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Geotechnical Investigation

Proposed Residential Development
Terry Fox Drive at Cope Drive - Ottawa

Prepared For

Claridge Homes

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Report: PG4466-1

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

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

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the subject site located at the northeast corner of the intersection of Terry Fox Drive and Cope Drive, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

☐ determine the subsoil and groundwater conditions at this site by means of boreholes and relevant test holes completed as part of the previous geotechnical investigation.

☐ provide geotechnical recommendations for the design of the proposed development based on the results of the boreholes and other soil information available. These recommendations include permissible grade raises, long term settlements and other construction considerations which may affect its design.

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

Investigating the presence or potential presence of contamination on the proposed development was not part of the scope of work. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the current concept plan, it is understood that the development will consist of residential buildings, parkland and local roadways. It is further understood that the proposed development will be serviced by municipal water, sanitary and storm services.

3.0 Method of Investigation

3.1 Field Investigation

The field portion of the current geotechnical investigation was carried out on April 20, 2018. At that time, a total of three (3) boreholes were placed in a manner to provide general coverage of the subject site taking into consideration site features, underground utilities and existing test holes completed during the previous investigation. The field portion of the previous geotechnical investigation was carried out on April 26 and 27, 2006. During that time, a total of three (3) boreholes were distributed in a manner to provide general coverage of the subject site. The relevant boreholes completed during the previous geotechnical investigation are presented in Appendix 1 of the current geotechnical report. The location of the boreholes are presented on Drawing PG4466-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, 73 mm diameter thin walled (TW) Shelby tubes in conjunction with a piston sampler or from the auger flights. All soil samples were visually inspected and initially classified on site. The auger and 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 auger, split-spoon, and Shelby tube samples were recovered from the test holes are shown as AU, 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.

The thickness of the silty clay layer was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT). The DCPTs were completed at BH 2 and BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed at the test hole locations were recorded in detail in the field. Our findings are presented in 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 were selected by Paterson personnel to provide general coverage of the site. The locations of the boreholes were determined in the field by Paterson personnel using a hand-held global positioning system (Garmin - GPS map76). The ground surface elevations at the boreholes were not determined as part of the current investigation.

The locations of the boreholes are presented on Drawing PG4466-1 - Test Hole Location Plan, included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

Two (2) Shelby tube samples collected from the boreholes during the current investigation and three (3) Shelby tube samples collected during the previous investigation were submitted for unidimensional consolidation. The results of the consolidation are presented on the Unidimensional Consolidation Test Results in Appendix 1 and are further discussed in Sections 4 and 5.

Three (3) representative soil samples were submitted for Atterberg limit testing during the previous investigation. The results of the Atterberg testing are presented on the Atterberg Limit Results sheet presented in Appendix 1 and are further discussed in Sections 4 and 5.

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.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The results are provided in Appendix 1, and are discussed further in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site consists of agricultural land bordered to the west by Terry Fox Drive and to the south by Cope Drive. The Monahan Drain (Branch “B”) borders the north boundary of the site followed by agricultural land. Newly constructed townhouses occupy the neighboring property to the east.

The site is relatively flat and slight below the level of Terry Fox Drive and Cope Road to the west and south, respectively.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the borehole locations consist of topsoil overlying a thin silty sand layer followed by a deep silty clay deposit. Practical refusal to DCPT testing was observed at depths of 34.3 and 35.8 m below the ground surface in BH 2 and BH 3, respectively. 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 test hole location.

Based on available geological mapping, the depth to bedrock in the general area of the site is expected to range from 25 to 50 m. The bedrock in this area is part of the Gull River formation and consists of interbedded limestone and dolomite.

Silty Clay/Clayey Silt

Grey silty clay to clayey silt was encountered below the silty sand at all test hole locations.

The upper portion of the silty clay frequently contains silty sand/sandy silt or silt seams. In situ shear vane field testing carried out within the silty clay yielded undrained shear strength values ranging from approximately 25 to 55 kPa, indicating a soft to stiff consistency. This layer is generally firm in consistency and increases in strength with depth.

Two (2) silty clay samples collected during the current investigation and three (3) silty clay samples collected during the previous investigation were subjected to unidimensional consolidation testing. The results of this testing are presented in Appendix 1 and summarized in Table 4, Subsection 5.3 and indicate that the silty clay is overconsolidated with overconsolidation ratios varying between 1.5 and 2.2.

The results of Atterberg Limits tests conducted on trimmings from the three (3) consolidation test samples of the silty clay/clayey silt recovered during the previous investigation are presented below in Table 1 - Summary of Attergerg Limits' Results and on the Atterberg Limits' Results sheet (plasticity chart) in Appendix 1. The tested silty clay/clayey silt samples had measured liquid limits of 33 to 36% and plasticity indices ranging from 14 to 17, which classifies them as clays of low to medium plasticity (CL) in accordance with the Unified Soil Classification System.

Table 1 - Summary of Atterberg Limits' Results

Sample	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification
BH 1 - TW 7	34	19	15	CL
BH 2 - TW 4	36	19	17	CL
BH 3 - TW11	34	20	15	CL

4.3 Groundwater

Groundwater level readings were recorded on May 3, 2018 at the boreholes completed during the current geotechnical investigation. In addition, the groundwater level readings recorded on May 5, 2006 completed as part of the previous geotechnical investigation are provided. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1 and provided in Table 2 - Summary of Groundwater level Readings.

It is important to note that groundwater readings at piezometers can be influenced by surface water perched within the borehole backfill material. Groundwater conditions can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected between 1.5 to 2.5 m depth. Groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

Table 2 - Summary of Groundwater Level Readings

Borehole Number	Groundwater Levels (m)	Recording Date
	Depth	
Current Investigation		
BH 1-18	0.33	May 3, 2018
BH 2-18	0.76	May 3, 2018
BH 3-18	1.92	May 3, 2018
Previous Investigation		
BH1	0.96	May 5, 2006
BH2	0.79	May 5, 2006
BH2	0.64	May 5, 2006

Note: - Groundwater levels measures from existing ground surface at each test hole location.

5.0 Discussion

5.1 Geotechnical Assessment

It is anticipated that the proposed buildings will be supported by shallow footings placed over stiff to firm silty clay or compact silty sand bearing surface. However, due to the presence of the sensitive silty clay layer, the proposed development will be subjected to grade raise restrictions. The permissible grade raise recommendations are further discussed in Subsection 5.3.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

It is anticipated that deep services will be completed through mainly OHSA Type 3 soils, which has the potential for basal heave. It is assumed that the excavations will be carried out within the confines of a fully-braced steel trench box or other acceptable shoring system designed by a qualified structural engineer to resist the design lateral earth pressures and potential basal heave issues.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill used for grading beneath the proposed building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be placed in lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill 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 excavated brown silty clay or silty sand, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, it is recommended that the material be placed under dry conditions and in above freezing temperatures, compacted in thin lifts using a suitable compaction equipment for the lift thickness by making several passes and approved by the geotechnical consultant. 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.

Protection of Subgrade and Bearing Surfaces

It is expected that site grading and preparation will consist of stripping of the soils containing significant amounts of organic materials. The contractor should take appropriate precautions to avoid disturbing the subgrade and bearing surfaces from construction and worker traffic. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional fill.

5.3 Foundation Design

Bearing Resistance Values

Using continuously applied loads, footings for the proposed buildings can be designed using the bearing resistance values presented in Table 3.

Table 3 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Compact silty sand	60	125
Firm Clayey Silt/Silty Clay	60	125
Stiff Silty Clay/Clayey Silt	100	150

Note: Footings, up to 3 m wide, can be designed using the abovenoted bearing resistance values placed over a silty clay bearing surface.

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.

Bearing resistance values for footing design should be determined on a per lot basis at the time of construction.

Where fill is required to raise the grade below the footing level, the fill located within the zone of influence of the footings should consist of engineered fill. The engineered fill should consist of OPSS Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD. A bearing resistance value at SLS of **80 kPa** and a factored bearing resistance value at ULS of **140 kPa** can be used for footings placed on an approved engineered fill bearing surface provided the fill layer is at least 0.5 m thick.

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 the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Settlement/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. Two (2) site specific consolidation tests were conducted as part of the current investigation and three (3) relevant consolidation testing conducted during the previous investigation has been emended to the current report. The results of the consolidation tests from our investigation is presented in Table 4 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 4 - Summary of Consolidation Test Results

Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
Current Investigation:							
BH 1-18	TW 4	4.9	100	47	0.024	1.428	G
BH 3-18	TW 3	4.9	103	47	0.024	1.409	G
Previous Investigation:							
BH 1	TW 7	8.0	138	87	0.010	0.882	G
BH 2	TW 4	4.2	110	55	0.013	1.127	G
BH 3	TW 11	14.0	200	136	0.014	1.241	F-P

* - 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 carried out for the present investigation are based on the long term groundwater level observed at each borehole location. 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 the consolidation testing results and undrained shear strength values at the borehole locations, our permissible grade raise restriction for lot grading at the residential buildings is **1.8 m** above original ground surface. A permissible grade raise restriction of **2.2 m** can be used for the proposed roadways.

Several options could be considered to accommodate proposed grade raises in exceedance of our permissible grade raise recommendations, such as, the use of lightweight fill, which allow for raising the grade without adding a significant load to the underlying soils. Alternatively, it is possible to preload or surcharge the subject site in localized areas provided sufficient time is available to achieve the desired settlements.

5.4 Foundation Options

Based on the above discussion, several options could be considered for the foundation support of the proposed buildings:

Scenario A

Where the grade raise is close to, but below, the maximum permissible grade raise, consideration should be given to using more reinforcement in the design of the foundation (footings and walls) to reduce the risks of cracking in the concrete foundation. The use of control joints within the brick work between the garage and basement area should also be considered.

Scenario B

Where the grade raise cannot be accommodated with soil fill, the following options could be used alone or in combination.

Option 1 - Use of Lightweight Fill

Lightweight fill (LWF) can be used, consisting of EPS (expanded polystyrene) Type 19 or 22 blocks or other light weight materials which allow for raising the grade without adding a significant load to the underlying soils. However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the garage and porch areas to reduce the fill-related loads.

Option 2 - Preloading or Surcharging

It is possible to preload or surcharge the proposed site in localized areas provided sufficient time is available to achieve the desired settlements based on theoretical values from the settlement analysis. If this option is considered, a monitoring program using settlement plates will have to be implemented. This program will determine the amount of settlement in the preloaded or surcharged areas. Obviously, preloading to proposed finished grades will allow for consolidation of the underlying clays over a longer time period. Surcharging the site with additional fill above the proposed finished grade will add additional load to the underlying clays accelerating the consolidation process and allowing for accelerated settlements. Once the desired settlements are achieved, the site can be unloaded and the fill can be used elsewhere on site.

With both the preloading and surcharging methods, the loading period can be reduced by installing vertical wick drains or sand drains in the silty clay layer to promote the movement of groundwater towards the ground surface. However, vertical drains are expensive for this type of residential project.

