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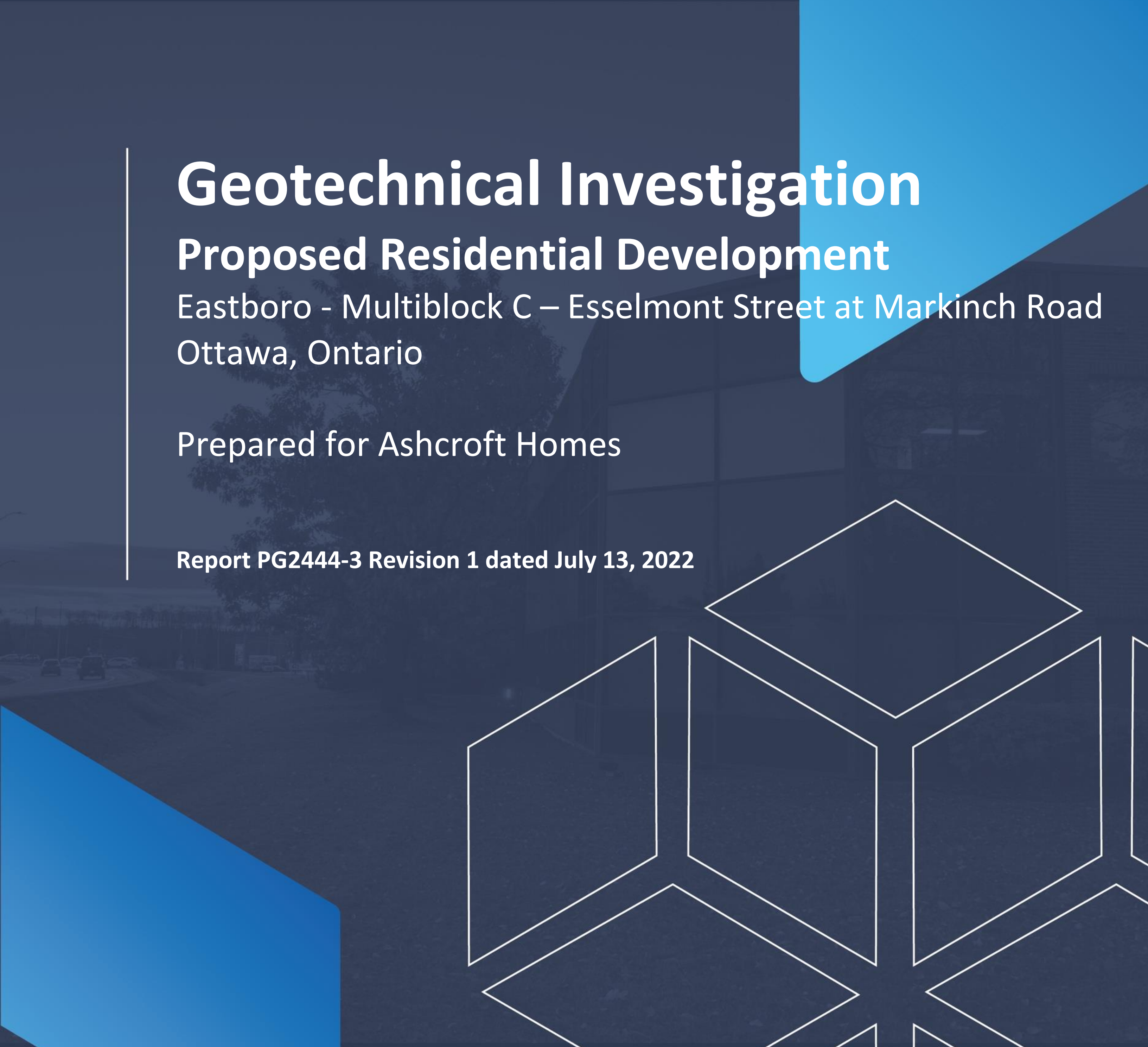

Geotechnical Investigation

Proposed Residential Development

Eastboro - Multiblock C – Esselmont Street at Markinch Road
Ottawa, Ontario

Prepared for Ashcroft Homes

Report PG2444-3 Revision 1 dated July 13, 2022

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ashcroft Homes to conduct a geotechnical investigation for the proposed residential development to be located at Multiblock C, within the Eastboro residential development, along Esselmont Street at Markinch Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes and available soils information.

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

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

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

2.0 Proposed Development

It is understood that the proposed development will consist of single-family residential dwellings as well as townhouse style residential buildings with associated local roadways, access lanes and landscaped areas. Municipal services are also anticipated as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current supplemental geotechnical investigation was carried out at Multiblock C by Paterson on May 24, 2022. At that time, a total of 3 boreholes were advanced to a maximum depth of 7.3 m below the existing ground surface. Two of the new boreholes are within the confines of the Multiblock C while the third borehole is in close proximity to the site's boundaries.

Previous geotechnical investigations were completed by this firm between 2009 and 2014 within the subject block and surrounding areas. A total of four (4) boreholes and seven (7) test pits were advanced to maximum depths of 9.6 and 4.0 m, respectively, during the historical investigations within the confines of Multiblock C. Previous investigations were also completed in proximity to the subject block by others between 2007 and 2008 and consisted of advancing 3 boreholes to a maximum of depth of 11.6 m below existing grade.

The supplemental borehole locations were distributed in a manner to provide general coverage of the subject block, taking into consideration existing site features and underground utilities. The locations of test holes are shown on Drawing PG2444-5 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted auger drilling rig operated by a two-person crew while the test pits were excavated using a hydraulic shovel. The test hole procedure consisted of augering or excavating 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.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using a 50 mm diameter split-spoon sampler, a 73 mm diameter thin walled Shelby tube in conjunction with a piston sampler, or the auger flights. All soil samples were classified on site. The split-spoon and auger samples were placed in sealed plastic bags, while the Shelby tubes were sealed at both ends on site. All samples were transported to the laboratory. Upon recovery, all soil samples were immediately sealed in appropriate containers to facilitate the preliminary screening procedure.

The depths at which the split-spoon, Shelby tube, and auger samples were recovered from the test holes are shown as “SS”, “TW” and “AU”, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

Soil samples were recovered from the sidewalls of the test pits. All soil samples were visually inspected and classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the soil samples were recovered from the test pits are shown as “G” on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

Overburden thickness was also evaluated during the course of the 2011 investigation by dynamic cone penetration testing (DCPT) at BH 22. 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 in the boreholes and test pits were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed at boreholes BH 11-22, BH 12-22 and BH 13-22 during the supplemental investigation in order to permit monitoring of the groundwater levels. Flexible polyethylene standpipes were installed within all historical borehole locations subsequent to the completion of the sampling program. Additionally, the depth at which groundwater infiltration was encountered through the sidewalls of the test pits was recorded prior to the completion of excavation as noted in the field.

Sample Storage

All samples from the supplemental investigation 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 supplemental borehole locations were selected by Paterson to provide general coverage of Multiblock C, taking into consideration existing site features and underground utilities. The borehole locations, and ground surface elevation at each borehole location were surveyed by Paterson using a high precision GPS unit with respect to a geodetic datum.

The test hole locations and ground surface elevation at the historical test holes were recovered in the field by Annis O'Sullivan Vollebekk. The ground surface elevations were referenced to a geodetic datum. The ground surface elevation and location of the test holes are presented on Drawing PG2444-5 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

The soil samples recovered from the previous investigations were visually examined in our laboratory to review the results of the field logging. A total of four (4) Atterberg Limits test and one (1) grains size distribution analysis were carried out on samples collected during the supplemental and historic geotechnical investigations. The test results are included in Appendix 1 and further discussed in Subsection 4.2.

3.4 Analytical Testing

Five (5) soil samples were submitted for analytical testing from within the subject site and the surrounding areas of Multiblock C during the previous investigations to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

Generally, the subject site is undeveloped, grass covered with scattered bushes, small trees, several patches of densely populated mature trees. The ground surface across the subject site is relatively flat and at grade with the surrounding roadways and developed areas.

The site is bordered by residential dwellings and roadways to the north, agricultural lands to the west, south and east.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil conditions encountered at the test hole locations consist of topsoil followed by compact to loose brown silty sand. A thin layer of brown silty clay crust was encountered underlying the above noted layers followed by a grey firm silty clay deposit. Practical refusal to DCPT was encountered at a depth of 38.08 m below the existing grade in BH 22.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test pit location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of limestone of the Lindsay Formation and/or shale of the Billings Formation with an overburden drift thickness of 25 to 50 m depth.

Laboratory Testing

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at all borehole locations during the supplemental investigation and on select samples collected during the historic geotechnical investigations. Based on the Atterberg Limits testing results within, and in proximity to, Multiblock C, the silty clay test samples near to subject site were classified as Inorganic Clays of High Plasticity (CH). The results of the Atterberg Limits testing are presented in Table 1 on the following page and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 – Atterberg Limits Results

Test Hole	Sample	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Moisture Content (%)	Classification
BH 11-22	SS3	1.8	78	23	55	71	CH
BH 12-22	SS4	2.6	37	18	19	73	CH
BH 13-22	SS3	1.8	69	19	50	58	CL
BH 1-14	TW3	2.6	77	26	52	82	CH

Notes: CL: Inorganic Clay of Low Plasticity; CH: Inorganic Clay of High Plasticity

Grain size distribution analysis was completed on one (1) soil sample during the supplemental geotechnical investigation. The results of the grain size distribution analysis are summarized in Table 2 below and presented on the Grain Size Distribution Results sheets in appendix 1.

Table 2 – Summary of Grain Size Distribution

Test Hole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 13-22	SS4	2.6	0.0	14.4	39.1	46.5

Consolidation Testing

Three (3) samples were submitted by Paterson for consolidation testing within and in proximity to Multiblock. One (1) sample was submitted for consolidation testing by others. The test results are presented in Subsection 5.3 and on the Consolidation Test sheets in Appendix 1.

4.3 Groundwater

Groundwater infiltration levels recorded at each test hole location from the previous investigations are presented in Table 3. The long term groundwater level can also be estimated based on the moisture levels, colouring and consistency of the recovered soil samples. It is important to note that groundwater readings at piezometers can be influenced by surface water perched within the borehole backfill material. Based on these observations, the long term groundwater table is anticipated to be at 1.5 to 2.5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, groundwater level could vary at the time of construction.

Table 3 – Summary of Groundwater Levels				
Borehole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Recording Date**
		Depth (m)	**Elevation (m)**	
Groundwater Levels based on Monitoring Well Readings - Supplemental Investigation				
BH 11-22	85.31	1.72	83.59	June 17, 2022
BH 12-22	86.13	1.37	84.76	June 17, 2022
BH 13-22	85.94	1.22	84.72	June 17, 2022
Groundwater Levels based on Piezometer Readings and Field Observations - Previous Investigation (Report PG2444-2 dated October 15, 2014)				
BH 1-14	87.74	0.80	86.94	June 17, 2014
BH 2-14	86.41	0.06	86.35	September 2, 2014
Groundwater Levels based on Piezometer Readings - Previous Investigation (Report PG2444-1 Revision 2 dated November 3, 2017)				
BH 22	86.08	2.36	83.72	January 9, 2012
Groundwater Levels based on Field Observations - Previous Investigation (Report PG1829-1R dated January 13, 2010)				
TP 8-09	86.03	0.9	85.13	August 17, 2009
TP 10-09	86.47	1.6	84.87	August 17, 2009
TP 11-09	86.11	1.5	84.61	August 17, 2009
TP 15-09	-	1.8	n/a	October 28, 2009
TP 16-09	-	1.8	n/a	October 28, 2009
TP 17-09	-	1.8	n/a	October 28, 2009
TP 19-09	-	2.7	n/a	October 28, 2009
Groundwater Levels based on Piezometer Readings and Field Observations by Others				
BH 08-4	86.30	0.96	85.34	October 31, 2008
BH 07-5	86.91	1.83	85.08	July 24, 2007
BH 07-7	86.28	2.20	84.08	August 9, 2007

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed building be founded on conventional style shallow foundations placed on an undisturbed, compact silty sand, stiff to firm silty clay bearing surface and/or engineered fill.

