDROCK: Grey limestone	RC	1	89	77
	RC	2	97	83
	RC	3	93	87
	RC	4	98	83
	RC	5	78	78
7.75				
End of Borehole				
(GWL @ 2.95m-Nov. 3/08)				

DEPTH (m)	ELEV. (m)
0	63.42
1	62.42
2	61.42
3	60.42
4	59.42
5	58.42
6	57.42
7	56.42



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

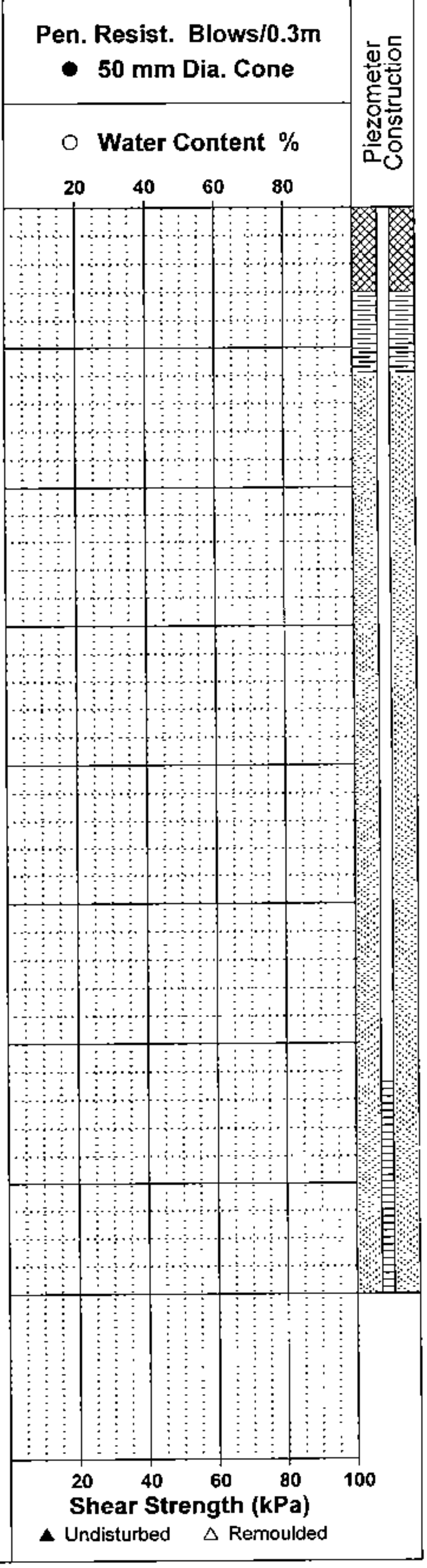
DATE 27 Oct 08

FILE NO. PG1769

HOLE NO. BH 5

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.20	AU	1		
FILL: Sand and gravel	0.91	SS	2		50+
Brown **SILTY SAND**, trace gravel	1.58				
BEDROCK: Grey limestone		RC	1	87	52
		RC	2	97	85
		RC	3	98	93
		RC	4	95	95
	7.80	RC	5	100	92
End of Borehole (Piezometer damaged - Nov. 3/08)					