5.5 Design for Earthquakes

The site class for seismic site response can be taken as **Site Class E** for the shallow foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.6 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, undisturbed native soil or approved engineered fill surface will be considered acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. 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-slab fill consist of 19 mm clear crushed stone.

5.7 Pavement Structure

Car only parking areas, local and collector roadways are anticipated at this site. The proposed pavement structures are shown in Tables 5, 6 and 7.

Table 5 - Recommended Pavement Structure - Driveways

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 6 - Recommended Pavement Structure - Local Residential Roadways

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
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	

Table 7 - Recommended Pavement Structure - Roadways with Bus Traffic

Thickness mm	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
600	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either in situ soil or OPSS Granular B Type II material placed over in situ soil

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. Weak subgrade conditions may be experienced over service trench fill materials, which will require the use of a woven geotextile liner, such as a Terratrack 200 or equivalent, as well as, an additional 300 to 600 mm thick granular layer, consisting of a 150 mm minus, well graded granular fill or crushed concrete, to provide adequate construction access. Consideration could also be given using a biaxial geogrid, such as Geosynthetics TBX2500 or equivalent.

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. Minimum Performance Graded (PG) 64-28 asphalt cement should be used for this project.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. 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

A perimeter foundation drainage system is recommended for proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, 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 or sump pit.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 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 other exterior unheated footings.

6.3 Excavation Side Slopes

Excavations will be mostly through loose silty sand and grey silty clay. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used.

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 slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A minimum of 4 to 6 m setback should be considered from the excavation face depending on the excavation depth and soil consistency.

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.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
☐ Piping from water seepage through granular soils, and
☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

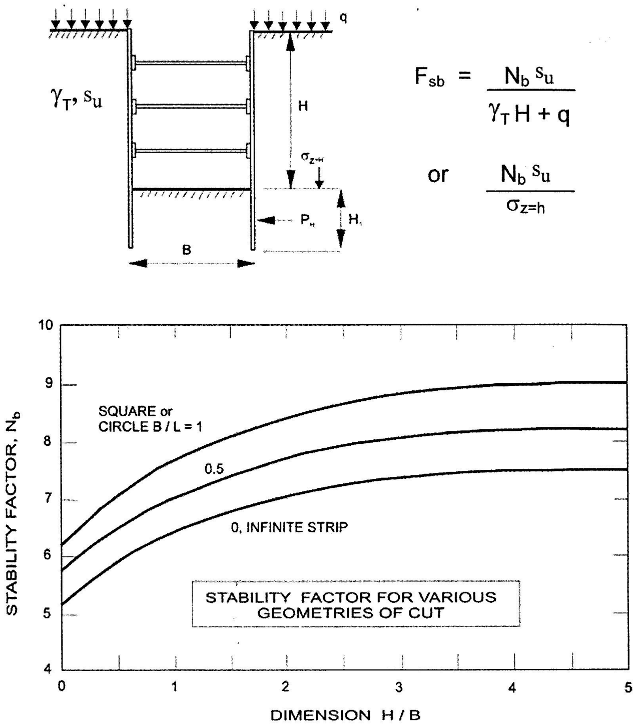


Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa.

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

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

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

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

To reduce long-term lowering of the groundwater at this site, clay seals should be provided within the service trenches excavated through the silty clay deposit. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches excavated through the silty clay deposit.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

Permit to Take Water

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

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

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

6.6 Winter Construction

The subsurface conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur. Precautions should be taken if winter construction is considered for this project.

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

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Landscaping Considerations

Tree Planting Setbacks

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. The results of our testing are presented in Table 1 in Subsection 4.1 and in Appendix 1.

Based on the results of our review, the site is located in a low/medium sensitivity clay soil according to the City of Ottawa Tree Planing in Sensitive Marine Clay Soils.

Low/Medium Sensitivity Clay Soils

A low to medium sensitivity clay soil was encountered between design underside of footing elevations and 3.5 m below finished grade as per City Guidelines. Based on our Atterberg Limits test results, the modified plasticity limit generally does not exceed 40% and the following tree setbacks are recommended. Large trees (mature height over 14 m) can be planted within Area 2 provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

☐ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

❑ Grading surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

Swimming Pools

The in-situ soils are considered to be acceptable for swimming pools. Above ground swimming pools must be placed at least 5 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

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, 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.

6.8 Corrosion Potential and Sulphate

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

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined:

☐ Review detailed grading plan(s) from a geotechnical perspective.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ 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 placing backfilling materials.

☐ Field density tests to ensure that the specified level of compaction has been 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 Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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, Paterson requests to be notified immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Claridge Homes or their 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.

LICENSED PROFESSIONAL ENGINEER
July 14, 2018
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Richard Groniger, C. Tech.

David J. Gilbert, P.Eng.

Report Distribution:

☐ Claridge Homes (3 copies)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TESTING RESULTS

ATTERBERG LIMITS' TESTING RESULTS

ANALYTICAL TEST RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Terry Fox Drive
Ottawa, Ontario

DATUM

REMARKS

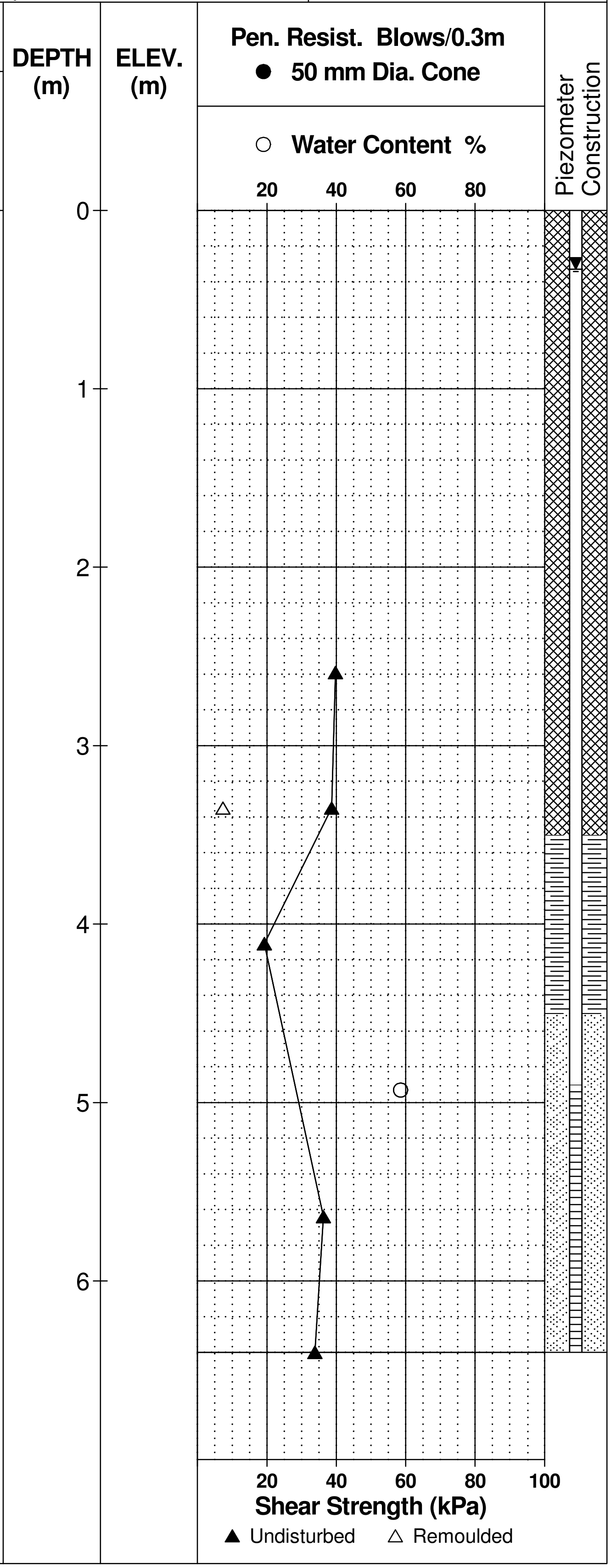
BORINGS BY CME 75 Power Auger

DATE April 20, 2018

FILE NO. PG4466

HOLE NO. BH 1-18

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30	AU	1		
Compact to loose, brown **SILTY SAND,** trace to some silty clay	SS	2	67	7
1.80	SS	3	75	2
Interlayered very loose, grey **SILTY SAND** and firm, grey **SILTY CLAY / CLAYEY SILT** 2.90				
Firm, grey **SILTY CLAY / CLAYEY SILT**	TW	4		
6.40				
End of Borehole				
(GWL @ 0.33m - May 3, 2018)				



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Terry Fox Drive
Ottawa, Ontario

DATUM

REMARKS

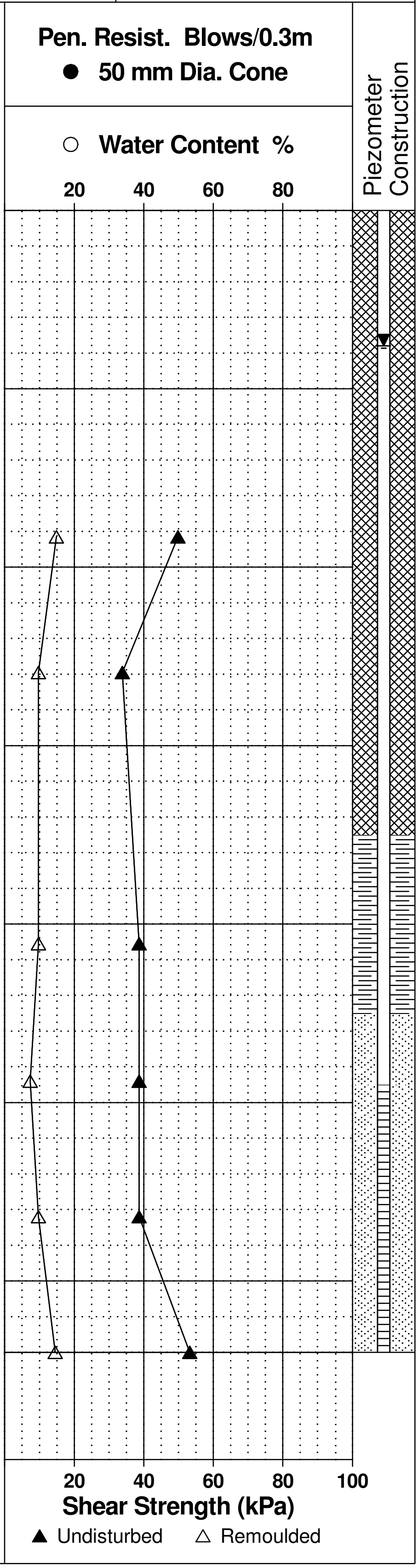
BORINGS BY CME 75 Power Auger

DATE April 20, 2018

FILE NO. PG4466

HOLE NO. BH 2-18

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL	0.25	AU	1				
Compact to loose, brown **SILTY SAND,** trace to some clay	1.50	SS	2	50	5	1	
Interlayered very loose, grey **SILTY SAND** and firm, grey **SILTY CLAY to CLAYEY SILT**	2.70					2	
Firm, grey **SILT YCLAY / CLAYEY SILT** with silty sand seams	6.40	TW	3			3 4 5 6	
End of Borehole (GWL @ 0.76m - May 3, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Terry Fox Drive
Ottawa, Ontario