Due to the presence of a silty clay deposit, the subject site will be subjected to a permissible grade raise restriction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and deleterious fill, such as material containing high content of organic materials, should be stripped from under the proposed buildings footprint and other settlement sensitive structures.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, stiff brown silty clay bearing or on engineered fill placed directly over the undisturbed, stiff brown silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa** incorporating a geotechnical factor of 0.5 at ULS.

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, firm silty clay bearing or on engineered fill placed directly over the undisturbed, firm silty clay can be designed using a bearing resistance value at SLS of **60 kPa** and a factored bearing resistance value at ULS of **100 kPa** incorporating a geotechnical factor of 0.5 at ULS.

Conventional footings placed on an undisturbed, compact silty sand bearing surface, or on engineered fill placed directly over the undisturbed, compact silty sand can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **150 kPa** incorporating a geotechnical factor of 0.5 at ULS. Where silty sand is found in a loose state of compactness, it is recommended that the silty sand be proof rolled using suitable vibratory equipment, making several passes, under dry conditions and above freezing temperatures and approved by Paterson at the time of construction.

Footings placed on a soil bearing surface and designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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 silty sand or silty clay 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 / 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. Three (3) site specific consolidation tests were conducted. One (1) consolidation test was also completed by others. The results of the consolidation tests are presented in Table 4 below 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 (*)
BH 1-14	TW3	2.74	75	35.1	0.040	4.332	A
BH 2-14	TW5	5.82	73	58.1	0.035	2.696	A
BH 4-14	TW2	2.64	58	32.6	0.046	4.502	A
BH 08-4	3	3.66	60	34	0.048	4.50	n/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. To determine the p'_o values, 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.

A **permissible grade raise restriction of 0.5 m** is recommended above original grades within 5 m of the proposed building. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

Based on the above discussion, if the proposed grading exceeds the permissible grade raise restrictions provided herein, 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 12 or 15 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.

As an alternative to lightweight fill in the interior of the garage and porch, a structural slab can be designed to create a void beneath the floor slab and therefore reduce fill-related loads. Additional information can be provided once the design of the buildings is known.

Option 2 - Preloading or Surcharging

It is possible to preload or surcharge the subject 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 and electronic piezometers will have to be implemented. This program will determine the amount of settlement in the preloaded or surcharged areas. 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.

Underground Utilities

The underground services may be subjected to unacceptable total or differential settlements. In particular, the joints at the interface building/soil may be subjected to excessive stress if the differential settlements between the building and the services are excessive. This should be considered in the design of the underground services.

Once the required grade raises are established, the above options could be further discussed along with further recommendations on specific requirements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations at the subject 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 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the compact silty sand or stiff to firm silty clay will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. Soft or poor performing areas should be sub-excavated and replaced with OPSS Granular A or Granular B Type II and compacted to 98% of the material’s SPMDD. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

5.6 Pavement Design

For design purposes, the pavement structures presented in the following tables are recommended for the design of car only parking areas, local residential roads, and heavy-duty access lanes.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	4Material 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
- **Minimum Performance Graded (PG) 58-34** asphalt cement should be used for this project.

Table 6 - Recommended Pavement Structure - Local Residential Roadways and Access Lanes

Thickness (mm)	4Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II

- SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill
- **Minimum Performance Graded (PG) 58-34** asphalt cement should be used for this project.

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 fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill
- **Minimum Performance Graded (PG) 58-34** asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terrafix 200W or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

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 shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended for each proposed structure. 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 which is 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.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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.

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

6.3 Excavation Side Slopes

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

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil 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.

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

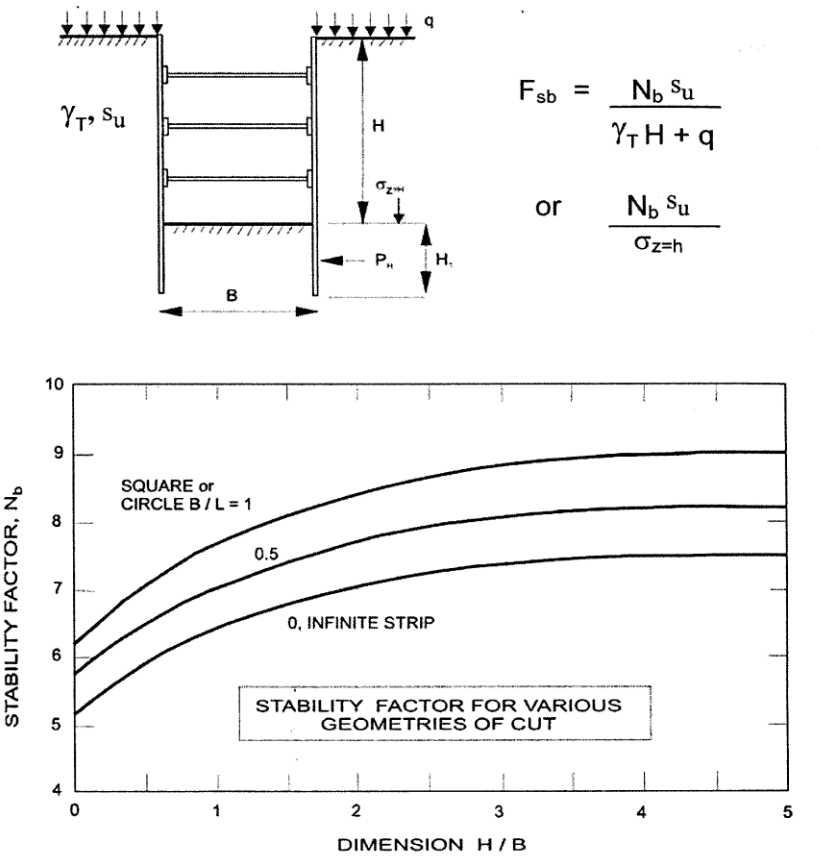


Figure 1 - Stability Factor for Various Geometries of Cut

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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes should consist of a minimum of 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 a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent 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 a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay 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.

Clay Seals

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

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

Permit to Take Water

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the shallow excavation. 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.

A temporary Ministry of the Environment, Conservation and Parks (MECP) 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 MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically 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 MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building’s perimeter or sub-slab drainage system will be directed to the proposed building’s sump pit (is proposed). It is expected that groundwater flow will be low to medium with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Adverse Impacts on Neighboring Structures

Based on our geotechnical analysis, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to minimal temporary groundwater lowering.

The neighboring structures are expected to be founded within the native silty clay bearing surface. No issues are expected with respect to groundwater lowering that would cause long-term damage to adjacent sound structures surrounding the proposed building. However, underpinning requirements for adjacent structures should be evaluated at the time of excavation.

6.6 Winter Construction

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

The results on analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland Cement would be appropriate for the subject 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 in indicative of an aggressive to very aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the subject multiblock to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) for trees planted within a public right-of-way (ROW). Atterberg limits testing was completed for recovered silty clay samples during the supplemental geotechnical investigation as well as during the historical geotechnical investigations. Grain size distribution analysis was also completed on 1 soil sample. The above noted test results were completed on samples taken at depths between the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

A medium to high sensitivity clay soil was encountered between anticipated underside of footing elevations and 3.5 m below preliminary finished grade as per City Guidelines at the subject site. Based on our Atterberg Limits' test results, the modified plasticity limits generally exceed 40% for the majority of the boreholes across the subject multiblock development. Therefore, the following tree planting setbacks are recommended for the medium to high sensitivity area.

Large trees (mature height over 14 m) can be planted within this area 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). A tree planting setback limit of **7.5 m** is applicable 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. **It should be noted that where the footings are proposed at a shallower depth, a combination of**

engineered fill and/or root barrier system can be designed to accommodate a reduced footing depths which can be discussed in a separate report upon completion of the design grading plans.

☐ 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 surrounding 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.

Aboveground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered to be acceptable for in-ground 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.

Additional grading around the hot tub should not exceed permissible grade raises. Hot tubs should be placed at least 2 m away from the nearest foundation wall to minimize additional weight on the foundation walls and the underlying stiff to firm silty clay bearing surface. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

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.

☐ Once Available, a review of the final grading plan should be completed from a geotechnical perspective.

☐ Once Available, a review of the landscaping plan should be completed from a geotechnical perspective due to the presence of high sensitivity silty clay deposit within the subject development.

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

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 13, 2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Kevin Pickard, EIT

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ☐ Ashcroft Homes (e-mail copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

RECORD OF BOREHOLE LOGS BY OTHERS

CONSOLIDATION TESTING RESULTS

CONSOLIDATION TESTING RESULTS BY OTHERS

ATTERBERG LIMITS' TESTING RESULTS

GRAIN SIZE DISTRIBUTION ANALYSIS RESULTS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development - Eastboro Phase 2
Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

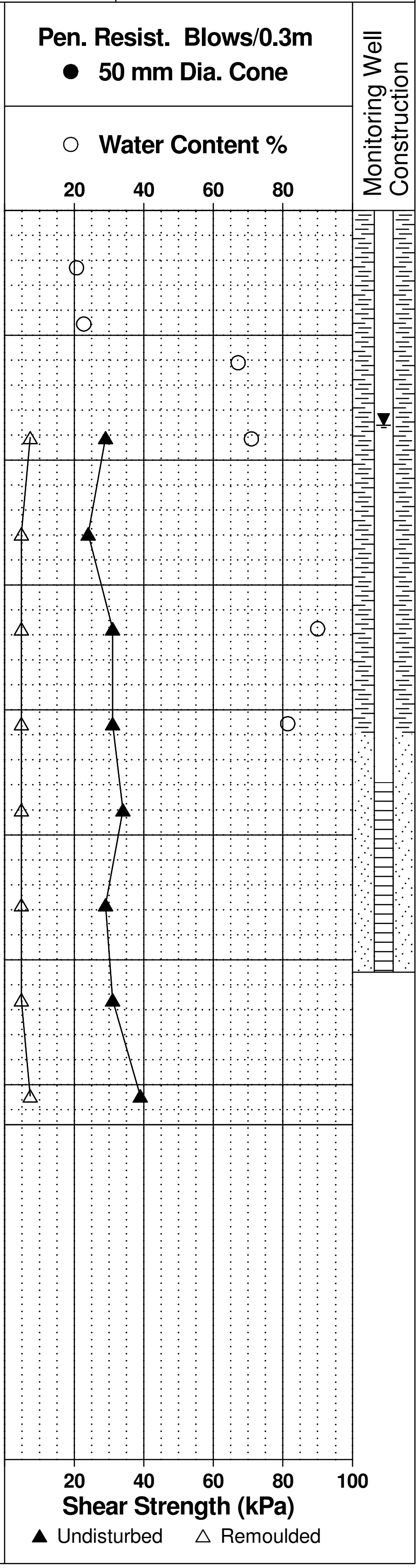
BORINGS BY Track-Mount Power Auger

DATE May 20, 2022

FILE NO. **PG2444**

HOLE NO. **BH11-22**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	85.31
Loose, brown **SILTY SAND** - grey by 0.6m depth	AU	1				
0.91	SS	2	50	4	1	84.31
Soft to firm, grey **SILTY CLAY**	SS	3	58		2	83.31
	SS	4	0			
	SS	5	100		3	82.31
	SS	6	100		4	81.31
					5	80.31
					6	79.31
					7	78.31
7.32						
End of Borehole (GWL @ 1.72m - June 17, 2022)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development - Eastboro Phase 2
Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