DEPTH (m)	ELEV. (m)
0	64.27
1	63.27
2	62.27
3	61.27
4	60.27
5	59.27
6	58.27
7	57.27



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

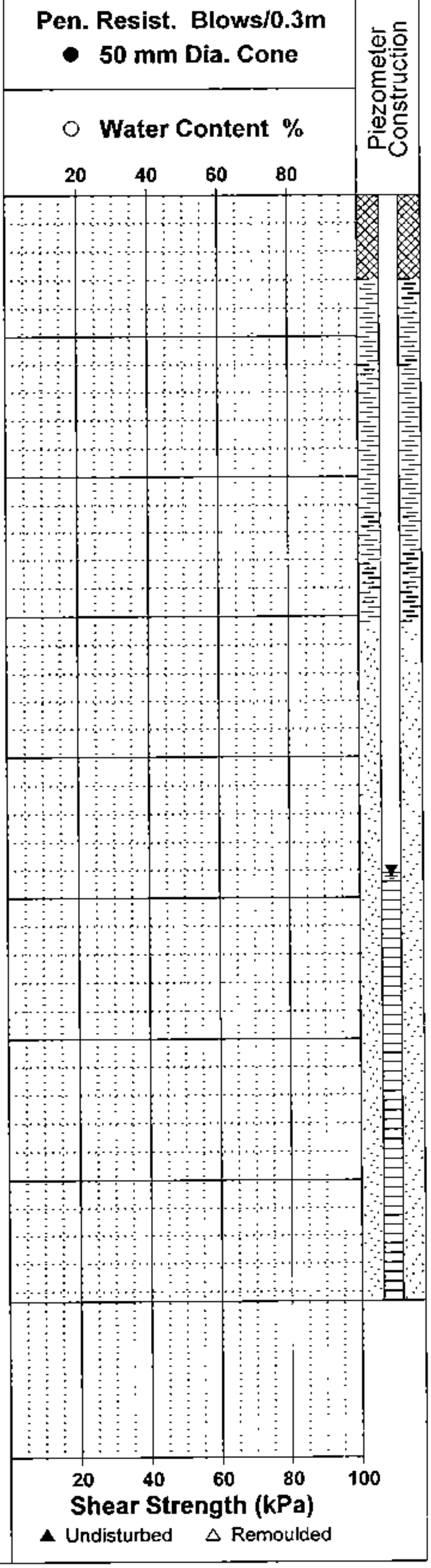
DATE 27 Oct 08

FILE NO. PG1769

HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.05				
FILL: Brown silty sand with gravel and crushed stone 0.46	AU	1		
FILL: Brown silty sand with cobbles and boulders	SS	2	12	50+
1.73	SS	3	17	50+
BEDROCK: Grey limestone	RC	1	78	42
	RC	2	100	85
	RC	3	100	95
	RC	4	100	95
7.87	RC	5	100	100
End of Borehole				
(GWL @ 4.85m-Nov. 3/08)				