DATUM

REMARKS

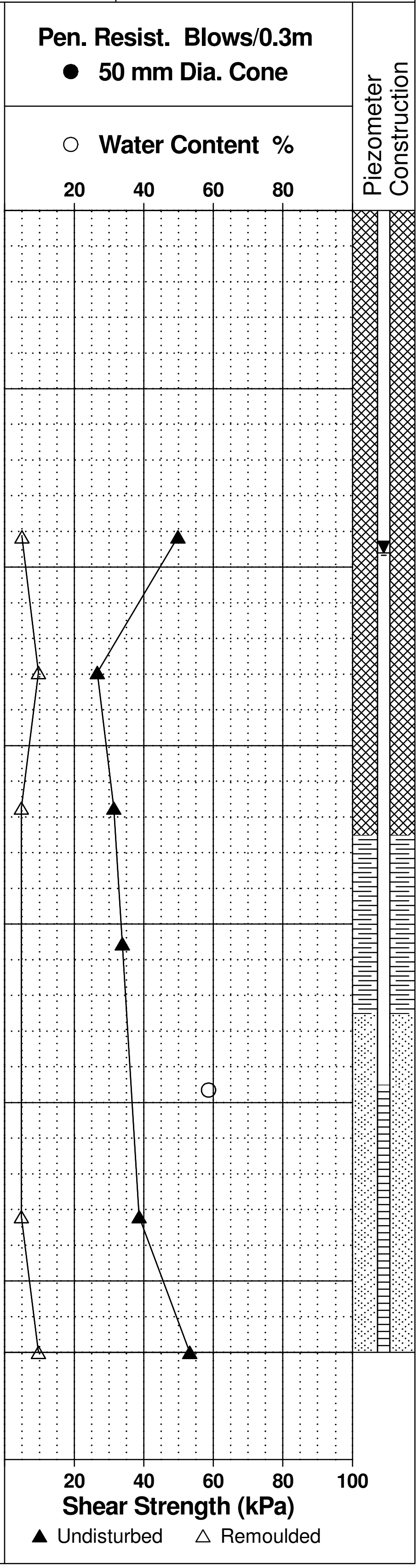
BORINGS BY CME 75 Power Auger

DATE April 20, 2018

FILE NO. **PG4466**

HOLE NO. **BH 3-18**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL 0.28		AU	1				
Compact to loose, brown **SILTY SAND,** trace to some silty clay 1.40		SS	2	71	2	1	
Interlayered very loose, grey **SILTY SAND** and firm, grey **SILTY CLAY / CLAYEY SILT** 2.50						2	
Firm, grey **SILTY CLAY to CLAYEY SILT** 6.40		TW	3			3 4 5 6	
End of Borehole (GWL @ 1.92m - May 3, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terry Fox Drive at Fernbank Road
Ottawa, Ontario

DATUM

REMARKS

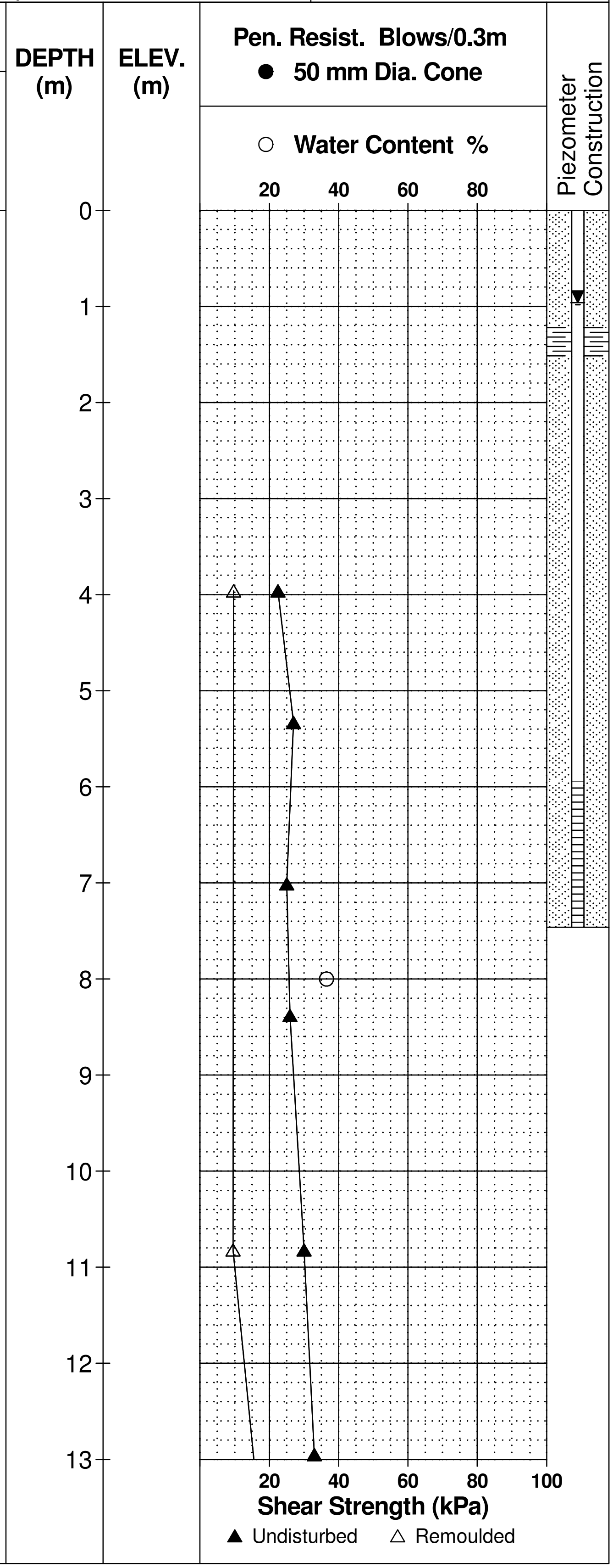
BORINGS BY CME 75 Power Auger

DATE April 27, 2006

FILE NO. PG0809

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30				
Compact to loose, brown **SILTY SAND**, trace clay	SS	1	50	12
1.73				
Interlayered very loose, grey **SILTY SAND** and firm, grey **CLAYEY SILT/SILTY CLAY**	SS	2	33	5
	SS	3	75	3
	SS	4	100	2
3.66				
Soft to firm, grey **SILTY CLAY/CLAYEY SILT** with silty sand seams	TW	5	100	
	SS	6	100	1
	TW	7	100	
	SS	8	100	1
	TW	9	100	



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terry Fox Drive at Fernbank Road
Ottawa, Ontario

DATUM

REMARKS

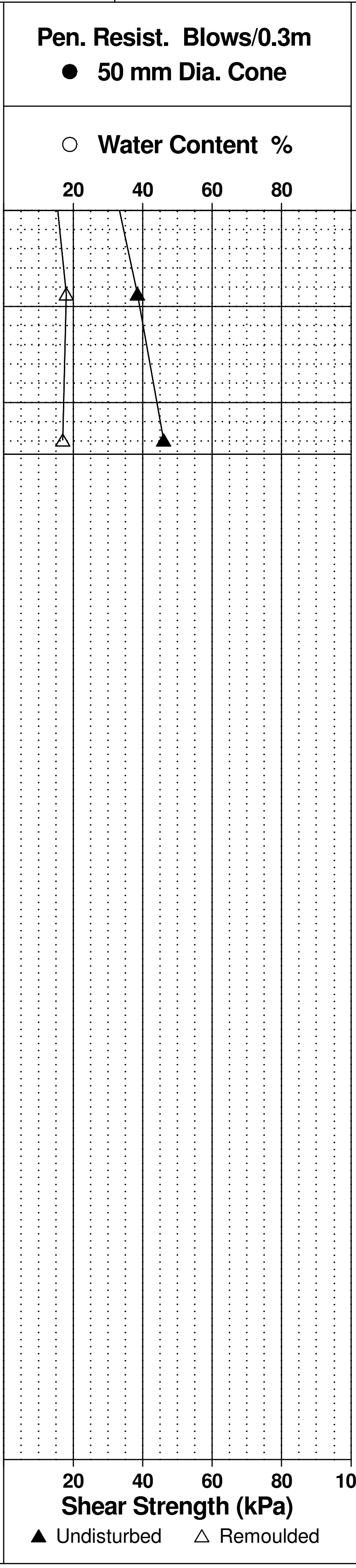
BORINGS BY CME 75 Power Auger

DATE April 27, 2006

FILE NO. PG0809

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						13		
Firm, grey **SILTY CLAY**	▨					14 15		
15.54								
End of Borehole (GWL @ 0.96m - May 5, 2006)								



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Geotechnical Investigation
Terry Fox Drive at Fernbank Road
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DATUM

REMARKS

BORINGS BY CME 75 Power Auger

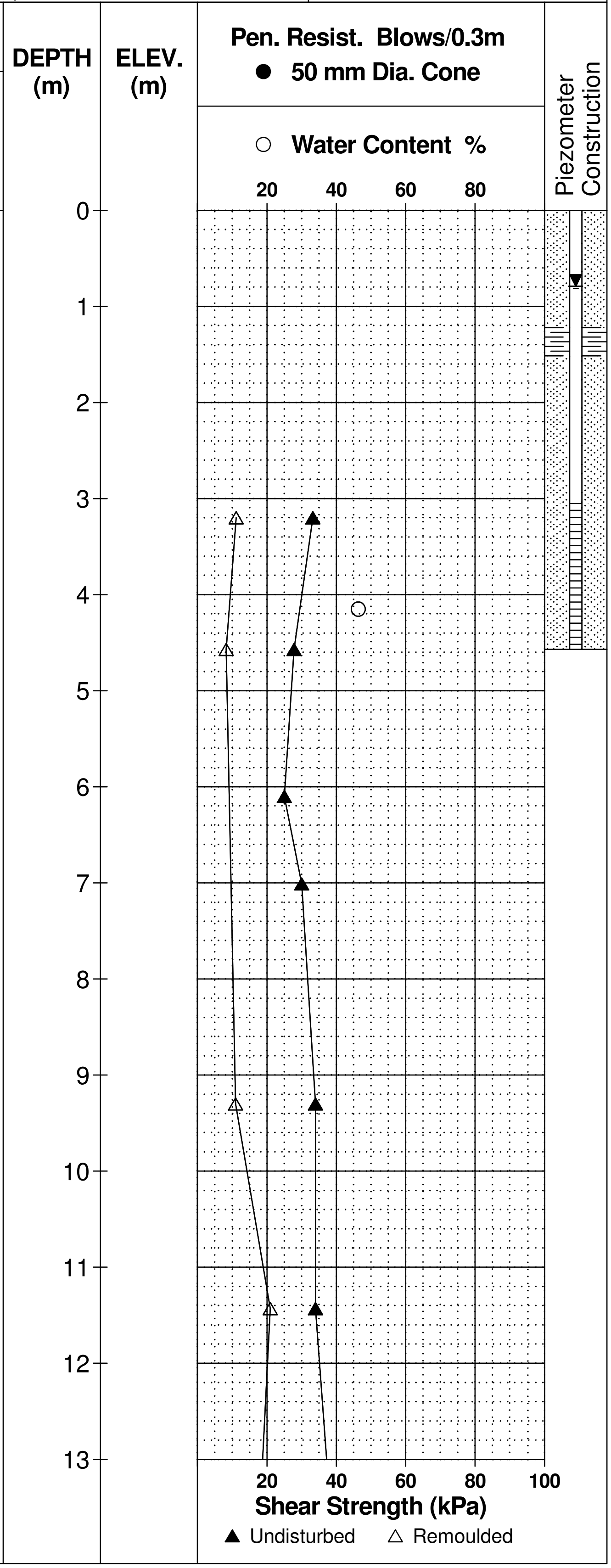
DATE April 27, 2006

FILE NO. PG0809

HOLE NO. BH 2

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.36
Compact, brown **SILTY SAND** - trace clay seams by 0.8m depth	1.20
Interlayered very loose, grey **SILTY SAND** and firm **CLAYEY SILT/SILTY CLAY**	2.59
Firm, grey **SILTY CLAY/CLAYEY SILT**, some to trace sand seams to 6.25m depth	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	25	10
SS	2	25	3
SS	3	100	1
TW	4	96	
TW	5	100	
SS	6	100	1
TW	7	100	
SS	8	100	1



