Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation

Proposed Residential Development
1765 Trim Road
Ottawa, Ontario

Prepared For

Longwood Building Corporation

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September 7, 2012

Report: PG2606-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Longwood Building Corporation to conduct a geotechnical investigation for a proposed residential development located at 1765 Trim Road, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current investigation was to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.

- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

2.0 PROPOSED DEVELOPMENT

Details of the proposed development were not known at the time of writing this report. However, it is expected that the residential development will consist of detached and townhouse style buildings. Associated car parking and/or local roadways are also anticipated. It is further anticipated that the proposed development will be municipally serviced.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on January 12, 2012. At that time, a total of three (3) boreholes were distributed in a manner to provide general coverage of the proposed development. The locations were determined in the field by Paterson personnel taking into consideration site features and underground services. Locations of the test holes are shown in Drawing PG2606-1 - Test Hole Location Plan included in Appendix 2.

Boreholes were put down using a track-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 from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon sampler or a 73 mm diameter thin walled Shelby tubes in conjunction with a piston sampler. All soil samples were visually inspected and initially classified on site. The split-spoon samples were placed in sealed plastic bags and the Shelby tubes were sealed at both ends on site. All samples were transported to the our laboratory for examination and classification. The depths at which the split-spoon and Shelby tube samples were recovered from the test holes are shown as SS and TW, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing was conducted in cohesive soils using a field vane apparatus.

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

The subsurface conditions observed at the borehole locations were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Flexible standpipes were installed in all boreholes 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 and ground surface elevations at the borehole locations were surveyed by Paterson field personnel. The ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located in front of 2080 Breezewood Street. An assumed elevation of 100.00 m was assigned to the TBM. The locations and ground surface elevations of the boreholes, and the location of the TBM are presented on Drawing PG2606-1 - Test Hole Location Plan, in Appendix 2.

3.3 Laboratory Testing

All soil samples recovered from the subject site were sealed in plastic bags and visually examined in our laboratory to review the results of the field logging.

One (1) Shelby tube sample was submitted for unidimensional consolidation testing.

The result of the consolidation testing is presented on the Consolidation Test sheet presented in Appendix 1 and are further discussed in Sections 4 and 5.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site consists of former agricultural land. The site is bordered to the east by single family residential dwellings, to the north and south by vacant land and to the west by Trim Road followed by residential development. A hydro corridor is located north of the subject site.

The site is relatively flat and approxmimately at grade with neighbouring properties and adjacent roadways.

4.2 Subsurface Profile

Based on the soil profile encountered at the test hole locations, the soil profile consists of a 200 to 300 mm thick layer containing organics and roots overlying a deep silty clay deposit. The subsurface conditions observed in the test holes 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.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of either limestone of the Bobcaygeon formation, shale of the Rockcliffe formation, or interbedded limestone and dolomite of the Gull River formation. The overburden drift thickness is estimated to be between 25 to 50 m. Based on the results of the DCPT testing, refusal on assumed bedrock surface was encountered at 30.1 m depth at BH 2.

Silty Clay

The upper 3 m of the silty clay deposit has been desiccated to a stiff to very stiff brown to brown “crust”. The grey silty clay has measured shear strengths of between 30 and 90 kPa, indicating a firm to stiff consistency. All boreholes terminated in the firm grey silty clay.

One (1) sample of the sensitive grey silty clay were subjected to unidimensional consolidation (oedometer) testing. The test result is presented in Subsection 5.3 and on the Consolidation Test sheet in Appendix 1. The consolidation test result indicates that the silty clay is overconsolidated with overconsolidation ratio (OCR) of 1.9. The OCR is the ratio of the preconsolidation pressure to the effective pressure at the sample depth. This is further discussed in Subsection 5.3.

4.3 Groundwater

The measured groundwater levels from piezometers installed at the boreholes are presented in Table 1. It should be noted that surface water can become perched within the backfill material of the borehole, which can result in higher than typical groundwater level readings. Based on field observations and recovered soil sample colour and consistency, the long term groundwater level is expected to be between 2.5 and 3 m depth. It should further be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could be different at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	98.76	0.78	97.98	January 27, 2012
BH 2	98.80	0.45	98.35	January 27, 2012
BH 3	98.69	1.23	97.46	January 27, 2012

Note: The ground surface elevations at the test hole locations were referenced to the TBM consisting of the top of spindle of the fire hydrant located in front of 2080 Breezewood Street. An assumed elevation of 100.00 m was assigned to the TBM.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed residential development. It is expected that the proposed residential buildings will be founded on conventional shallow foundations over a stiff silty clay bearing surface.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures.

Fill Placement

Fill used for grading purposes beneath the proposed buildings, 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 in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas 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 be compacted at minimum 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, unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance at serviceability limit states (SLS) value of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) value of **200 kPa**.