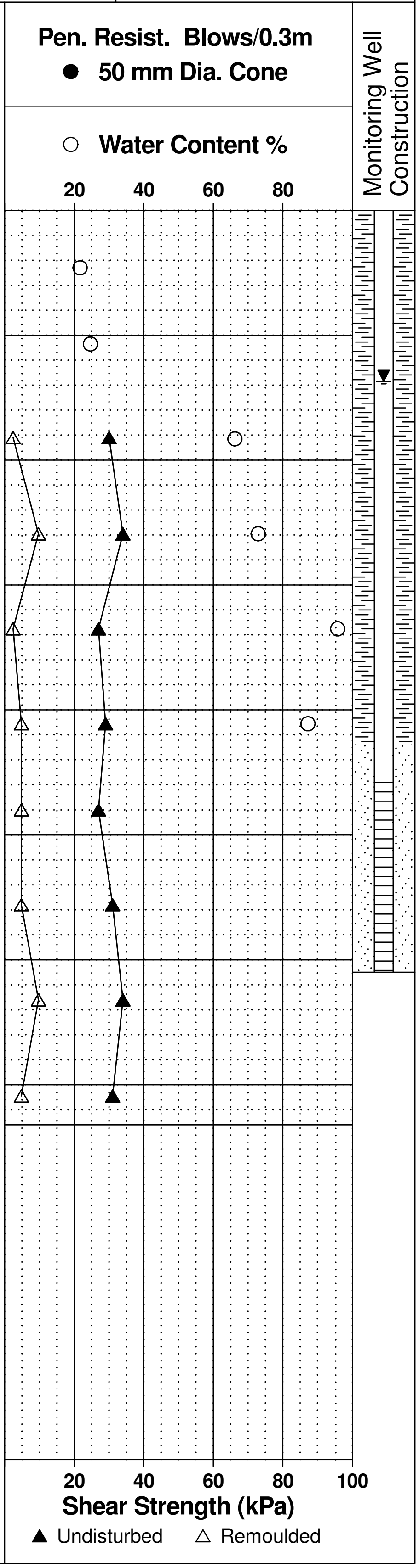
BORINGS BY Track-Mount Power Auger

DATE May 24, 2022

FILE NO. **PG2444**

HOLE NO. **BH12-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.13
TOPSOIL 0.18							
Compact, brown **SILTY SAND** - grey by 0.7m depth		AU	1				
		SS	2	50	11	1	85.13
1.45							
Firm, grey **SILTY CLAY**		SS	3	75		2	84.13
		SS	4	83			
		SS	5	100		3	83.13
		SS	6	100		4	82.13
						5	81.13
						6	80.13
						7	79.13
7.32							
End of Borehole (GWL @ 1.37m - June 17, 2022)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development - Eastboro Phase 2
Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

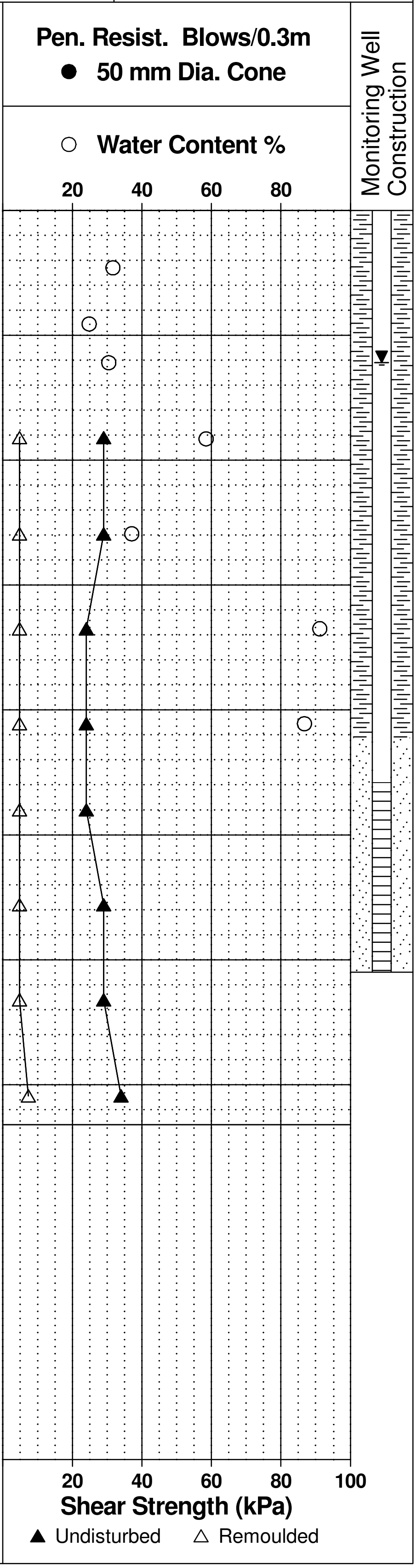
BORINGS BY Track-Mount Power Auger

DATE May 24, 2022

FILE NO. **PG2444**

HOLE NO. **BH13-22**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	85.94
Loose to compact, brown **SILTY SAND**	AU	1				
- grey by 0.9m depth	SS	2	50	10	1	84.94
1.52						
Soft to firm, grey **SILTY CLAY**	SS	3	100			
					2	83.94
	SS	4	58			
					3	82.94
	SS	5	100			
	SS	6	100		4	81.94
					5	80.94
					6	79.94
					7	78.94
7.32						
End of Borehole (GWL @ 1.22m - June 17, 2022)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

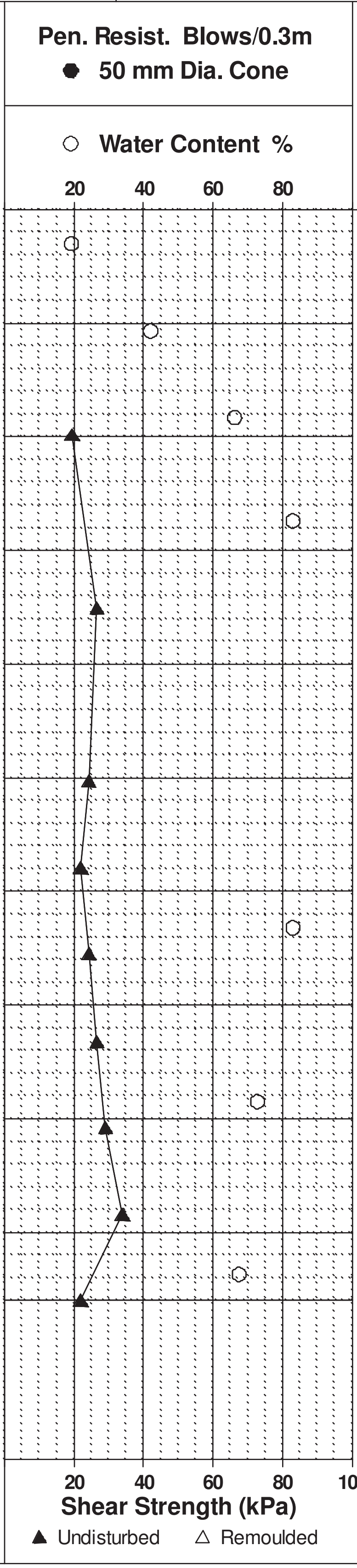
BORINGS BY CME 55 Power Auger

DATE June 17, 2014

FILE NO. PG2444

HOLE NO. BH 1-14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	87.74	
Loose, brown **SILTY SAND**		AU	1					
		SS	2	67	4	1	86.74	∇
1.37								
Soft to firm, brown **SILTY CLAY**						2	85.74	
		TW	3	100				
- grey by 3.0m depth						3	84.74	
		TW	4	100		4	83.74	
						5	82.74	
						6	81.74	
						7	80.74	
						8	79.74	
						9	78.74	
9.60								
End of Borehole (GWL @ 0.8m depth based on field observations)								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

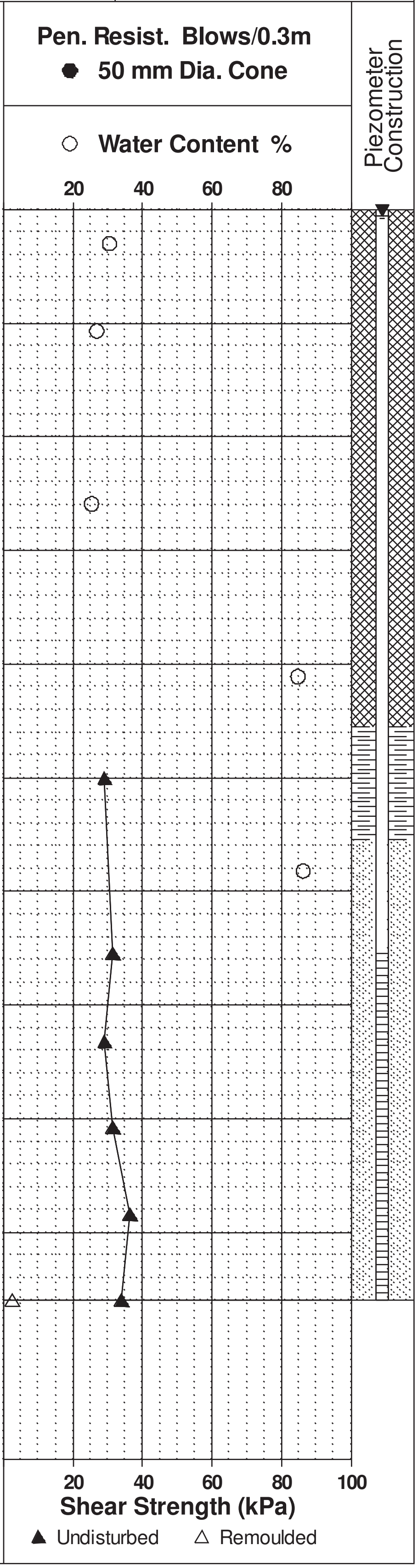
BORINGS BY CME 55 Power Auger

DATE June 13, 2014

FILE NO. PG2444

HOLE NO. BH 2-14

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.41
TOPSOIL	0.23	AU	1				
		SS	2	67	8	1	85.41
Loose to compact, brown to grey **SILTY SAND**						2	84.41
		SS	3	100	16		
						3	83.41
	3.66						
		SS	4			4	82.41
						5	81.41
		TW	5	100			
						6	80.41
Firm, grey **SILTY CLAY**							
						7	79.41
						8	78.41
						9	77.41
	9.60						
End of Borehole (GWL @ 0.06m-Sept. 2, 2014)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