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terry Fox Drive at Fernbank Road
Ottawa, Ontario

DATUM

REMARKS

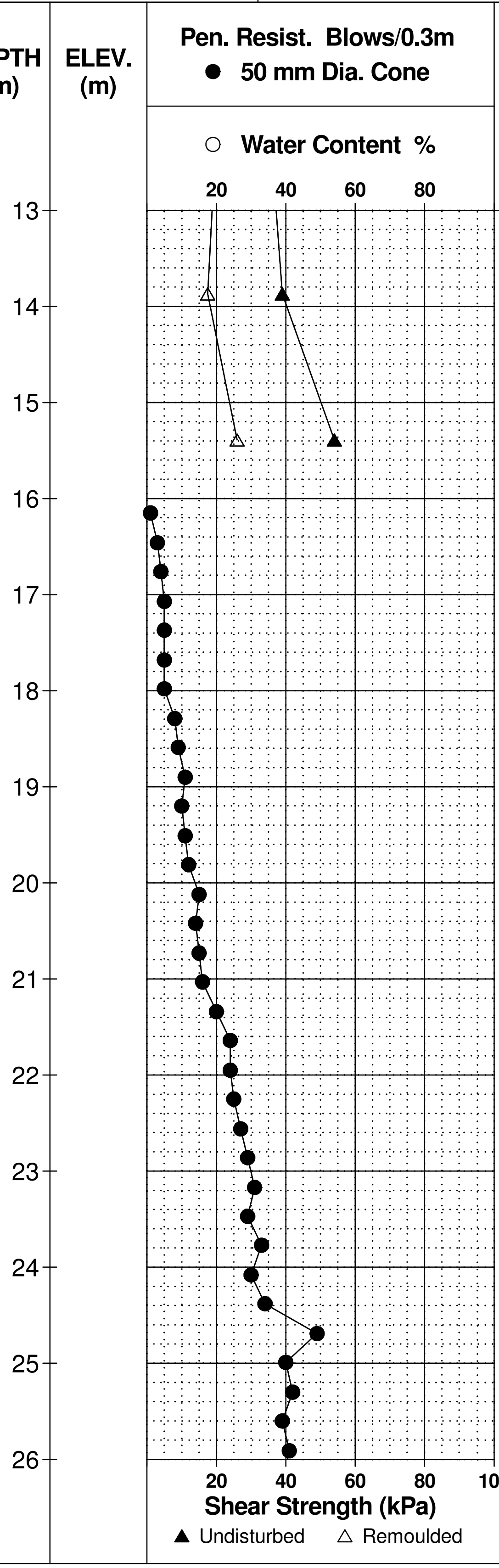
BORINGS BY CME 75 Power Auger

DATE April 27, 2006

FILE NO. PG0809

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
Firm to stiff, grey **SILTY CLAY**						13		
15.54								
Dynamic Cone Penetration Test commenced @ 15.54m depth								
Inferred **SILTY CLAY/CLAYEY SILT**								
24.00								
Inferred **GLACIAL TILL**						26		



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DATUM

REMARKS

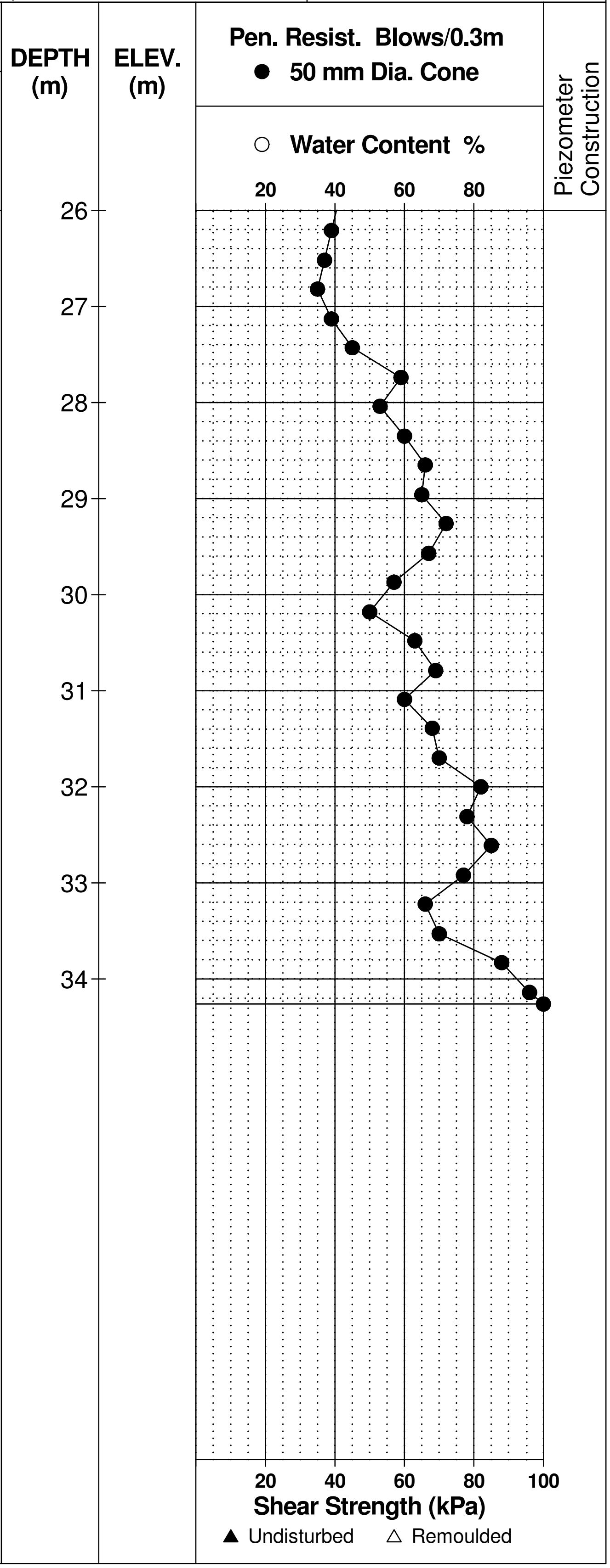
BORINGS BY CME 75 Power Auger

DATE April 27, 2006

FILE NO. PG0809

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
Inferred **GLACIAL TILL**						26–34		
34.26								
End of Borehole Practical DCPT refusal at 34.26m depth (GWL @ 0.79m - May 5, 2006)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terry Fox Drive at Fernbank Road
Ottawa, Ontario

DATUM

REMARKS

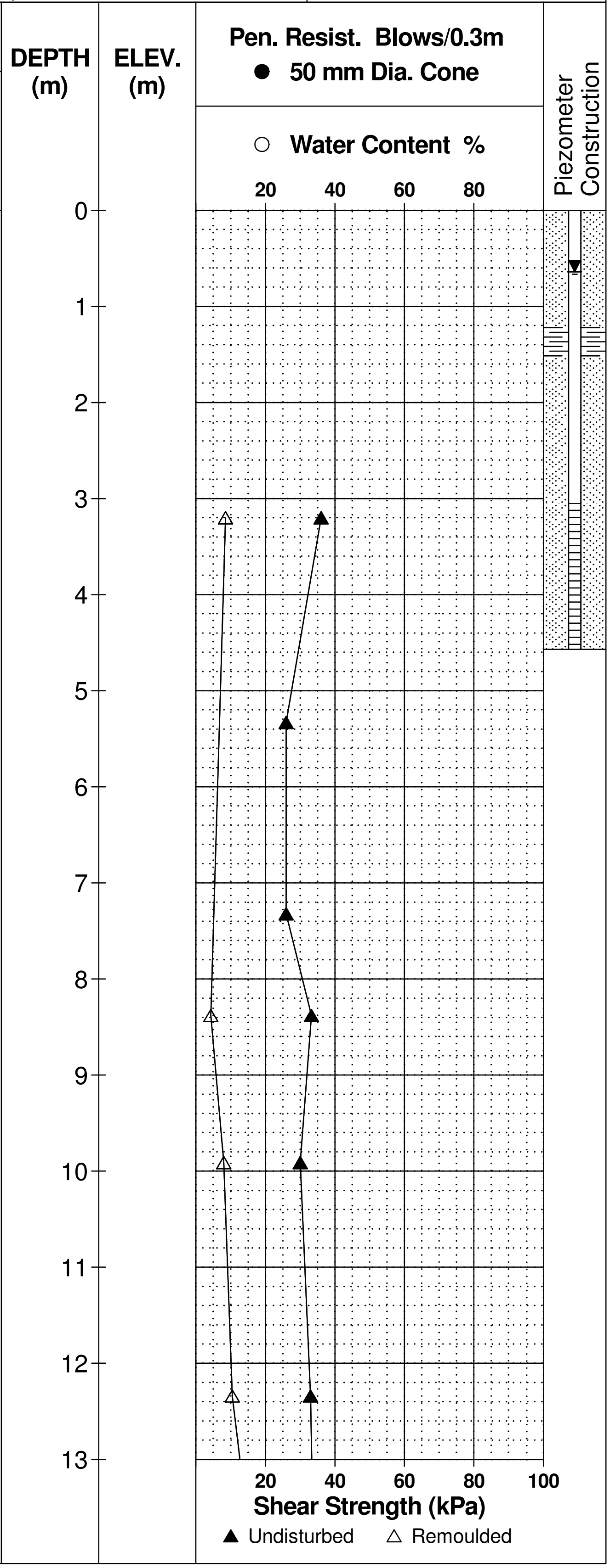
BORINGS BY CME 75 Power Auger

DATE April 26, 2006

FILE NO. PG0809

HOLE NO. BH 3

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.28	AU	1		
Loose, brown **SILTY SAND** with clay seams	SS	2	75	6
- grey by 1.2m depth 1.90	SS	3	54	5
Interlayered very loose, grey **SILTY SAND** and firm, grey **SILTY CLAY/CLAYEY SILT** 2.73	SS	4	50	3
Firm, grey **SILTY CLAY/CLAYEY SILT**, some sand seams to 7.0m depth	SS	5	100	1
	TW	6	100	
	TW	7	100	
	SS	8	100	1
	TW	9	100	
	SS	10	100	1