A geotechnical resistance factor of 0.5 was applied to the above-reported bearing resistance at ULS value.

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 prior to the placement of concrete for footings.

Settlement and Permissible Grade Raise

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied. For dwellings, a minimum value of 50% of the live load is recommended by Paterson.

Generally, the potential long term settlement is evaluated based on the compressibility characteristics of the silty clay. These characteristics are estimated in the laboratory by conducting unidimensional consolidation tests on undisturbed soil samples collected using Shelby tubes in conjunction with a piston sampler. One (1) site specific consolidation test was conducted. The results of the consolidation test are presented in Table 2 and in Appendix 1.

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

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

Table 2 - Summary of Consolidation Test Results

Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 3	TW 3	5.09	110	59	0.024	2.174	A

* - Q - Quality assessment of sample - G: Good A: Acceptable P: Likely disturbed

The values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering testing procedures and are estimates only. Natural variations within the soil deposit will affect the results. The p'_o parameter is directly influenced by the groundwater level. Groundwater levels were measured during the site investigation. Groundwater levels vary seasonally which has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests during the investigation are based on the long term groundwater level being at 0.5 m below the existing groundwater table. The groundwater level is based on the colour and undrained shear strength profile of the silty clay.

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when building are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

To reduce potential long term liabilities, consideration should be given to accounting for a larger groundwater lowering and to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the dwellings, etc). Buildings on silty clay deposits increases the likelihood of movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking compared to unreinforced foundations.

Based on our borehole information and testing results, a permissible grade raise restriction of **1.2 m** is recommended for the subject site.

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. Above the groundwater level, adequate lateral support is provided to a very stiff to stiff silty clay or engineered fill 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 or engineered granular fill, as described above.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations considered at this site. The soils underlying the proposed shallow foundations are not susceptible to liquefaction.

Reference should be made to the latest revision of the 2006 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade 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 II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of 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.

5.6 Pavement Design

Car only parking and local roadways are anticipated at this site. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - 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
350	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either inorganic in situ soils or OPSS Granular B Type I or II material placed over in situ soil.

Table 4 - Recommended Pavement Structure - Local 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
450	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either inorganic in situ soils or OPSS Granular B Type I or II material placed over in situ soil.

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

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

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed 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. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose, unless a composite drainage material, connected to the perimeter drainage system, is provided.

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, that are not located adjacent to a heated building area, such as wing walls and isolated exterior pier foundations.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Excavation side slopes above the groundwater level, extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. Flatter excavation slopes are required below the groundwater level.

The undisturbed native soils at this site are considered to be mainly Type 2 and 3 soils according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

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. 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.

Generally, it should be possible to re-use the moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. The silty clay, when wet, will be difficult to reuse due to its high fines content which makes compacting this material without an extensive drying period impractical.

Trench backfill material within the frost zone (approximately 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce 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 to moderate. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

It should also be noted that dewatering of the adjacent properties due to the construction of the proposed development is not anticipated.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Landscaping and Outdoor Structure Considerations

Tree Planting Restrictions

The subject site is located in an area of medium sensitive silty clay deposits for tree planting. It is expected that the combination of the proposed finished grades and the thickness of the underlying weathered clay crust will provide approximately 3 to 4 m thick buffer to the underlying firm silty clay deposit.

Tree planting for this subject development should be limited to low water demand trees. The minimum permissible distance from the foundation will depend on the nature of the tree, the depth of the clay crust and the final grade raise in relation to the permissible grade raise. A minimum permissible distance of 5 m from the foundation wall is recommended for a tree planting setback.

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

Swimming Pools

The in-situ soils are considered to be acceptable for swimming pools. Above ground swimming pools must be placed at least 4 m away from the residence foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

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

7.0 RECOMMENDATIONS

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

- ❑ Review of the grading plan from a geotechnical perspective.
- ❑ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be retained to review the Grading Plan once available. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

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 that we be notified immediately in order 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 Longwood Building Corporation or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Stephanie Boisvenue, B.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER D. J. GILBERT 100116130 PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNIDIMENSIONAL CONSOLIDATION TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-1765 Trim Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 2080 Breezewood Street. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 12, 2012