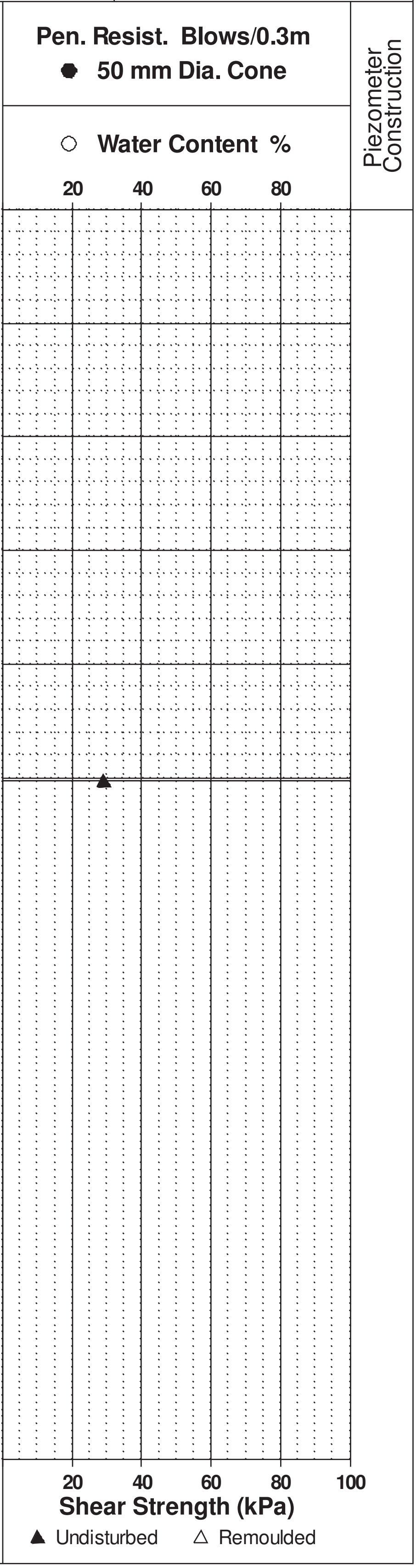
DATE June 13, 2014

FILE NO. PG2444

HOLE NO. BH 2A-14

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.23				
Loose to compact, brown to grey **SILTY SAND** 3.66				
Firm, grey **SILTY CLAY** 5.03	TW	1	100	
End of Borehole				

DEPTH (m)	ELEV. (m)
0	86.41
1	85.41
2	84.41
3	83.41
4	82.41
5	81.41



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

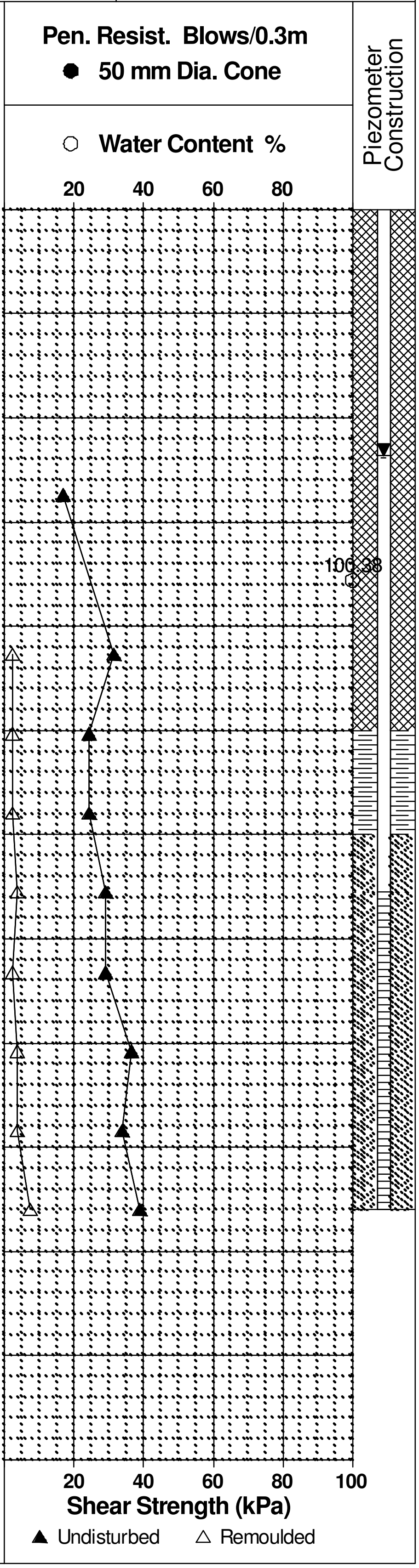
BORINGS BY CME 55 Power Auger

DATE 17 November 2011

FILE NO. PG2444

HOLE NO. BH22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.08
TOPSOIL 0.48		AU	1				
Compact, brown **SILTY SAND**		AU	2				
1.45		SS	3	71	10	1	85.08
Stiff, brown **SILTY CLAY,** trace sand 1.96		SS	4		3	2	84.08
Firm, grey **SILTY CLAY**		TW	5	100		3	83.08
						4	82.08
						5	81.08
						6	80.08
						7	79.08
						8	78.08
						9	77.08
9.60							
Dynamic Cone Penetration Test commenced @ 9.60m depth. Cone pushed to 34.7m depth.						10	76.08
						11	75.08
						12	74.08



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

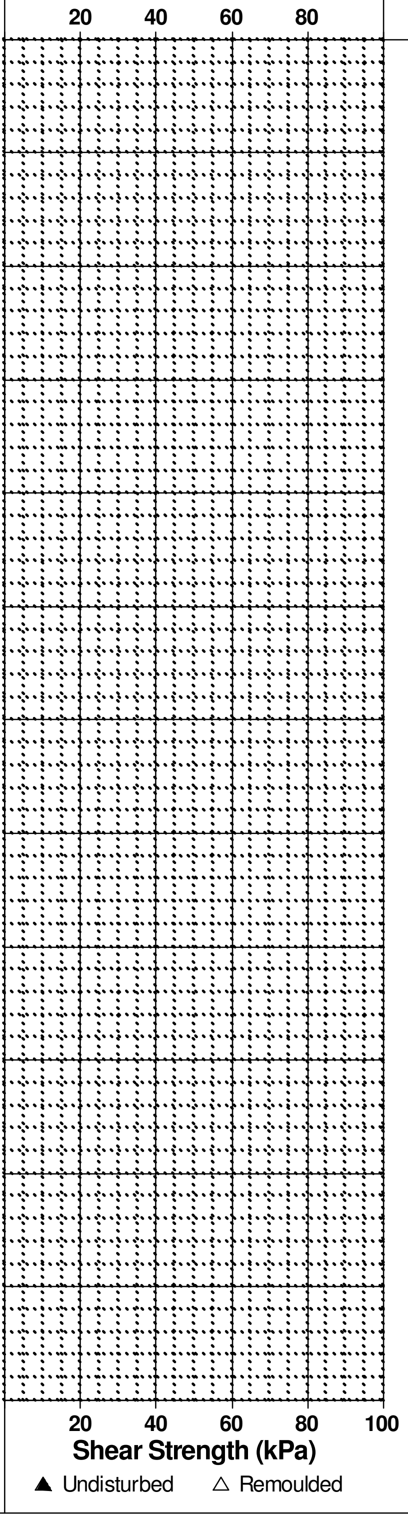
BORINGS BY CME 55 Power Auger

DATE 17 November 2011

FILE NO. PG2444

HOLE NO. BH22

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						12	74.08		
						13	73.08		
						14	72.08		
						15	71.08		
						16	70.08		
						17	69.08		
						18	68.08		
						19	67.08		
						20	66.08		
						21	65.08		
						22	64.08		
						23	63.08		
						24	62.08		



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

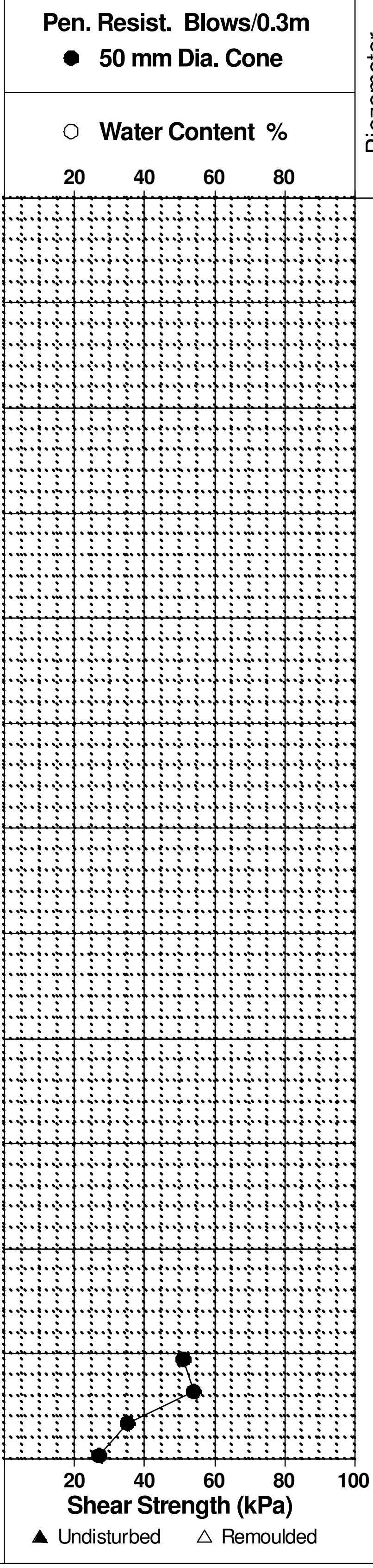
BORINGS BY CME 55 Power Auger

DATE 17 November 2011

FILE NO. PG2444

HOLE NO. BH22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
						24	62.08	
						25	61.08	
						26	60.08	
						27	59.08	
						28	58.08	
						29	57.08	
						30	56.08	
						31	55.08	
						32	54.08	
						33	53.08	
						34	52.08	
						35	51.08	
						36	50.08	



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

FILE NO. **PG2444**

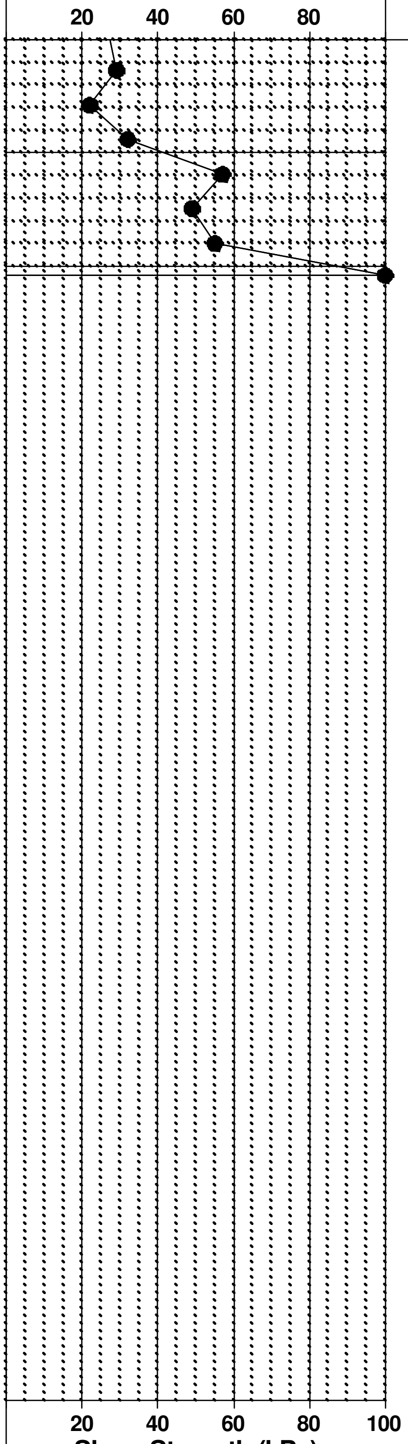
REMARKS

HOLE NO. **BH22**

BORINGS BY CME 55 Power Auger

DATE 17 November 2011

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									
						36	50.08		
						37	49.08		
38.08						38	48.08		
End of Borehole Practical DCPT refusal @ 38.08m depth (GWL @ 2.36m-Jan. 9/12)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community - Renaud Road
Ottawa, Ontario

DATUM Ground surface elevation at test pit locations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