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REMARKS

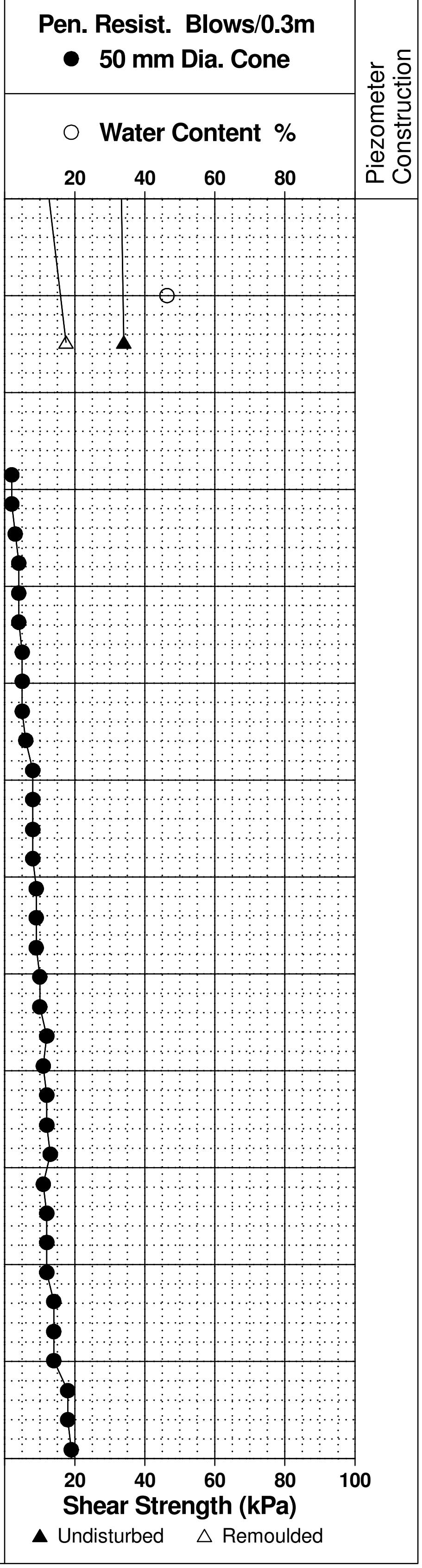
BORINGS BY CME 75 Power Auger

DATE April 26, 2006

FILE NO. PG0809

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						13	
Firm, grey **SILTY CLAY**		TW	11	100		14	
14.63							
Dynamic Cone Penetration Test commenced @ 14.63m depth						15	
						16	
						17	
						18	
Inferred **SILTY CLAY/CLAYEY SILT**						19	
						20	
						21	
						22	
						23	
						24	
						25	
						26	



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terry Fox Drive at Fernbank Road
Ottawa, Ontario

DATUM

REMARKS

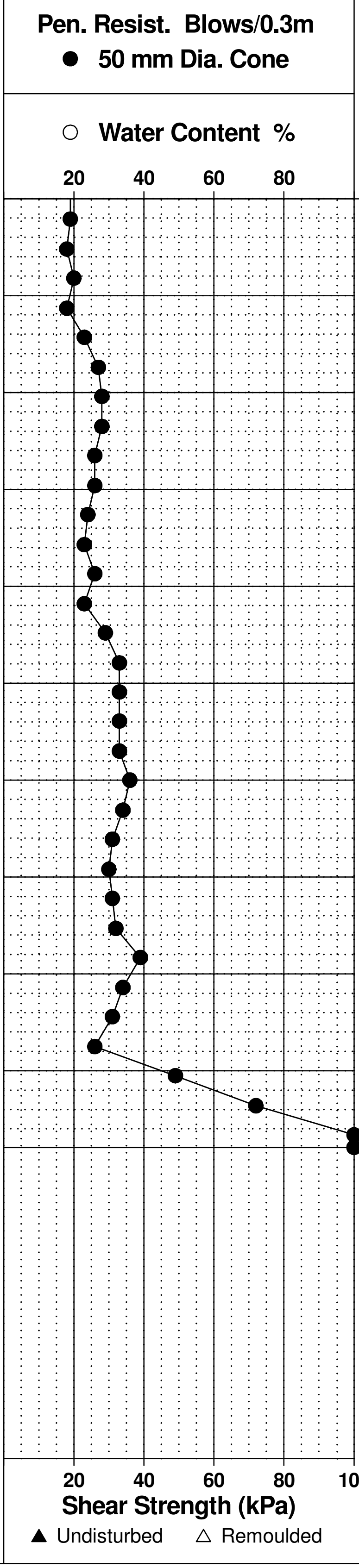
BORINGS BY CME 75 Power Auger

DATE April 26, 2006

FILE NO. PG0809

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						26	
Inferred **SILTY CLAY/CLAYEY SILT**						27 28 29 30	
31.00						31	
Inferred **GLACIAL TILL**						32 33 34 35	
35.79							
End of Borehole Practical DCPT refusal at 35.79m depth (GWL @ 0.64m - May 5, 2006)							



Piezometer Construction

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 = $(D30)^2 / (D10 \times D60)$
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)

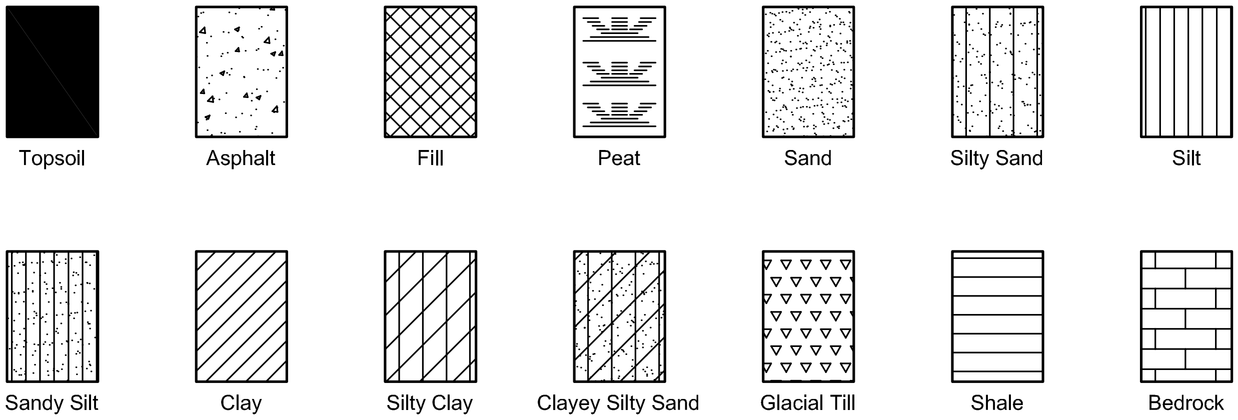
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

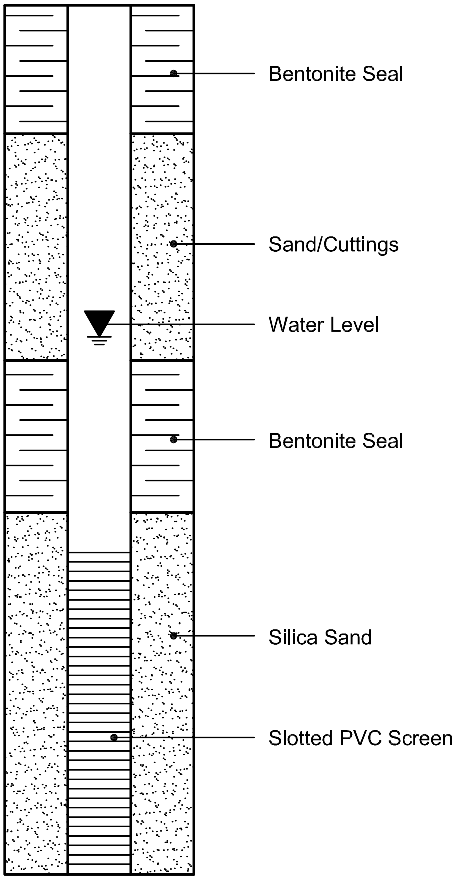
SYMBOLS AND TERMS (continued)