DEPTH (m)	ELEV. (m)
0	63.16
1	62.16
2	61.16
3	60.16
4	59.16
5	58.16
6	57.16
7	56.16



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

BORINGS BY CME 75 Power Auger

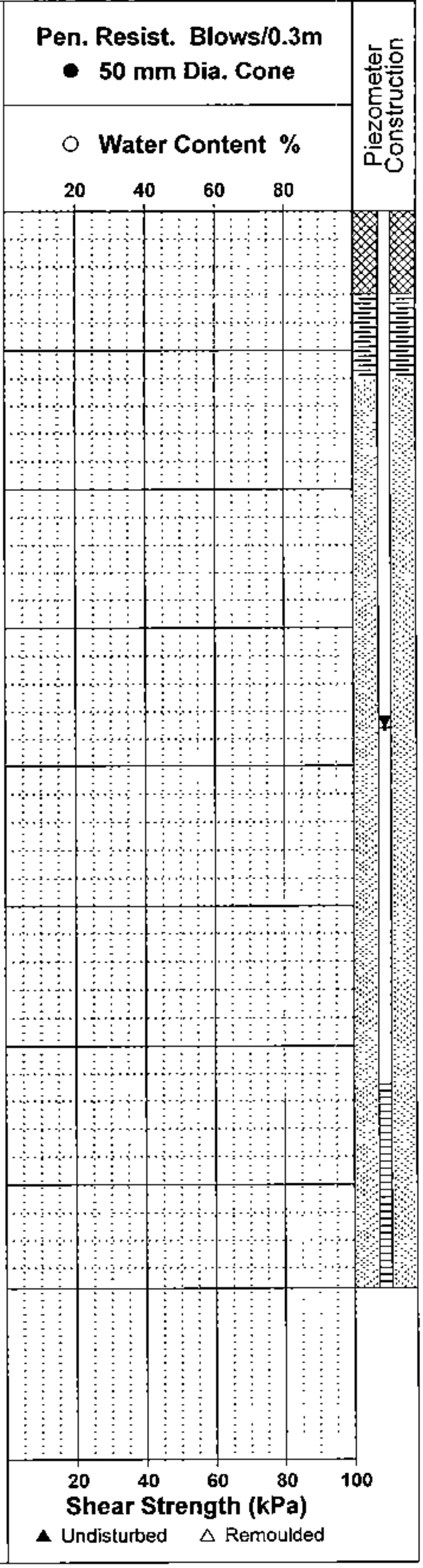
DATE 27 Oct 08

FILE NO. **PG1769**

HOLE NO. **BH 7**

SOIL DESCRIPTION	Depth of layer base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06				
FILL: Crushed stone	0.20	AU	1		
FILL: Sand and gravel	1.52	SS	2	40	50+
Grey-brown **SILTY SAND** with gravel, cobbles and boulders	3.10	SS	3	0	50+
		AU	5		
		SS	4	0	50+
BEDROCK: Grey limestone	7.75	RC	1	92	92
		RC	2	100	80
		RC	3	98	98
		RC	4	100	100
End of Borehole (GWL @ 3.72m-Nov. 3/08)					

DEPTH (m)	ELEV. (m)
0	63.61
1	62.61
2	61.61
3	60.61
4	59.61
5	58.61
6	57.61
7	56.61



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
319 and 320 McRae Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hdyrant at the intersection of McRae Avenue and Scott Street, elevation = 64.445m.

REMARKS

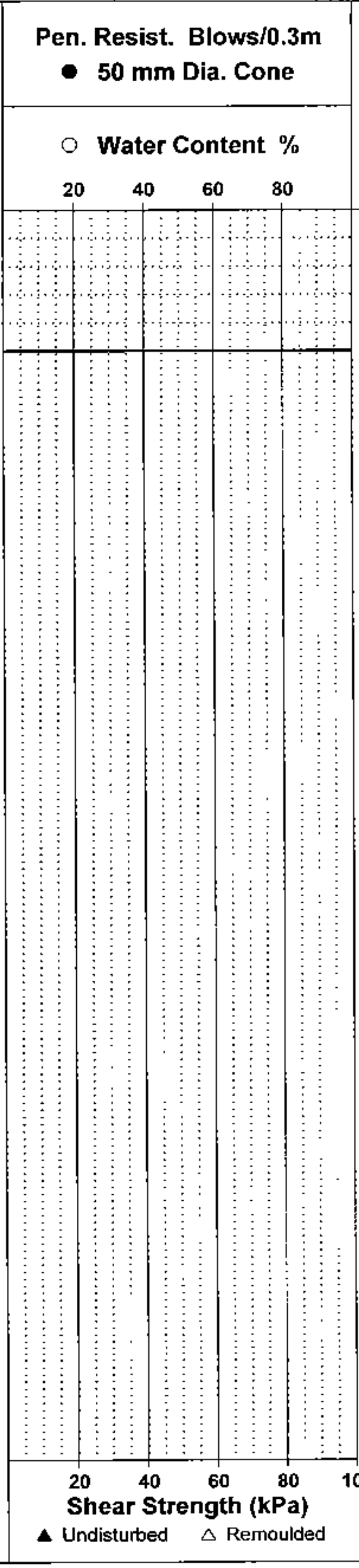
BORINGS BY CME 75 Power Auger

DATE 28 Oct 08

FILE NO. **PG1769**

HOLE NO. **BH 8**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
Asphaltic concrete 0.05	▨							
FILL: Crushed stone 0.30	▨	AU	1					
FILL: Sand and gravel 1.01	▨	SS	2	21	50+	1		
End of Borehole Practical refusal to augering @ 1.01m depth								



PARACEL

Order #: 0844090

Certificate of Analysis

Report Date: 31-Oct-2008
Order Date:29-Oct-2008

Client: **Paterson Group Consulting Engineers**

Client PO: 7290 Project Description: PG1769

	Client ID:	BH2-SS2	-	-	-
	Sample Date:	27-Oct-08	-	-	-
	Sample ID:	0844090-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.1	-	-	-
General Inorganics					
pH	0.05 pH Units	8.35	-	-	-
Resistivity	0.10 Ohm.m	39.0	-	-	-
Anions					
Chloride	5 ug/g dry	55	-	-	-
Sulphate	5 ug/g dry	27	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG1769-1 - TEST HOLE LOCATION PLAN