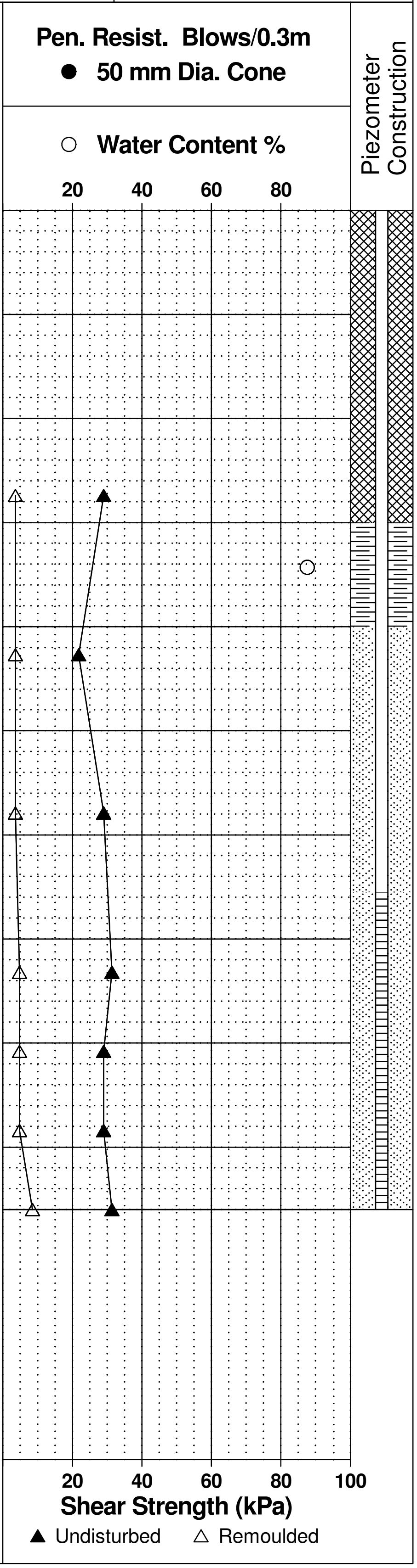
BORINGS BY CME 55 Power Auger

DATE 2010 July 26

FILE NO. PG1829

HOLE NO. BH 3-10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.11
Topsoil 0.15							
Loose, brown **SILTY SAND**		SS	1	100			
		SS	2	75	10	1	85.11
1.40							
Soft to firm, brown **SILTY CLAY** - grey by 2.3m depth		SS	3	50	2	2	84.11
						3	83.11
		TW	4				
						4	82.11
		TW	5			5	81.11
						6	80.11
		TW	6				
						7	79.11
						8	78.11
						9	77.11
9.60							
End of Borehole							



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community-Phase I, Renaud Road
Ottawa, Ontario

DATUM Ground surface elevation at test pit locations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY Backhoe

DATE 17 Aug 09

FILE NO. PG1829

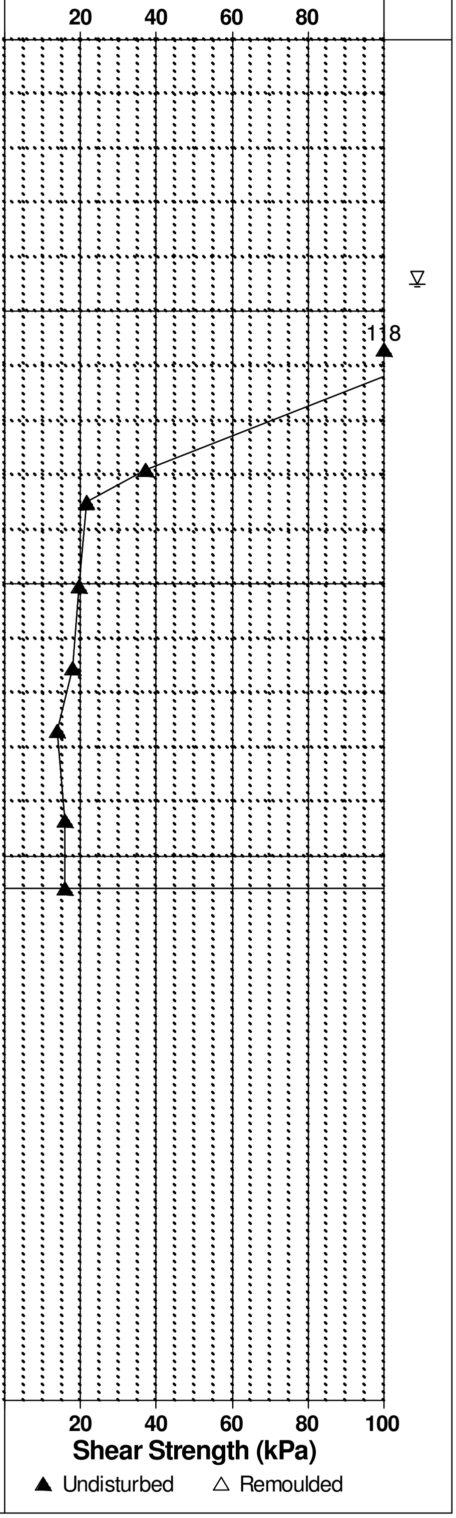
HOLE NO. TP 8-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.03
TOPSOIL — 0.18							
Brown **SILTY SAND** — 1.07						1	85.03
Very stiff to firm, red-brown **SILTY CLAY** - firm to soft and grey by 1.7m depth — 3.12						2	84.03
						3	83.03
End of Test Pit (GWL @ 0.9m depth based on field observations)							

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

○ Water Content %

Piezometer Construction



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community-Phase I, Renaud Road
Ottawa, Ontario

DATUM Ground surface elevation at test pit locations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

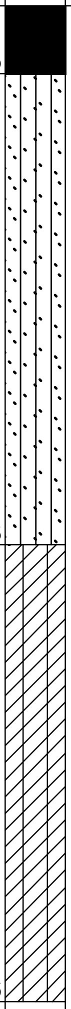
BORINGS BY Backhoe

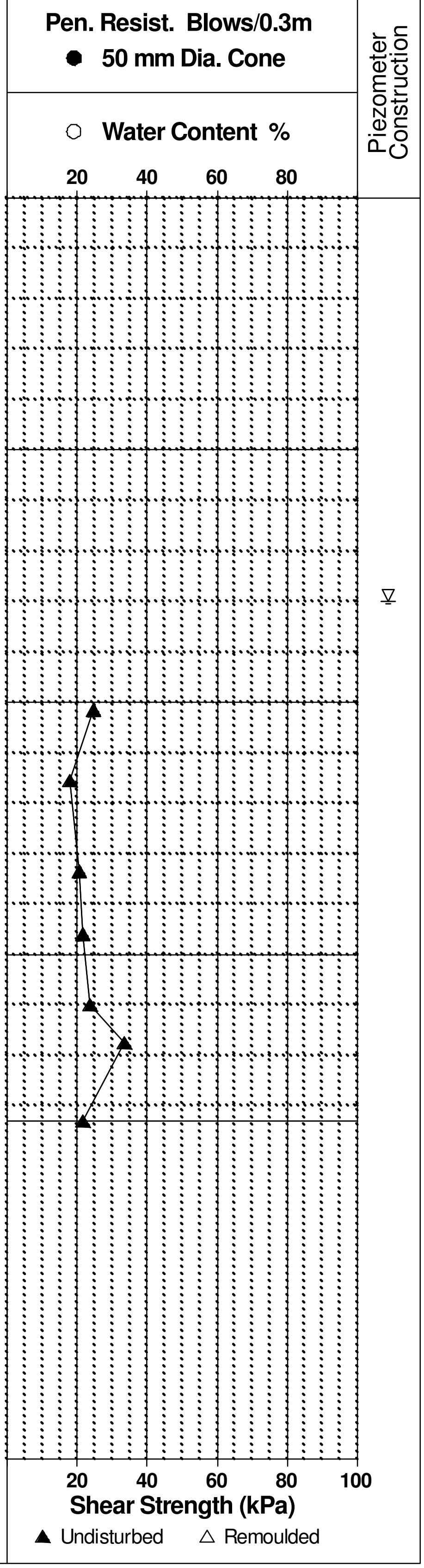
DATE 17 Aug 09

FILE NO. PG1829

HOLE NO. TP10-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.47
TOPSOIL							
0.25							
Compact, red-brown **SILTY SAND** - grey by 1.5m depth						1	85.47
1.98						2	84.47
Red-brown **SILTY CLAY** - soft and grey by 2.3m depth						3	83.47
3.66							
End of Test Pit (GWL @ 1.6m depth based on field observations)							





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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community-Phase I, Renaud Road
Ottawa, Ontario

DATUM Ground surface elevation at test pit locations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

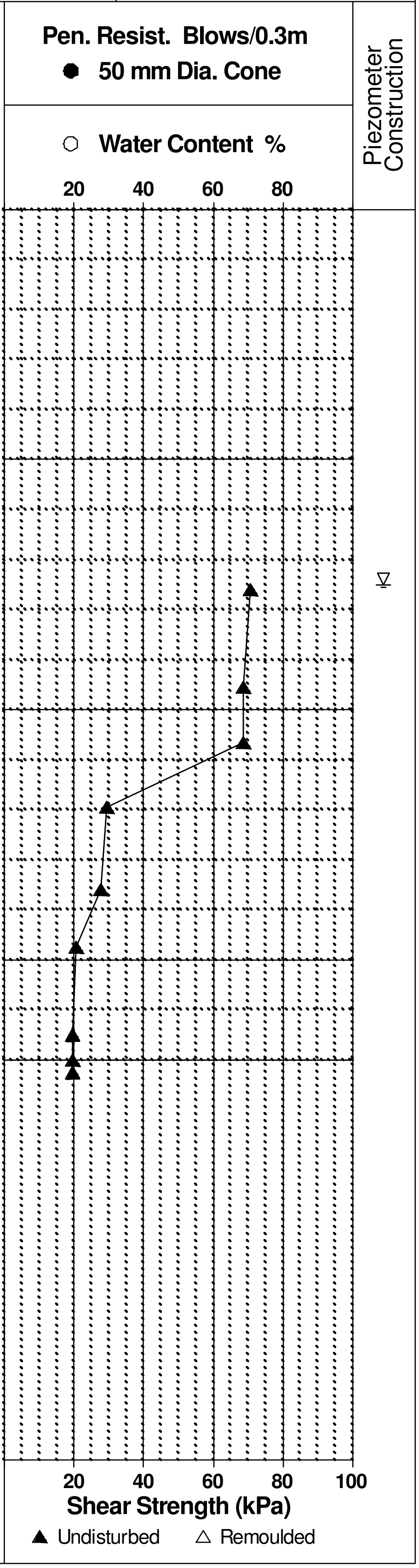
BORINGS BY Backhoe

DATE 17 Aug 09

FILE NO. PG1829

HOLE NO. TP11-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.11
TOPSOIL 0.28							
Loose, brown **SILTY SAND** 1.07						1	85.11
Stiff, grey-brown **SILTY CLAY,** trace sand - firm to soft and grey by 2.3m depth 3.40						2	84.11
						3	83.11
End of Test Pit (GWL @ 1.5m depth based on field observations)							



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community - Renaud Road
Ottawa, Ontario