STRATA PLOT

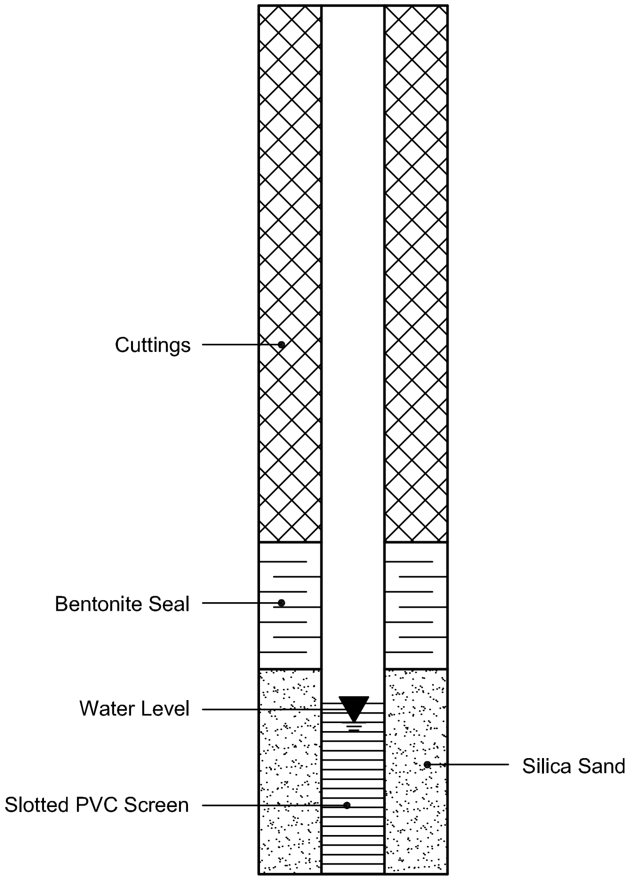


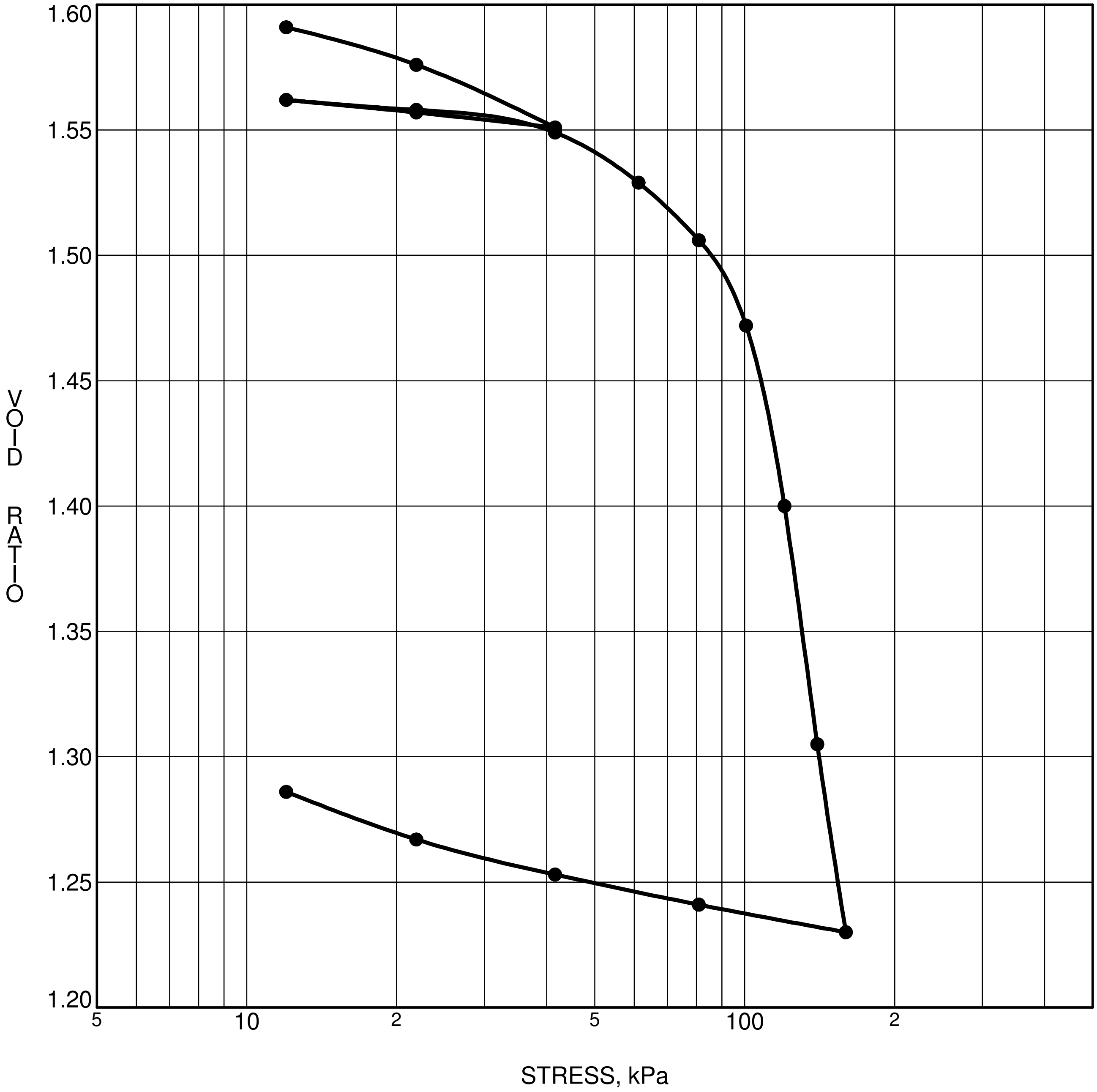
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1-18**	p'o	**47.22** kPa	Ccr	**0.024**
Sample No.	**TW 4**	p'c	**100.3** kPa	Cc	**1.428**
Sample Depth	**4.93** m	OC Ratio	**2.1**	Wo	**58.5** %
Sample Elev.	m	Void Ratio	**1.61**	Unit Wt.	**16.4** kN/m ^{3}

CLIENT **Claridge Homes**

FILE NO. **PG4466**

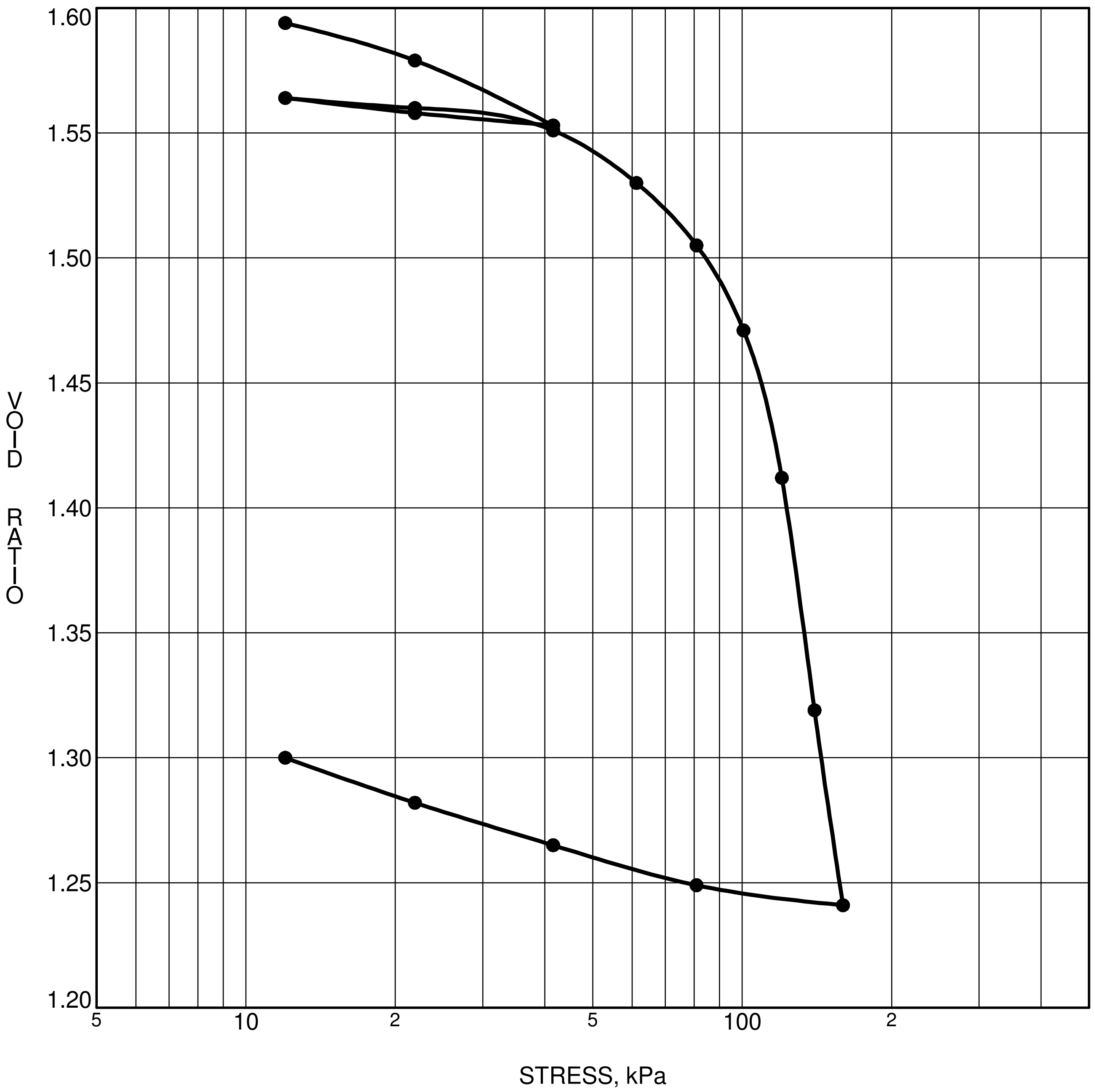
PROJECT **Geotechnical Investigation - Residential Development - Terry Fox Dr. at Fernbank Rd.**

DATE **14/05/2018**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



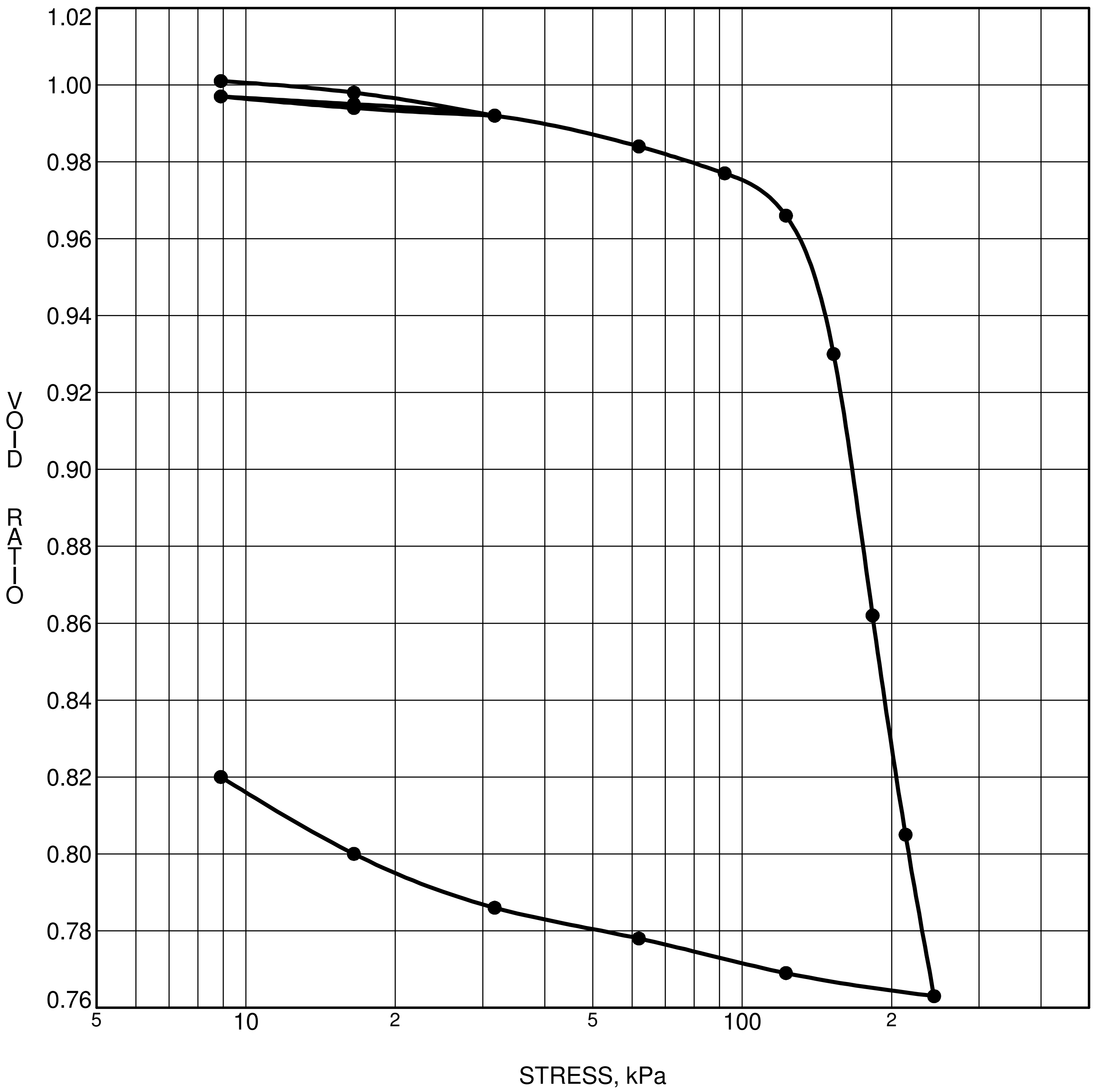
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3-18**	p'o	**47.22** kPa	Ccr	**0.024**
Sample No.	**TW 3**	p'c	**103.4** kPa	Cc	**1.409**
Sample Depth	**4.93** m	OC Ratio	**2.2**	Wo	**58.6** %
Sample Elev.	m	Void Ratio	**1.612**	Unit Wt.	**16.4** kN/m^3