FILE NO. PG2606

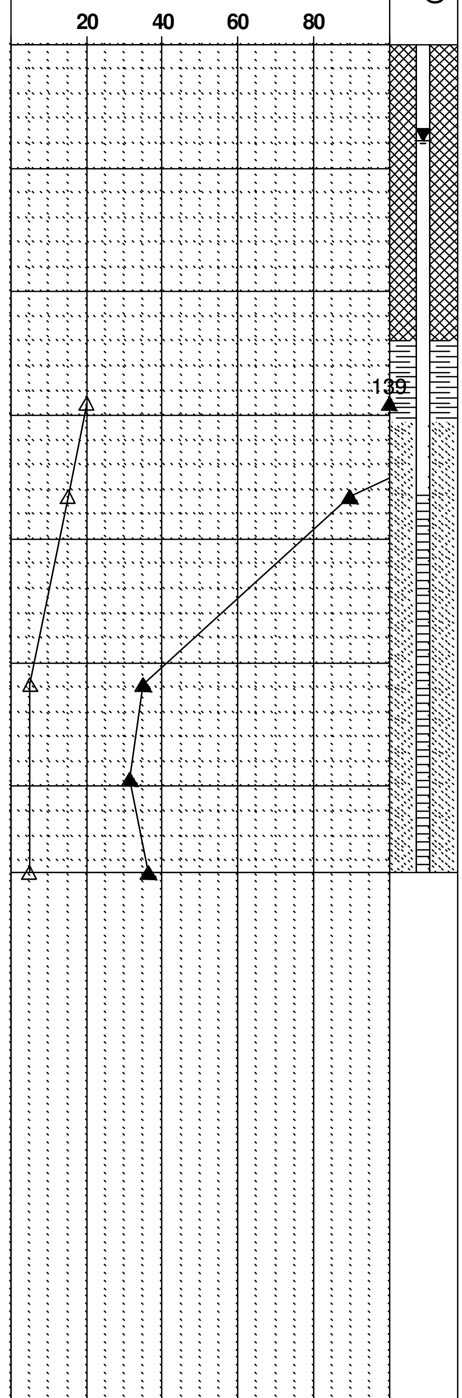
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.76
Organic rootlets within upper 200mm							
		SS	1	75	10	1	97.76
		SS	2	100	5		
Very stiff to stiff, grey-brown **SILTY CLAY**						2	96.76
- stiff to firm and grey by 3.5m depth						3	95.76
		TW	3	83		4	94.76
						5	93.76
						6	92.76
6.70							
End of Borehole							
(GWL @ 0.78m-Jan. 27, 2012)							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-1765 Trim Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 2080 Breezewood Street. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 12, 2012

FILE NO. PG2606

HOLE NO. BH 2

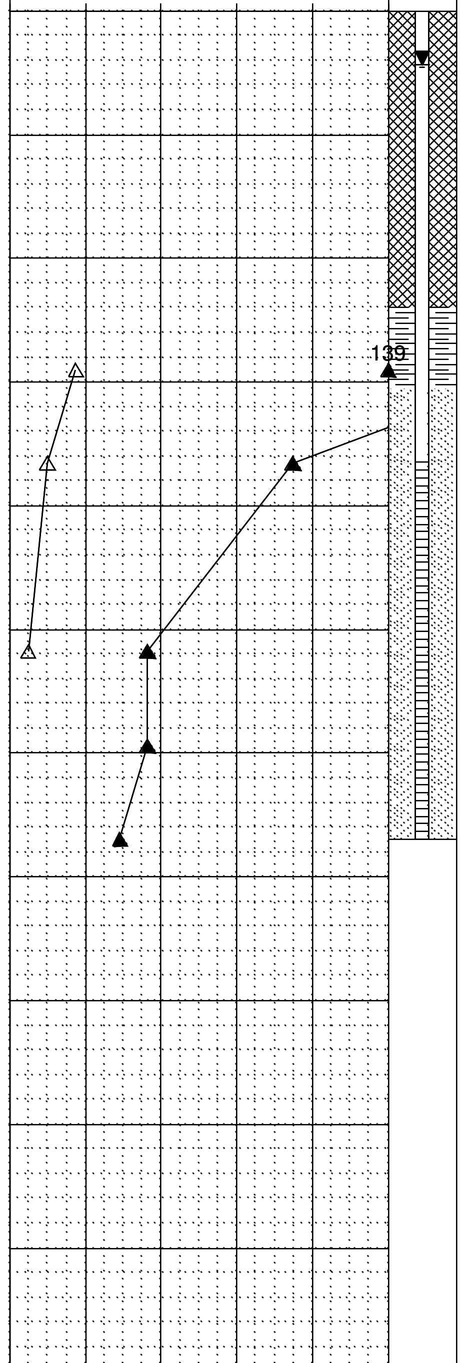
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.80
Organic rootlets within upper 300mm							
		SS	1	100	8	1	97.80
		SS	2	100	6		
Very stiff to stiff, brown **SILTY CLAY**						2	96.80
- stiff to firm and grey by 3.2m depth						3	95.80
		TW	3	83		4	94.80
						5	93.80
						6	92.80
6.70							
Dynamic Cone Penetration Test commenced @ 6.70m depth. Cone pushed to 29.7m depth.						7	91.80
						8	90.80
						9	89.80
						10	88.80
						11	87.80

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

○ Water Content %

20 40 60 80

Piezometer Construction



20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-1765 Trim Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 2080 Breezewood Street. Assumed elevation = 100.00m.