DATUM

REMARKS

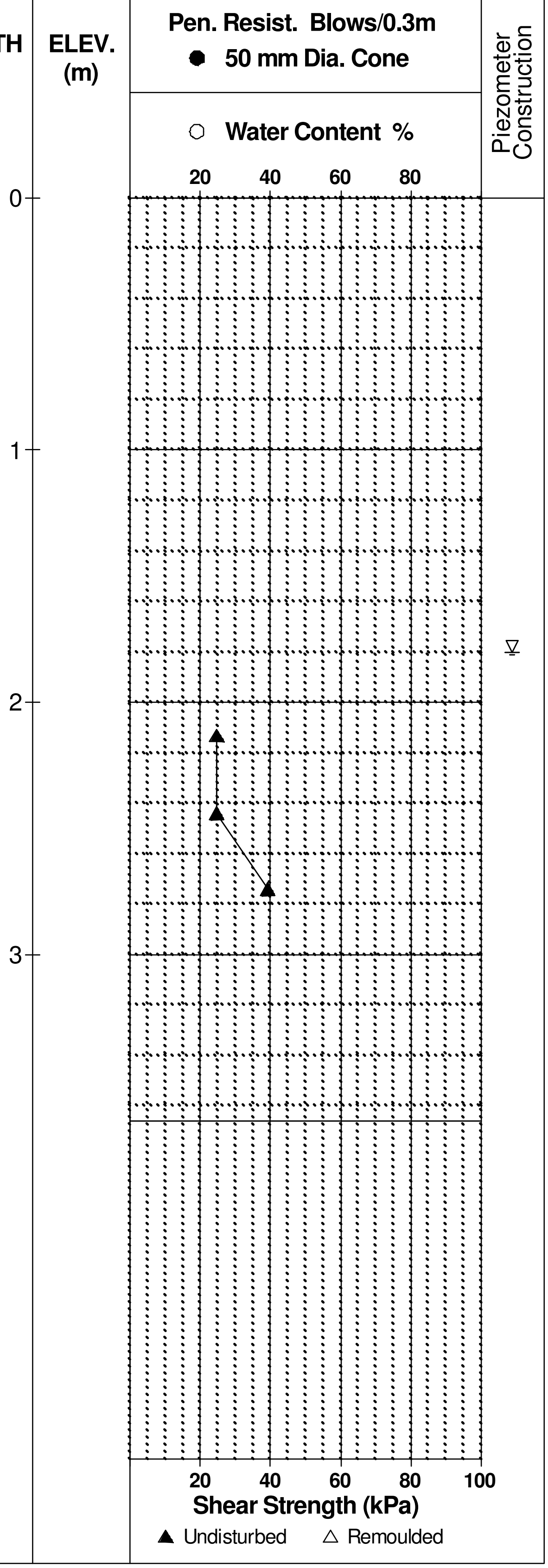
BORINGS BY Hydraulic Shovel

DATE 28 Oct 09

FILE NO. PG1829

HOLE NO. TP15-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25				
Red-brown **SILTY SAND** - grey by 0.7m depth	1.37	G G	1 2		
Soft to firm, brown **SILTY CLAY** - grey by 1.8m depth	3.66				
End of Test Pit (Open hole GWL @ 1.8m depth)					



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community - Renaud Road
Ottawa, Ontario

DATUM

REMARKS

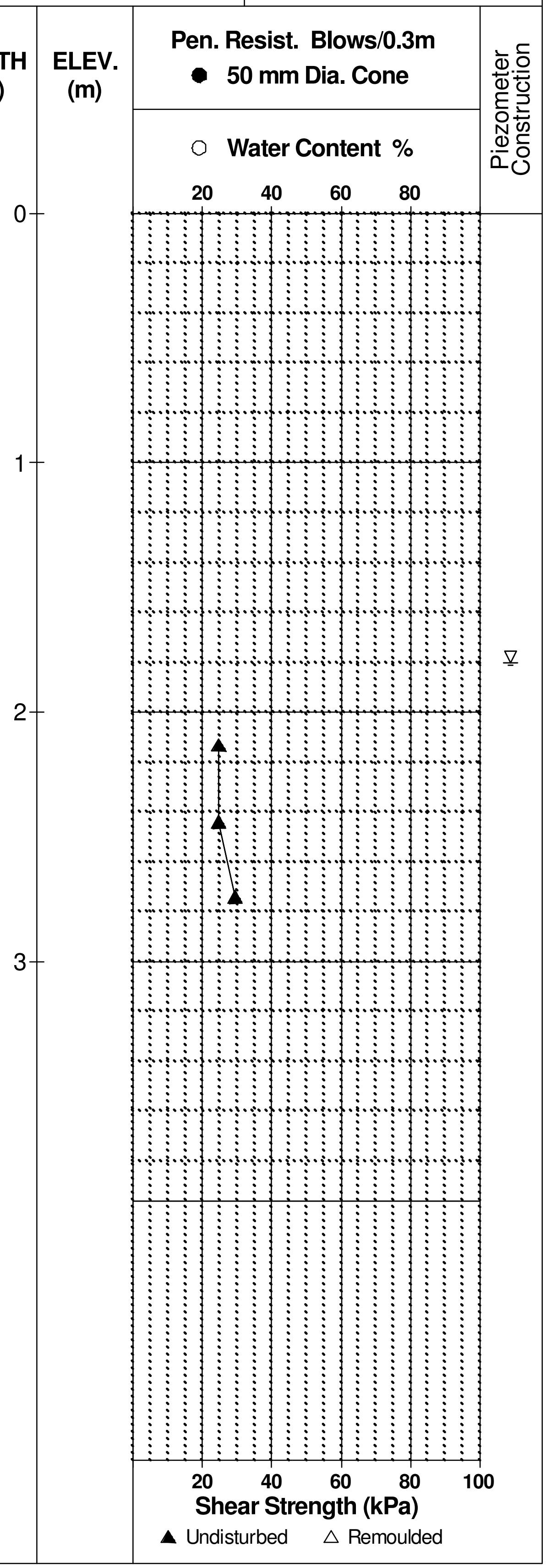
BORINGS BY Hydraulic Shovel

DATE 28 Oct 09

FILE NO. PG1829

HOLE NO. TP16-09

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE	0				
TOPSOIL	0.10				
Brown **SILTY SAND** - grey by 0.7m depth	1.65	G G	1 2		
Soft to firm, grey **SILTY CLAY**	3.96				
End of Test Pit (Open hole GWL @ 1.8m depth)					



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community - Renaud Road
Ottawa, Ontario

DATUM

REMARKS

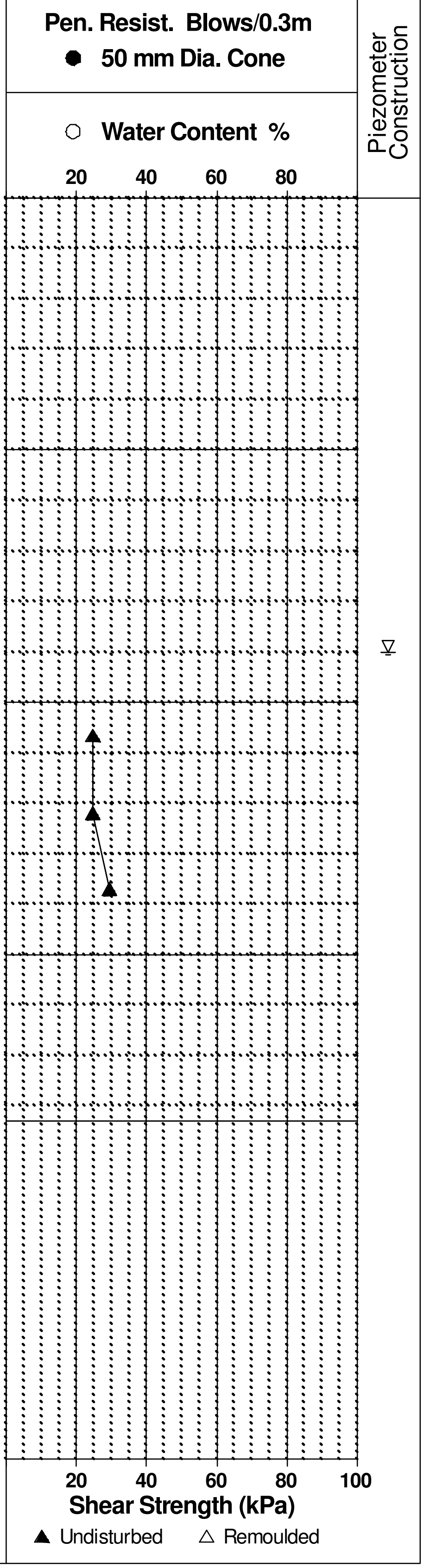
BORINGS BY Hydraulic Shovel

DATE 28 Oct 09

FILE NO. **PG1829**

HOLE NO. **TP17-09**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL 0.25							
Red-brown **SILTY SAND**		G	1				
- grey by 0.6m depth		G	2				
						1	
1.83							
Soft to firm, grey **SILTY CLAY**						2	
						3	
3.66							
End of Test Pit (Open hole GWL @ 1.8m depth)							



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
East Urban Community - Renaud Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY Hydraulic Shovel

DATE 28 Oct 09

FILE NO. PG1829

HOLE NO. TP19-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
TOPSOIL 0.25	■							
Brown **SILTY SAND** - grey by 0.5m depth		G G	1 2			1 2		
2.74								⊻
Soft to firm, grey **SILTY CLAY**	▨	G	3			3		
3.96								
End of Test Pit (Open hole GWL @ 2.7m depth)								

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

○ Water Content %

20 40 60 80

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

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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

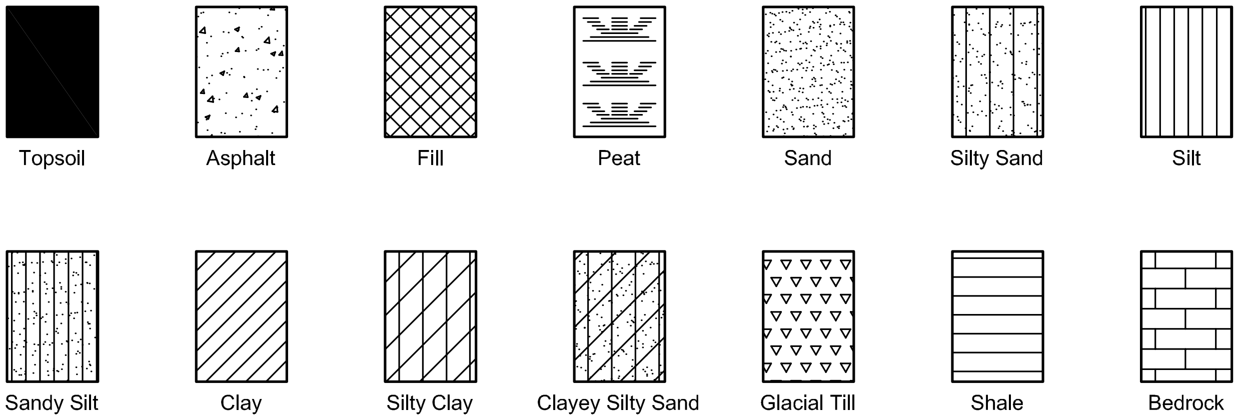
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

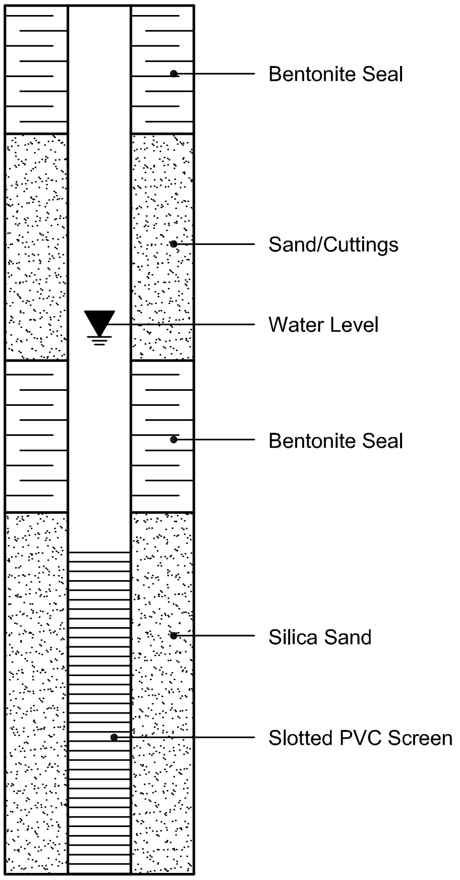
SYMBOLS AND TERMS (continued)