CLIENT	**Claridge Homes**	FILE NO.	**PG4466**
PROJECT	**Geotechnical Investigation - Residential Development - Terry Fox Dr. at Fernbank Rd.**	DATE	**14/05/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



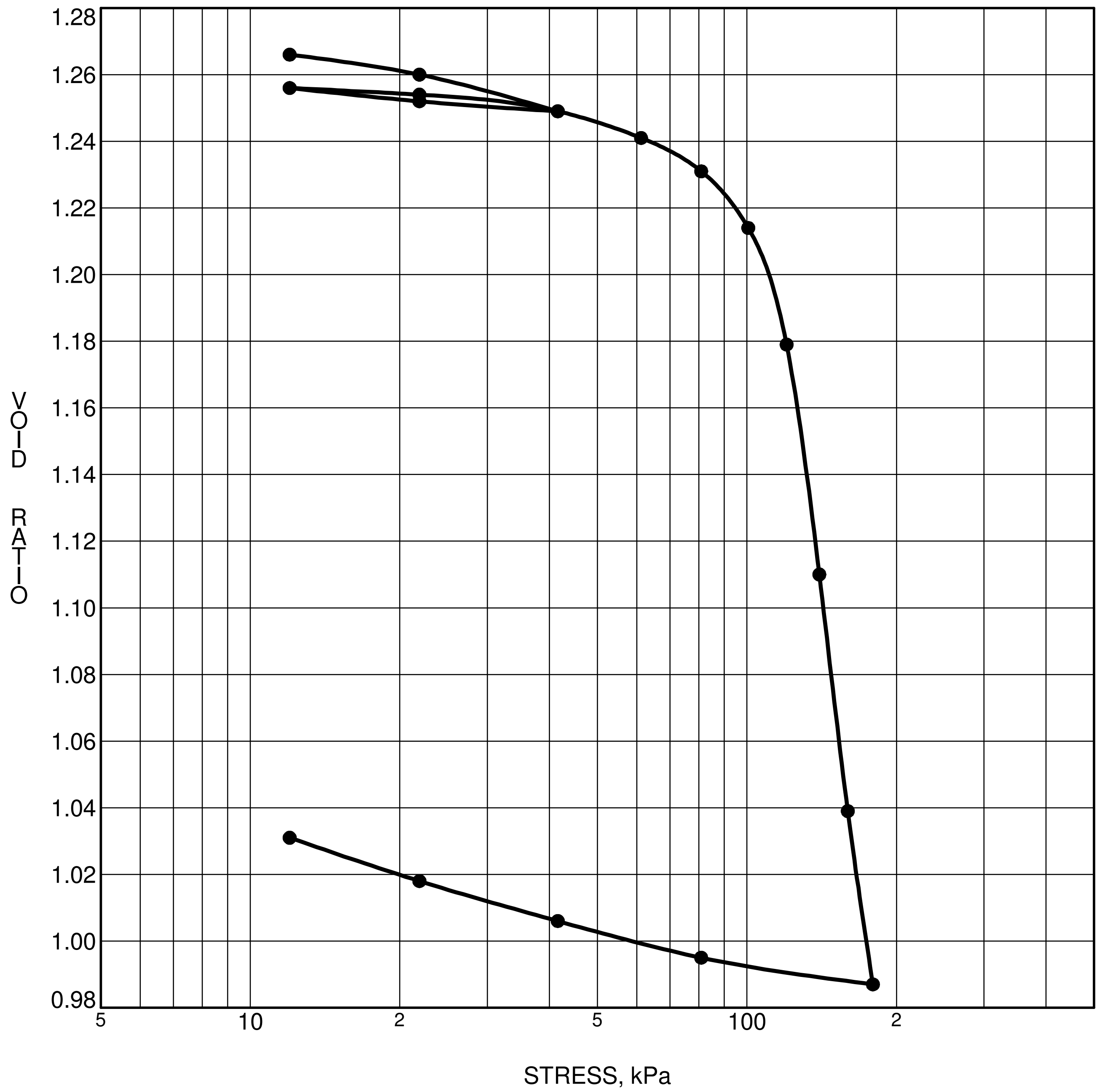
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1**	p'o	**87** kPa	Ccr	**0.010**
Sample No.	**TW 7**	p'c	**138** kPa	Cc	**0.882**
Sample Depth	**8.00** m	OC Ratio	**1.6**	Wo	**36.5** %
Sample Elev.	m	Void Ratio	**1.004**	Unit Wt.	**18.4** kN/m ^{3}

CLIENT	**Longwood Building Corporation**	FILE NO.	**PG0809**
PROJECT	**Geotechnical Investigation - Terry Fox Drive at Fernbank Road**	DATE	**23/05/2006**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY

Borehole No.	**BH 2**	p'o	**55** kPa	Ccr	**0.013**
Sample No.	**TW 4**	p'c	**110** kPa	Cc	**1.127**
Sample Depth	**4.15** m	OC Ratio	**2.0**	Wo	**46.3** %
Sample Elev.	m	Void Ratio	**1.274**	Unit Wt.	**17.4** kN/m ^{3}

CLIENT **Longwood Building Corporation**

FILE NO. **PG0809**

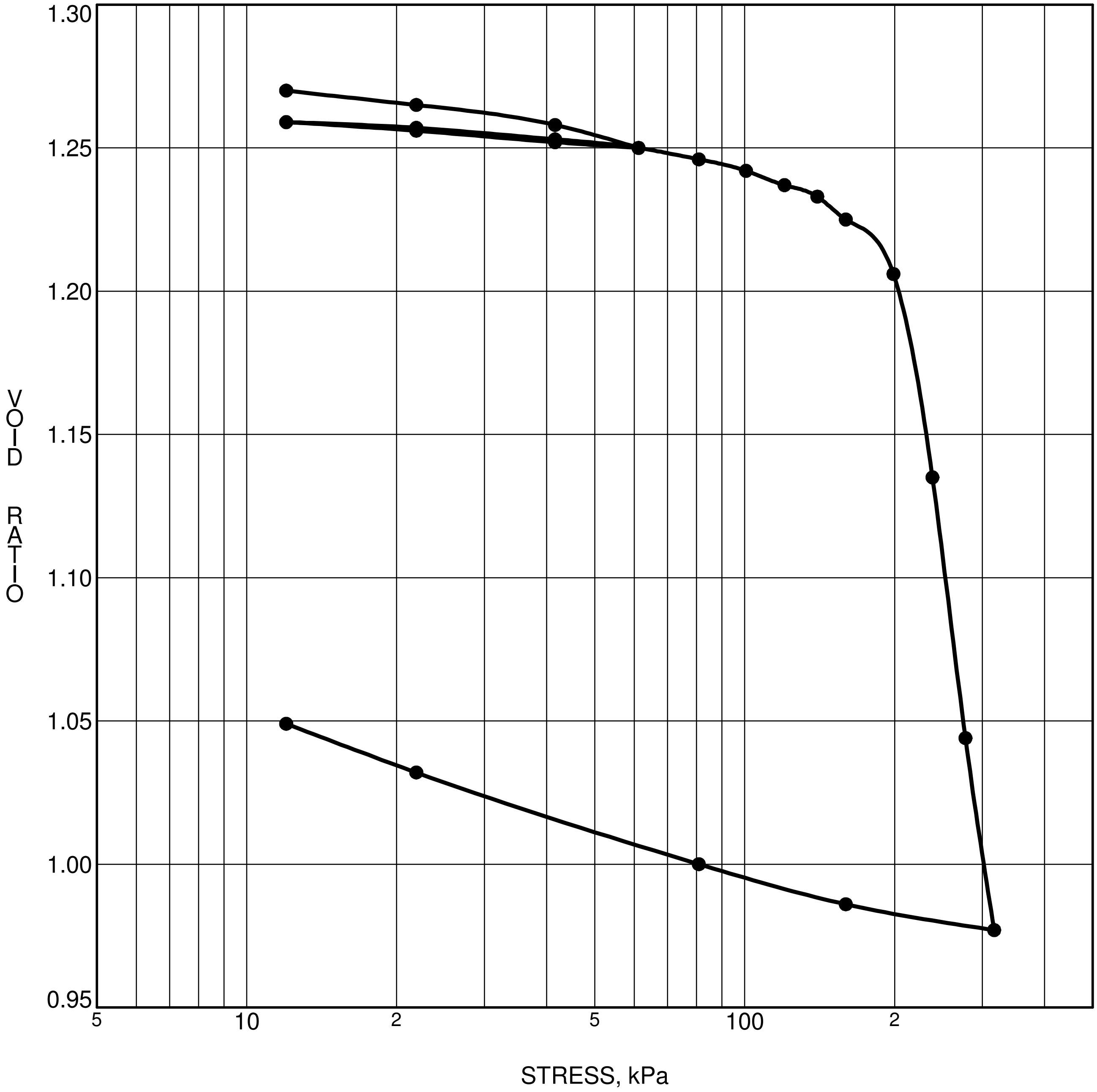
PROJECT **Geotechnical Investigation - Terry Fox Drive at Fernbank Road**

DATE **15/05/2006**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3**	p'o	**136** kPa	Ccr	**0.014**
Sample No.	**TW 11**	p'c	**200** kPa	Cc	**1.241**
Sample Depth	**14.00** m	OC Ratio	**1.5**	Wo	**46.3** %
Sample Elev.	m	Void Ratio	**1.274**	Unit Wt.	**17.4** kN/m ^{3}