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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
GROUND SURFACE						11	87.80		
						12	86.80		
						13	85.80		
						14	84.80		
						15	83.80		
						16	82.80		
						17	81.80		
						18	80.80		
						19	79.80		
						20	78.80		
						21	77.80		
						22	76.80		

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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DATUM TBM - Top spindle of fire hydrant located in front of 2080 Breezewood Street. Assumed elevation = 100.00m.

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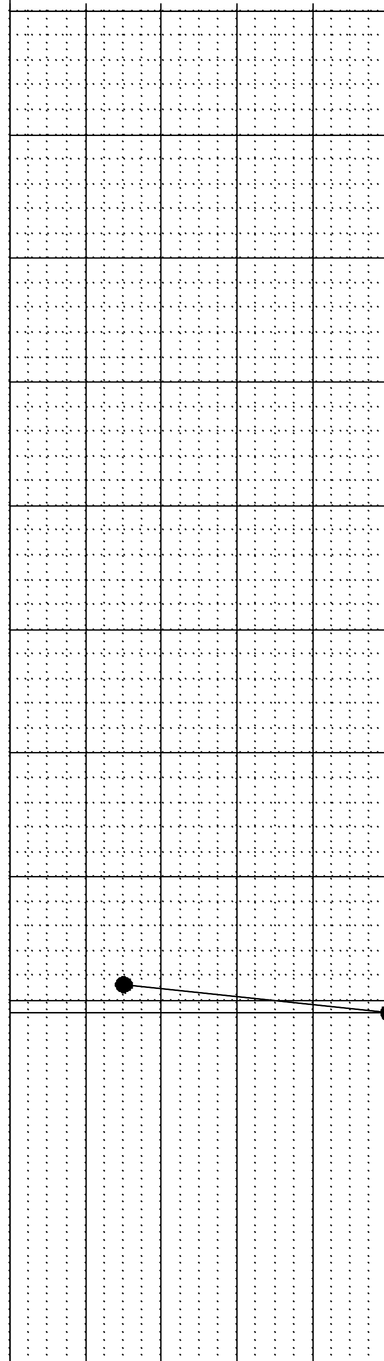
FILE NO. PG2606

HOLE NO. BH 2

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 %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						22	76.80		
						23	75.80		
						24	74.80		
						25	73.80		
						26	72.80		
						27	71.80		
						28	70.80		
						29	69.80		
30.10						30	68.80		
End of Borehole Practical refusal to DCPT @ 30.10m depth. (GWL @ 0.43m-Jan. 27, 2012)									



20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-1765 Trim Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 2080 Breezewood Street. Assumed elevation = 100.00m.