STRATA PLOT

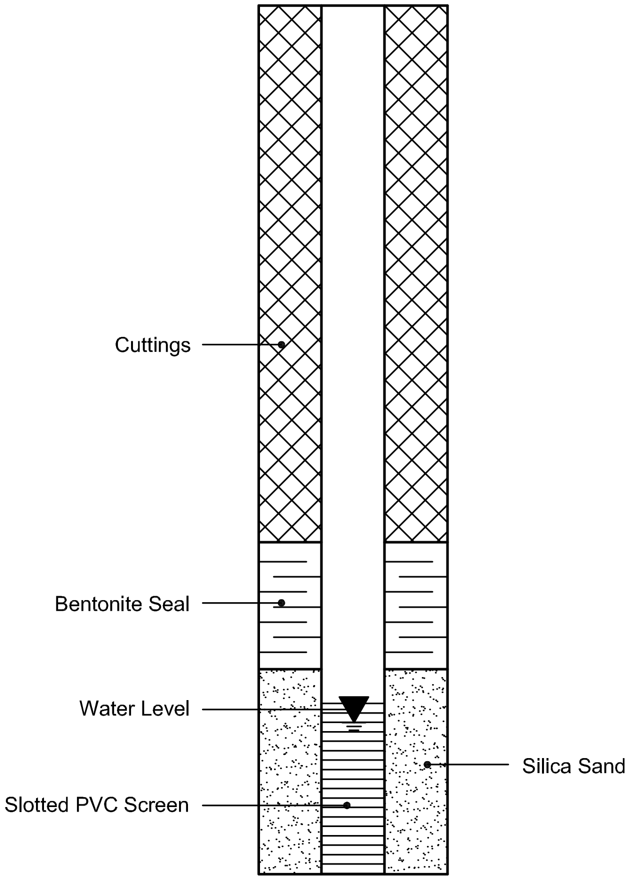


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 07-1121-0129

RECORD OF BOREHOLE: 08-4

SHEET 1 OF 1

LOCATION: See Site Plan

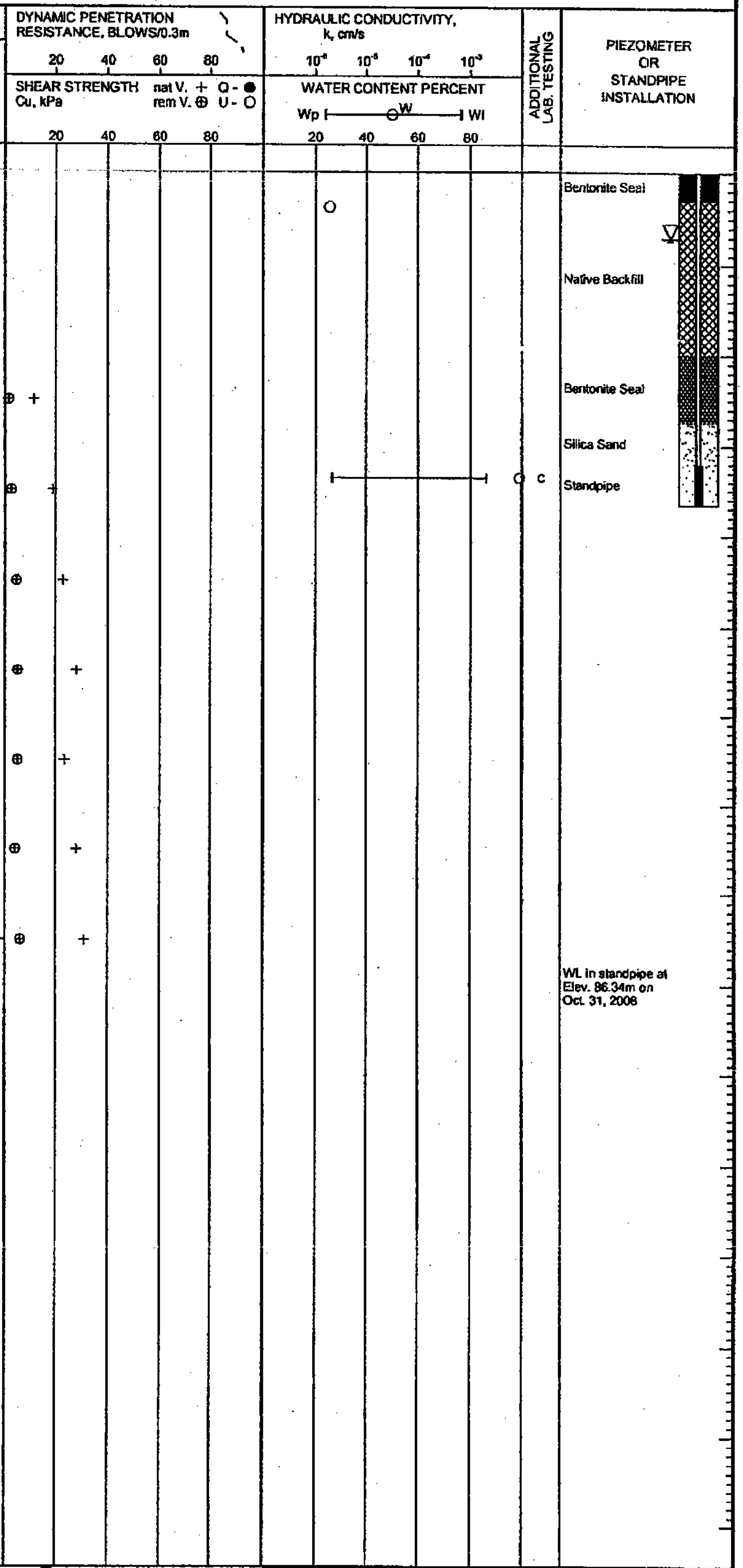
BORING DATE: October 27, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m
		GROUND SURFACE		86.30			
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00			
		Loose brown stratified SILTY SAND with occasional fine sand layer		86.00 0.30	1	50 DO	7
1		Loose grey SILTY SAND with occasional silty clay layer		85.08 1.22	2	50 DO	5
2		Stiff grey SILTY CLAY		84.32 1.98			
3					3	73 TP	PH
8		End of Borehole		77.80 8.50			



WL in standpipe at Elev. 86.34m on Oct. 31, 2008

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: RAG

PROJECT: 07-1121-0129

RECORD OF BOREHOLE: 07-5

SHEET 1 OF 3

LOCATION: See Site Plan

BORING DATE: July 24, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES: 0–15

BORING METHOD: Power Auger 200mm Diam. (Hollow Stem); 60° Cone Penetration

SOIL PROFILE

DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
GROUND SURFACE		86.91
Brown and grey brown silty sand, trace organic matter and gravel (FILL)	▩	0.00
TOPSOIL		86.23 0.68
Loose grey brown very fine SAND, occasional silt layers		0.81
Compact red orange to grey brown fine SAND, trace silt		85.69 1.22
Grey layered SILTY SAND, CLAYEY SILT and SILTY CLAY		84.01 2.90
Firm grey SILTY CLAY	▨	83.56 3.35
Probably grey Silty Clay	▨	75.33 11.58

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	A.S.	
2	50 DO	7
3	50 DO	14
4	50 DO	13
5	50 DO	11
6	73 TP	PH
7	50 DO	PM

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

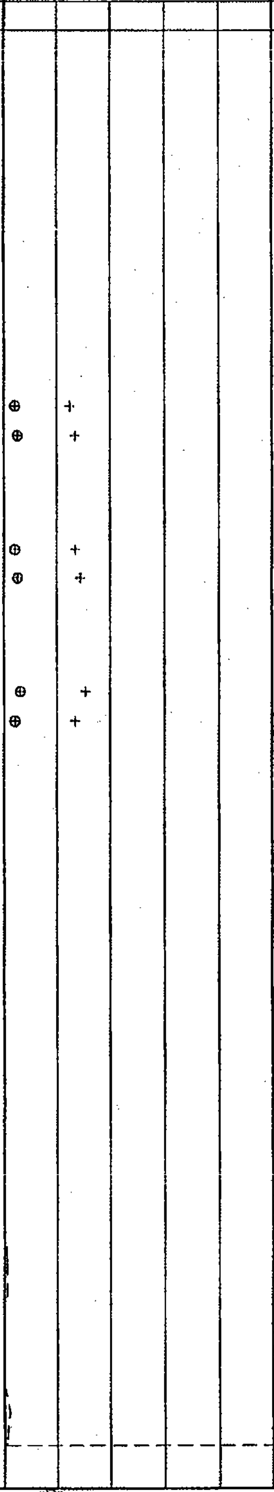
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Water level in open hole at 1.83m depth below ground surface upon completion of drilling

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: BDL

PROJECT: 07-1121-0129

RECORD OF BOREHOLE: 07-5

SHEET 3 OF 3

LOCATION: See Site Plan

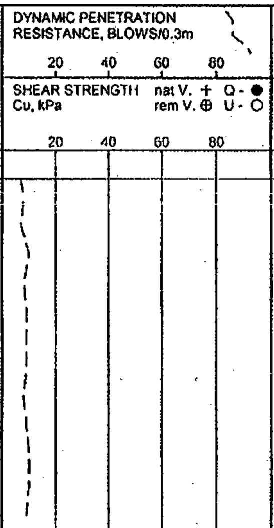
BORING DATE: July 24, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
30	60° Cone Penetration		▨	56.73					
		Probably loose Glacial Till	▨	30.18					
31									
32									
33				53.37					
		End of Borehole		33.54					
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									



HYDRAULIC CONDUCTIVITY, k, cm/s

10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT

Wp ⊢——O^W——⊣ Wl

20 40 60 80

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: BAG

MIS-BHS 001 07-1121-0129.GPJ GAL-MISS.GDT 1/19/09 J.M.