CLIENT **Longwood Building Corporation**

FILE NO. **PG0809**

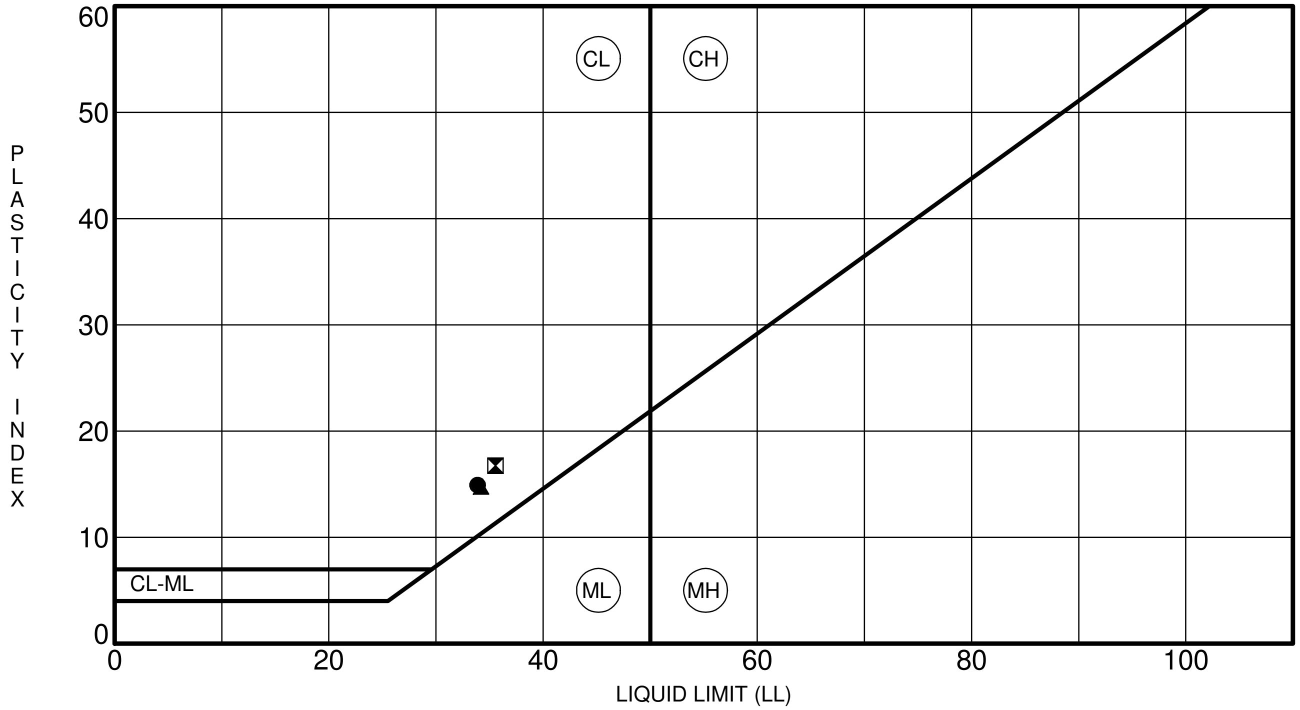
PROJECT **Geotechnical Investigation - Terry Fox Drive at Fernbank Road**

DATE **15/05/2006**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 1 TW 7**	**34**	**19**	**15**		**CL - Inorganic clays of low plasticity**
⧗	**BH 2 TW 4**	**36**	**19**	**17**		**CL - Inorganic clays of low plasticity**
▲	**BH 3 TW 11**	**34**	**20**	**15**		**CL - Inorganic clays of low plasticity**

CLIENT **Longwood Building Corporation**

PROJECT **Geotechnical Investigation - Terry Fox Drive at Fernbank Road**

FILE NO. **PG0809**

DATE **26 Apr 06**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Report Date: 08-May-2006
Order Date: 01-May-2006

Client: **Paterson Group Inc.**
Client PO: **2884**
Project: **PG0809**

Matrix: Soil

	Sample ID:	BH3 SS3
	Sample Date:	26/04/2006
Parameter	MDL/Units	L5774.1
Chloride	5 ug/g	140
Sulphate	5 ug/g	60
pH	0.05 pH units	9.15
Resistivity	0.1 ohm.m	25

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4466-1 - TEST HOLE LOCATION PLAN

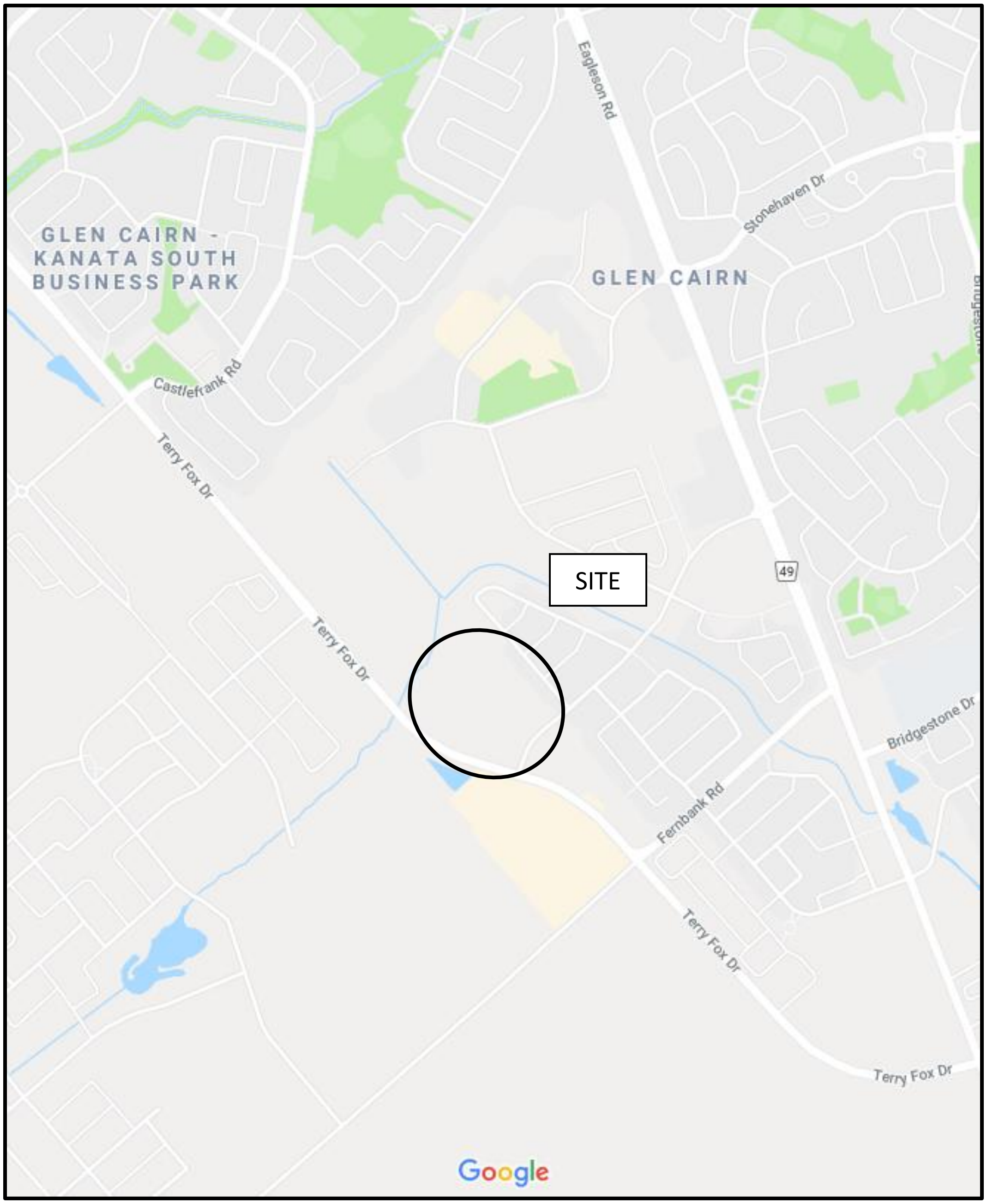


FIGURE 1 - KEY PLAN

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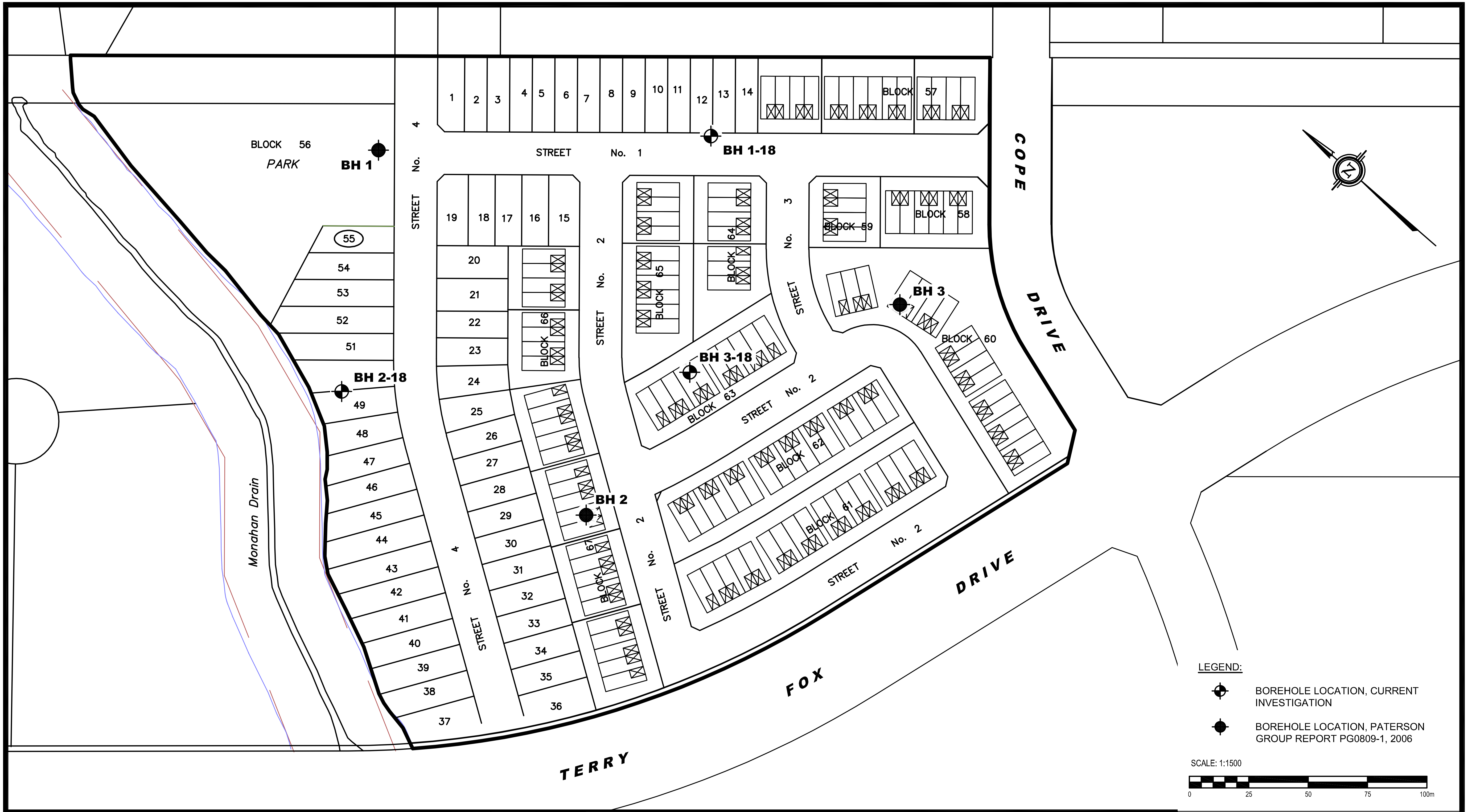

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
TERRY FOX DRIVE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500
Date: 05/2018
Drawn by: MPG
Report No.: PG4466-1
Checked by: RG
Dwg. No.: PG4466-1
Approved by: DJG
Revision No.: 0

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