REMARKS

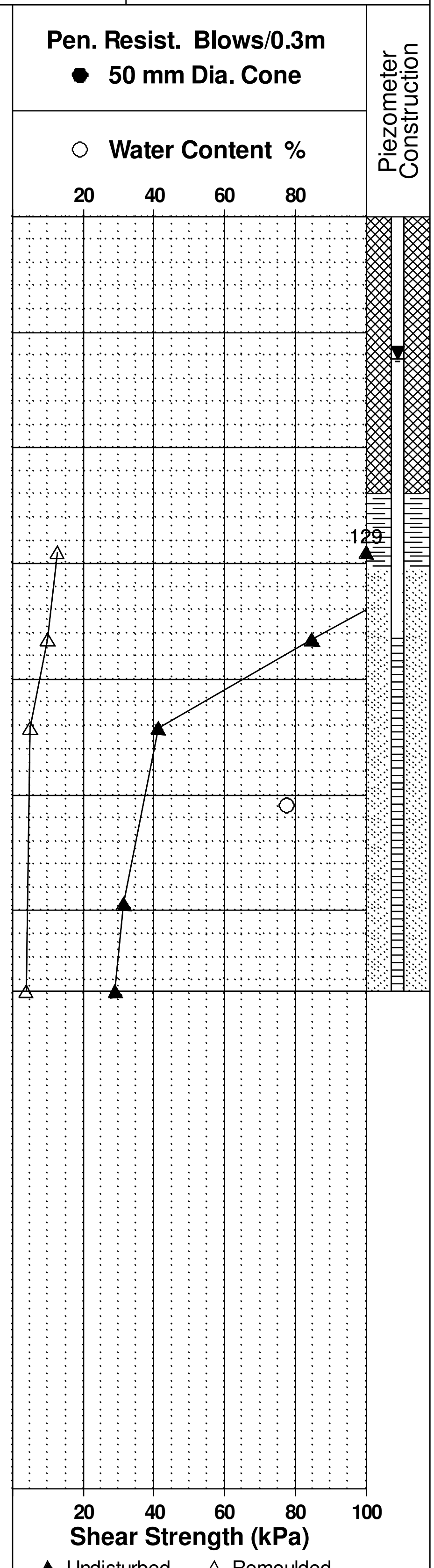
BORINGS BY CME 55 Power Auger

DATE January 12, 2012

FILE NO. PG2606

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.69
Organic rootlets within upper 200mm							
		SS	1	75	7	1	97.69
		SS	2	79	6	2	96.69
Very stiff to stiff, brown **SILTY CLAY**							
- stiff to firm and grey by 3.0m depth						3	95.69
						4	94.69
		TW	3	83		5	93.69
						6	92.69
6.70							
End of Borehole							
(GWL @ 1.23m-Jan. 27, 2012)							



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

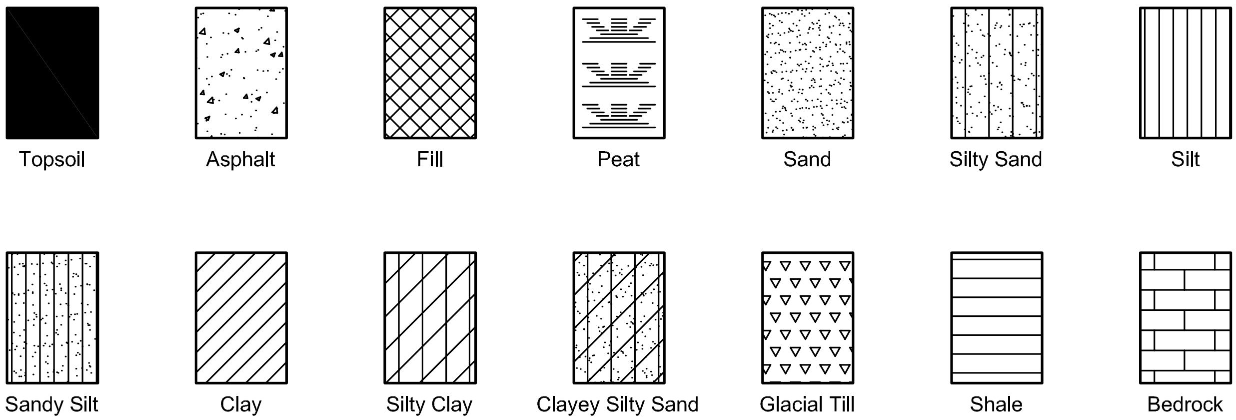
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

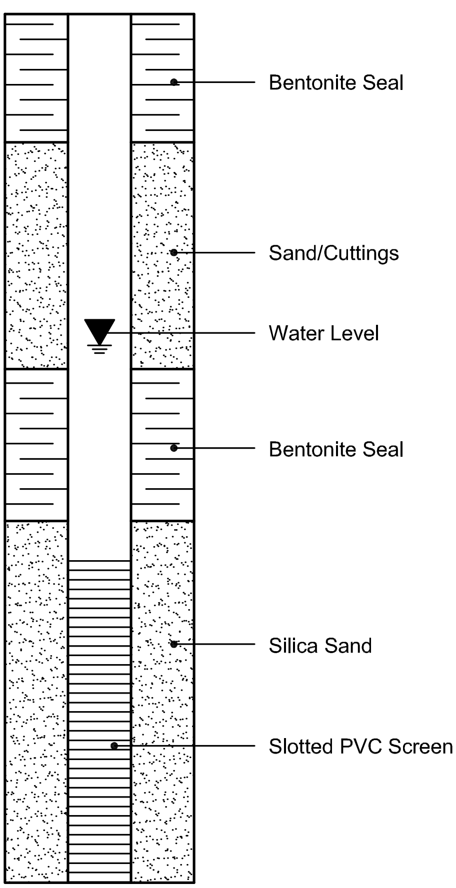
SYMBOLS AND TERMS (continued)