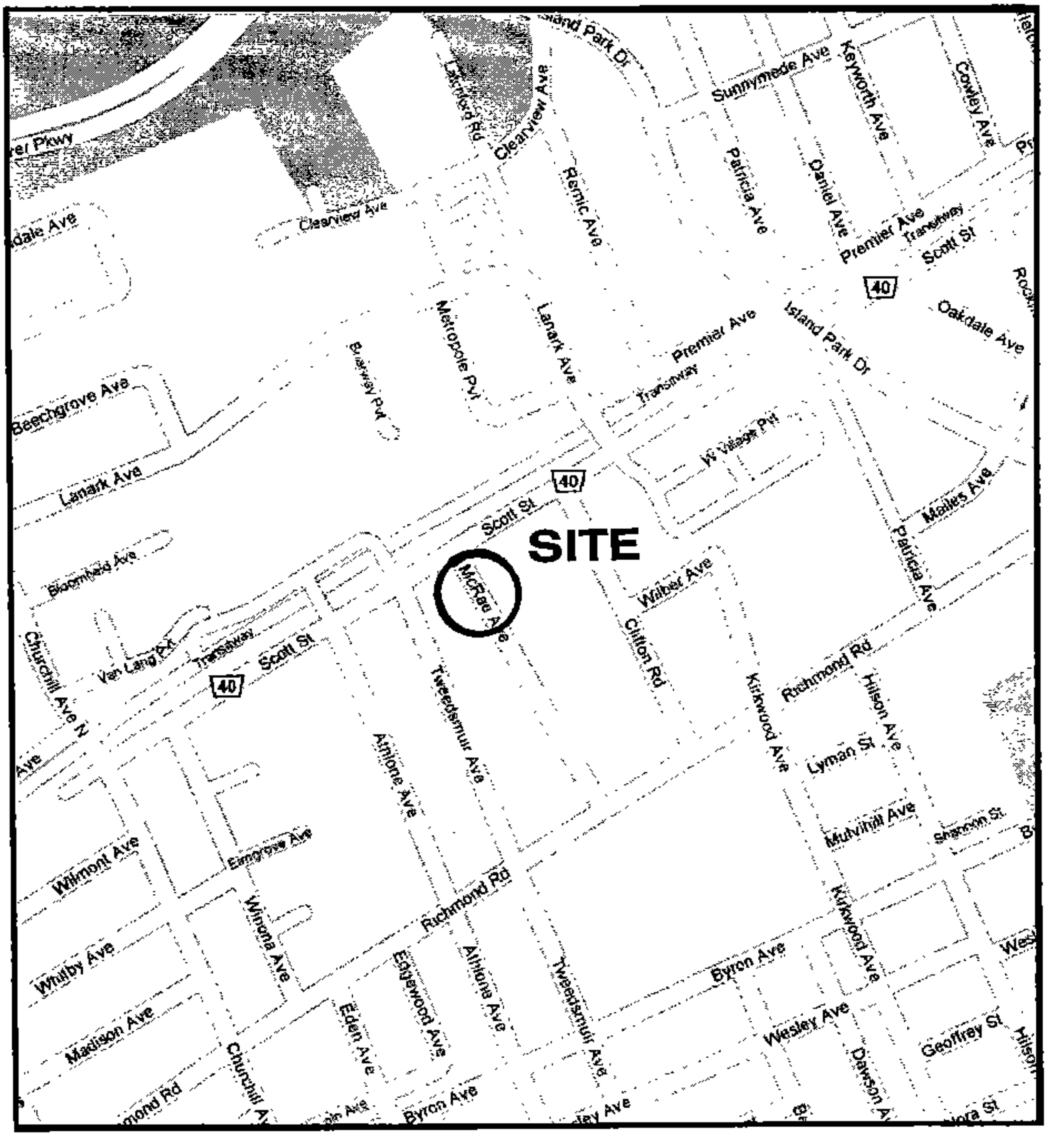


FIGURE 1

KEY PLAN