STRATA PLOT

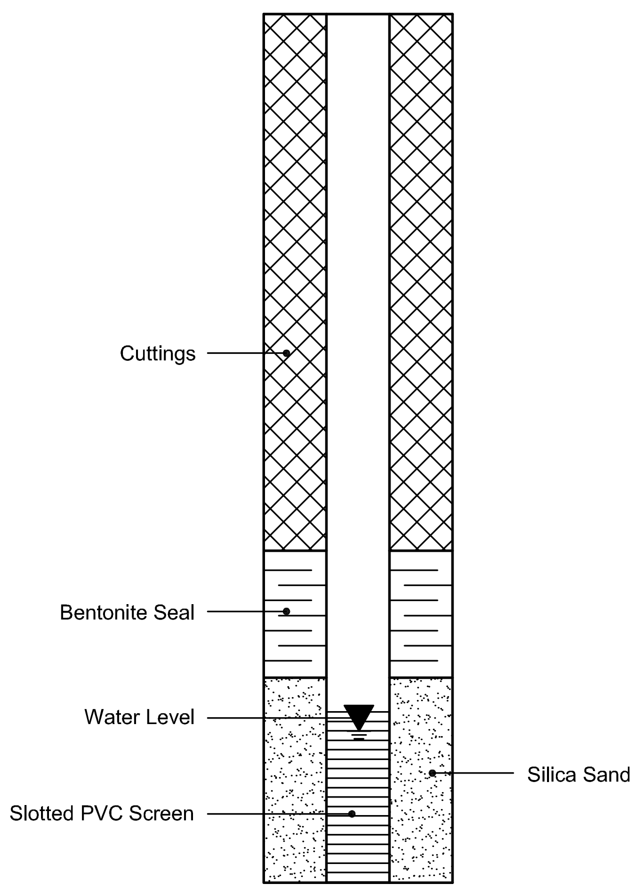


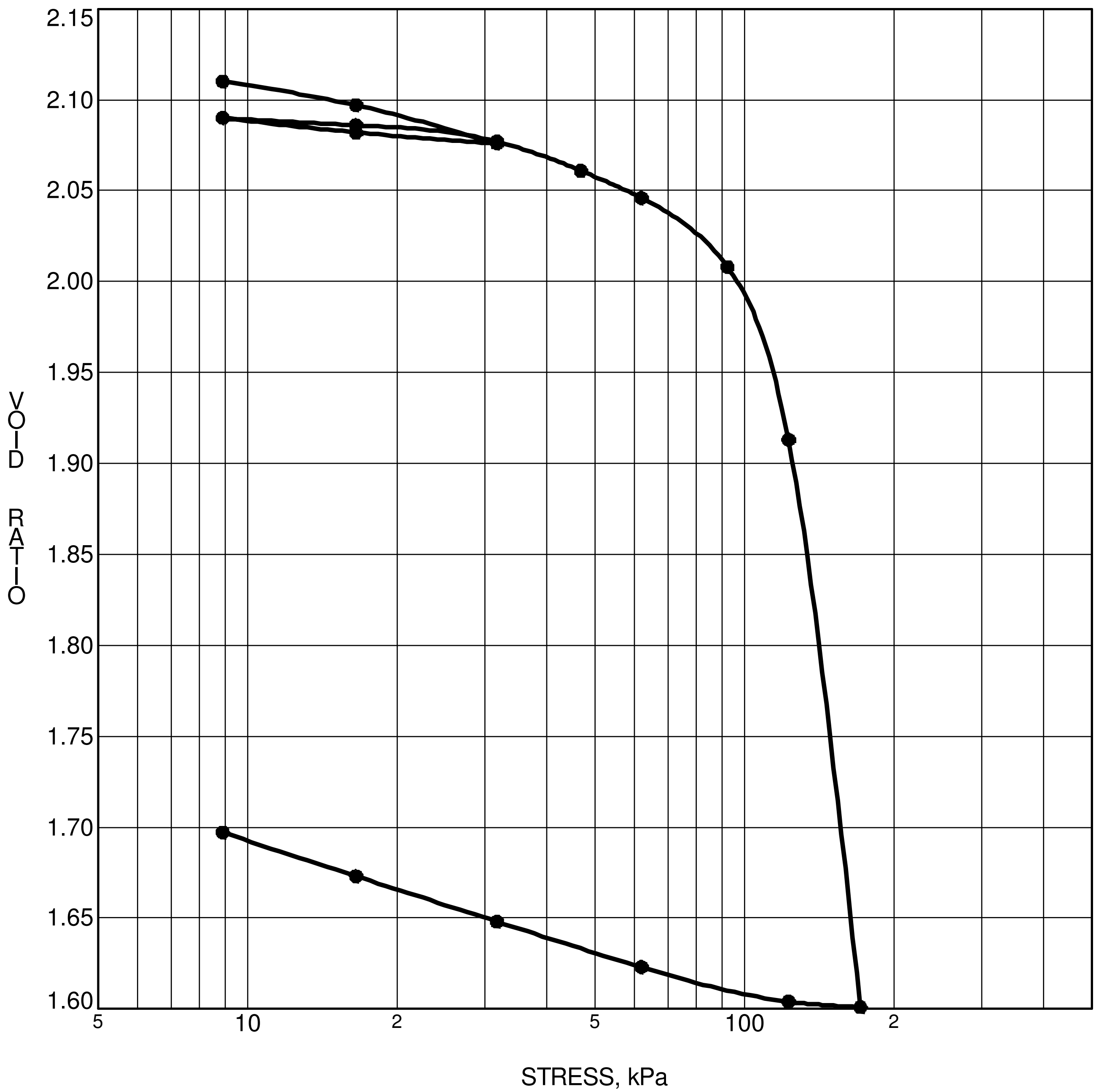
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 3	p'o	59 kPa	Ccr	0.024
Sample No.	TW 3	p'c	110 kPa	Cc	2.174
Sample Depth	5.09 m	OC Ratio	1.9	Wo	77.6 %
Sample Elev.	93.60 m	Void Ratio	2.133	Unit Wt.	15.7 kN/m ^{3}

CLIENT **Longwood Building Corporation** FILE NO. **PG2606**

PROJECT **Geotechnical Investigation - Proposed Residential Development-1765 Trim Road** DATE **01/19/2012**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2606-1 - TEST HOLE LOCATION PLAN

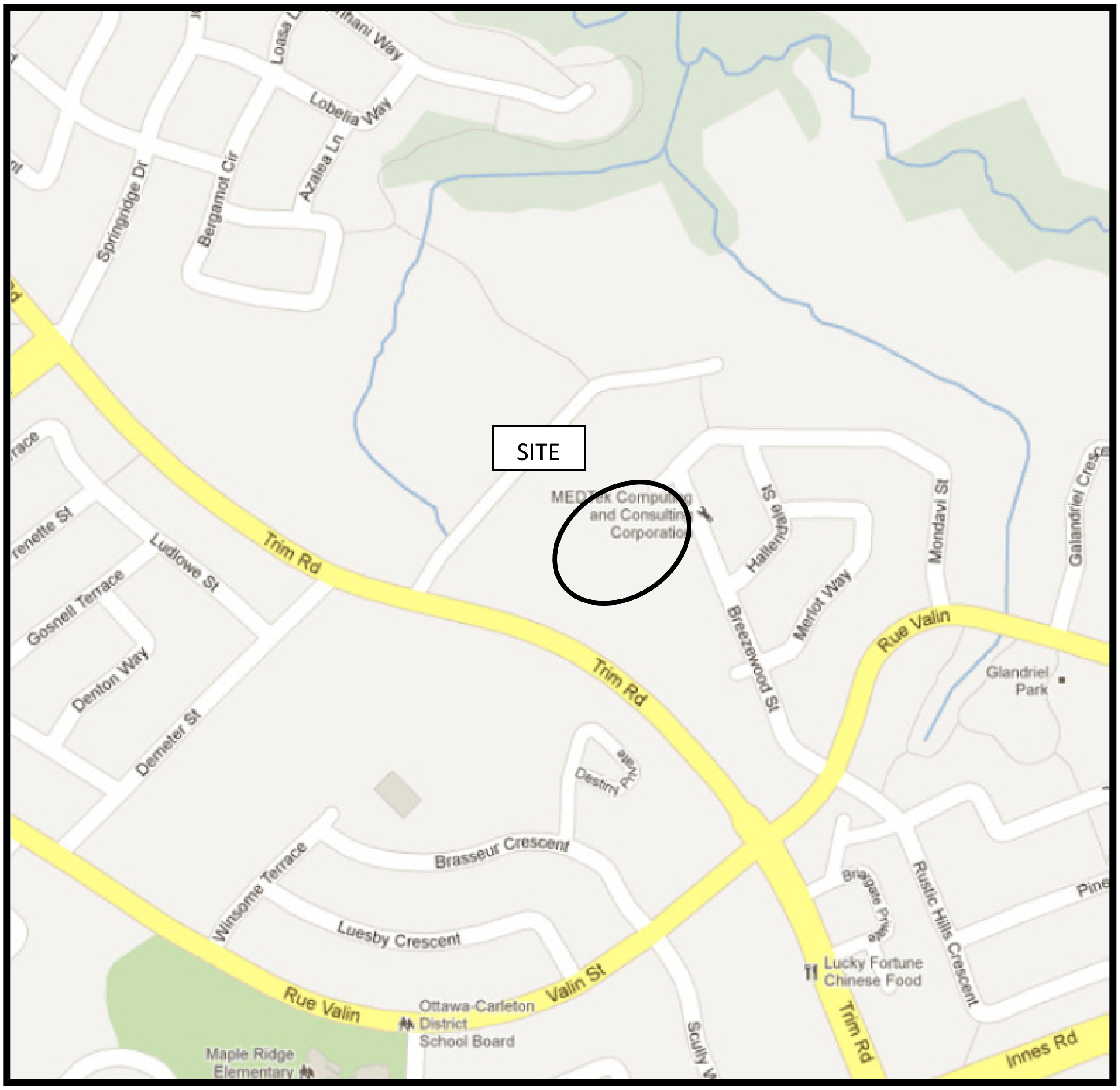
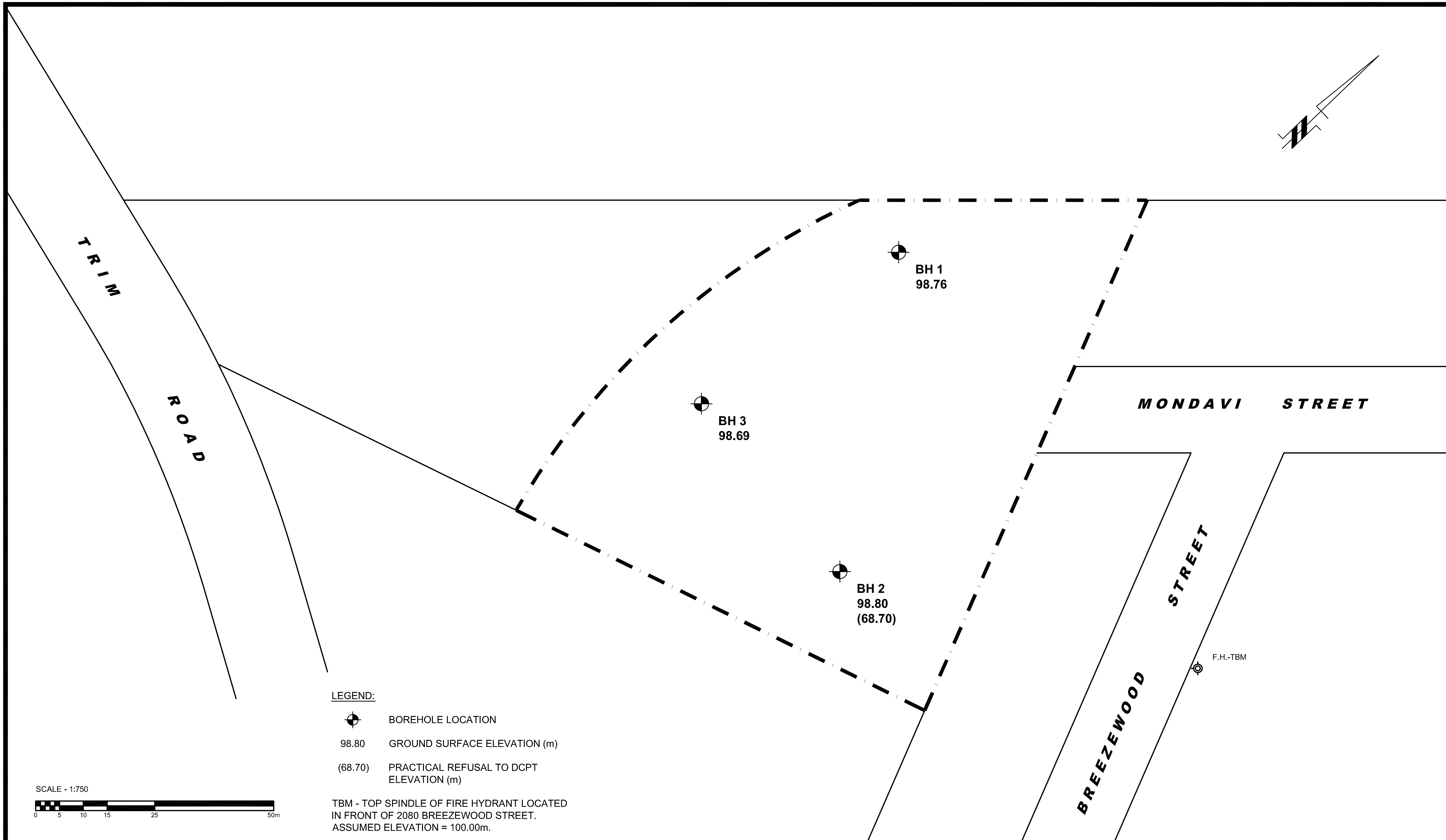


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
98.80	GROUND SURFACE ELEVATION (m)
(68.70)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED IN FRONT OF 2080 BREEZEWOOD STREET. ASSUMED ELEVATION = 100.00m.

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

Scale:	1:750
Des.:	DG
Dwn:	MPG
Chkd:	DG

LONGWOOD BUILDING
GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - 1765 TRIM ROAD
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG2606-1
Report No.:	PG2606-1
Date:	01/2012

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