PROJECT: 07-1121-0129

RECORD OF BOREHOLE: 07-7

SHEET 1 OF 1

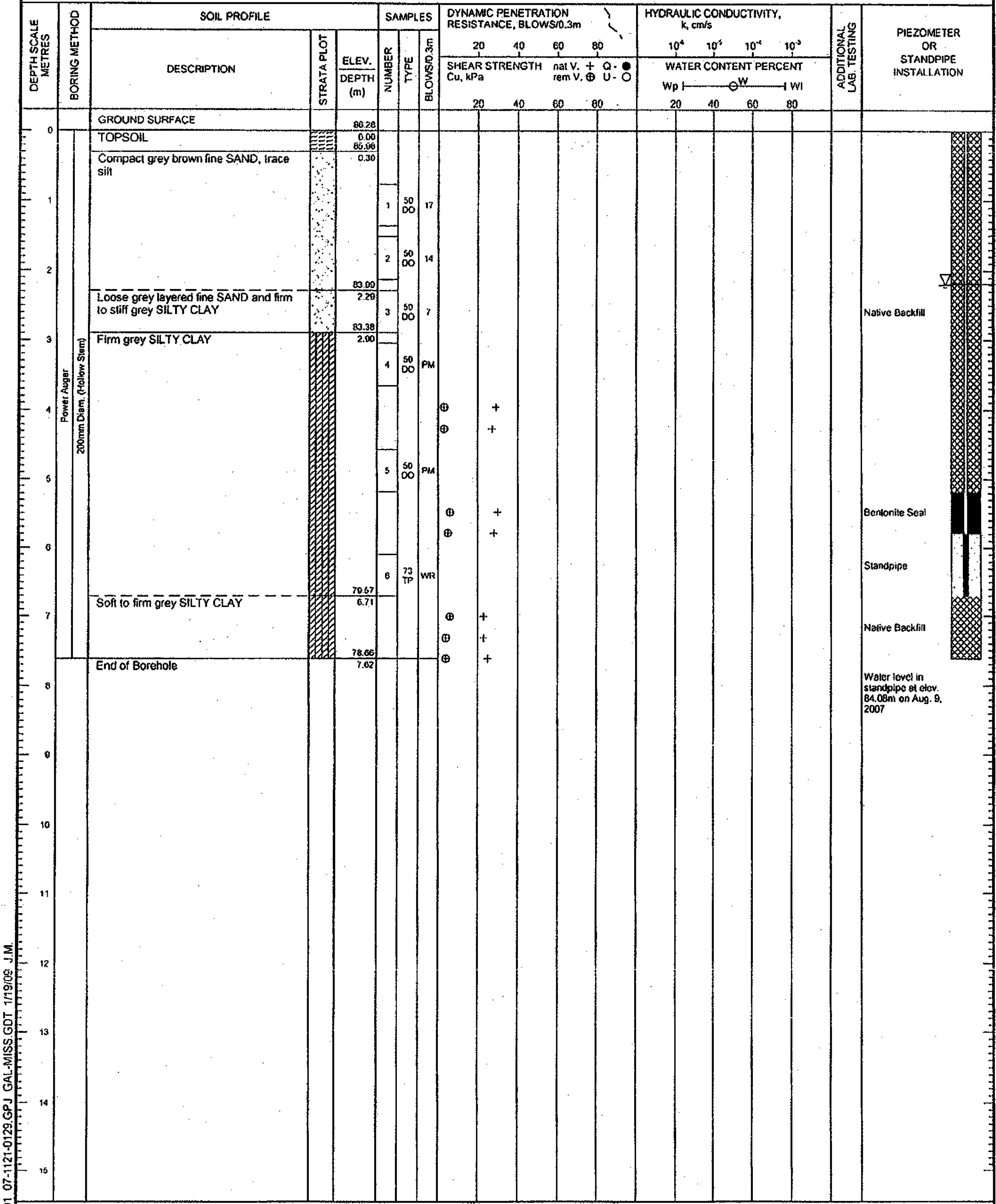
LOCATION: See Site Plan

BORING DATE: July 23, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



BORING METHOD: Power Auger, 200mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
GROUND SURFACE	86.28			
TOPSOIL	0.00			
Compact grey brown fine SAND, trace silt	85.98 0.30	1	50 DO	17
		2	50 DO	14
Loose grey layered fine SAND and firm to stiff grey SILTY CLAY	83.99 2.29	3	50 DO	7
Firm grey SILTY CLAY	83.38 2.90	4	50 DO	PM
		5	50 DO	PM
		6	73 TP	WR
Soft to firm grey SILTY CLAY	79.57 6.71			
End of Borehole	78.66 7.62			

PIEZOMETER OR STANDPIPE INSTALLATION:

Native Backfill

Bentonite Seal

Standpipe

Native Backfill

Water level in standpipe at elev. 84.08m on Aug. 9, 2007

DEPTH SCALE

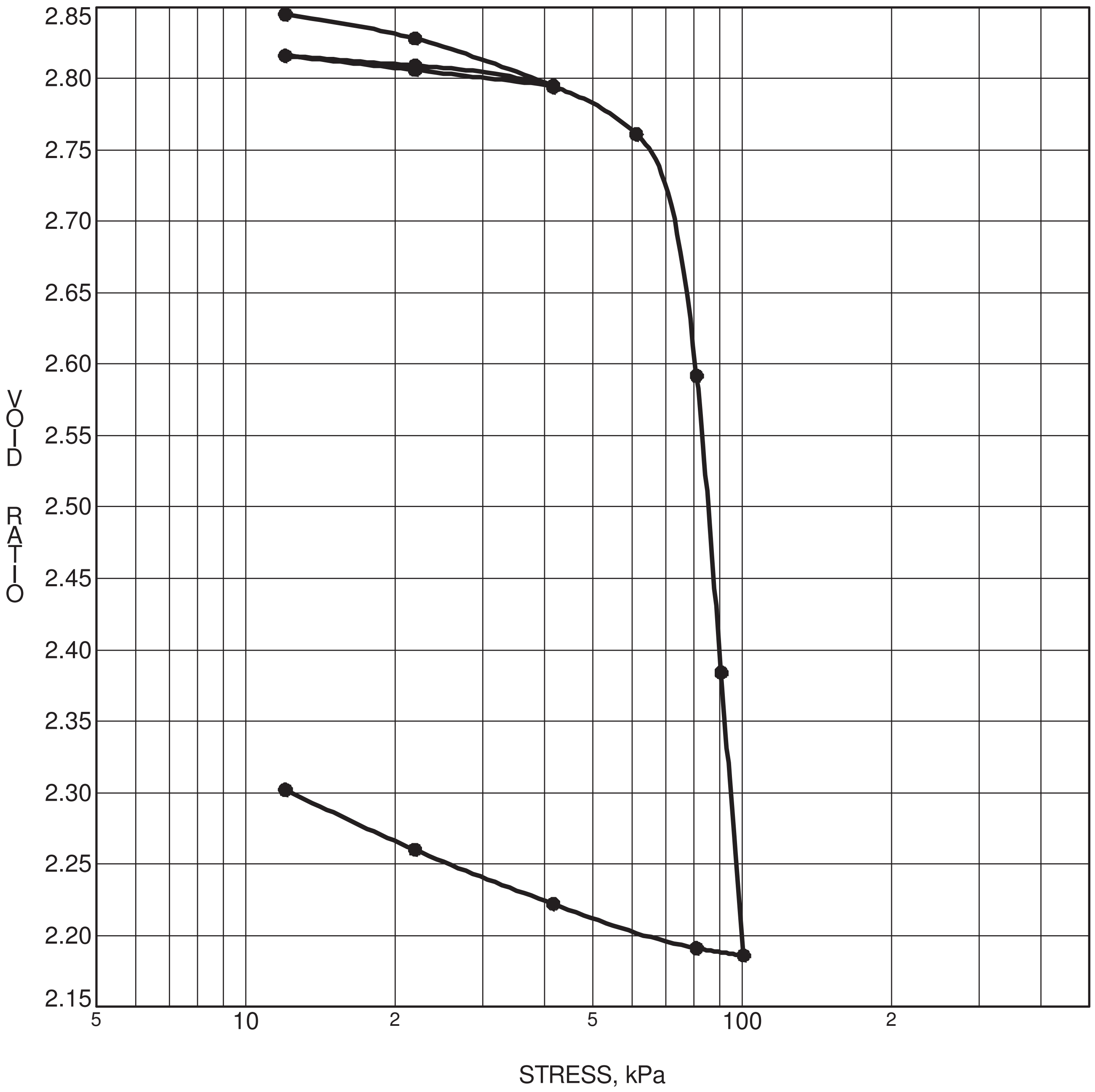
1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: BDL

MIS-BHS 001 07-1121-0129.GPJ GAL-MISS.GDT 1/19/09 J.M.



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1-14**	p'_o	**35.1** kPa	Ccr	**0.040**
Sample No.	**TW 3**	p'_c	**75** kPa	Cc	**4.332**
Sample Depth	**2.74** m	OC Ratio	**2.1**	Wo	**83.0** %
Sample Elev.	**85.00** m	Void Ratio	**2.873**	Unit Wt.	**14.2** kN/m^3

CLIENT **Ashcroft Homes**

PROJECT **Geotechnical Investigation - Prop. Residential Dev.-Eastboro Phase 2-Navan Road**

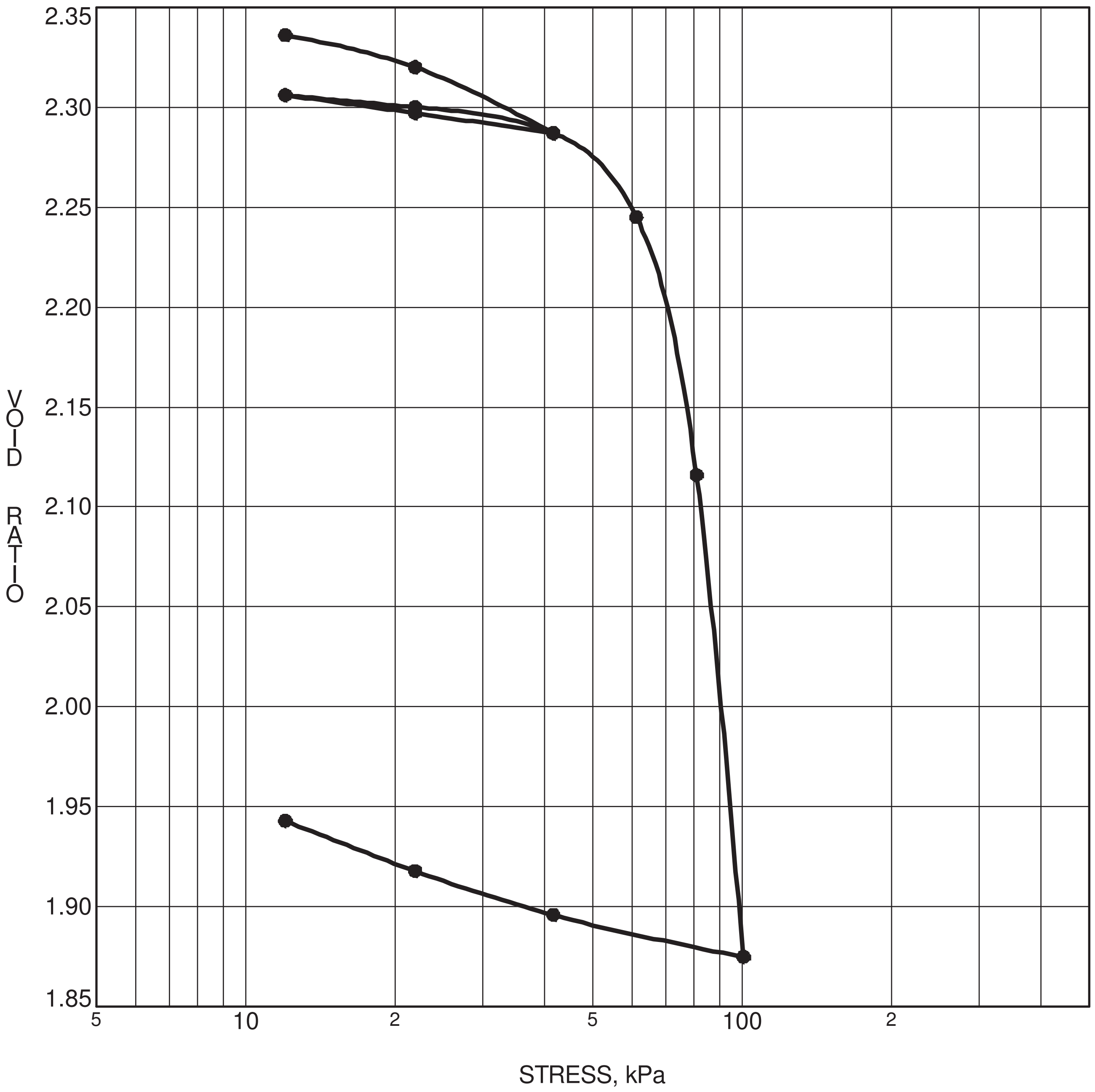
FILE NO. **PG2444**

DATE **08/07/2014**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2-14**	p'_o	**58.1** kPa	Ccr	**0.035**
Sample No.	**TW 5**	p'_c	**73** kPa	Cc	**2.696**
Sample Depth	**5.82** m	OC Ratio	**1.3**	Wo	**86.2** %
Sample Elev.	**80.59** m	Void Ratio	**2.371**	Unit Wt.	**14.9** kN/m^3

CLIENT **Ashcroft Homes**

FILE NO. **PG2444**

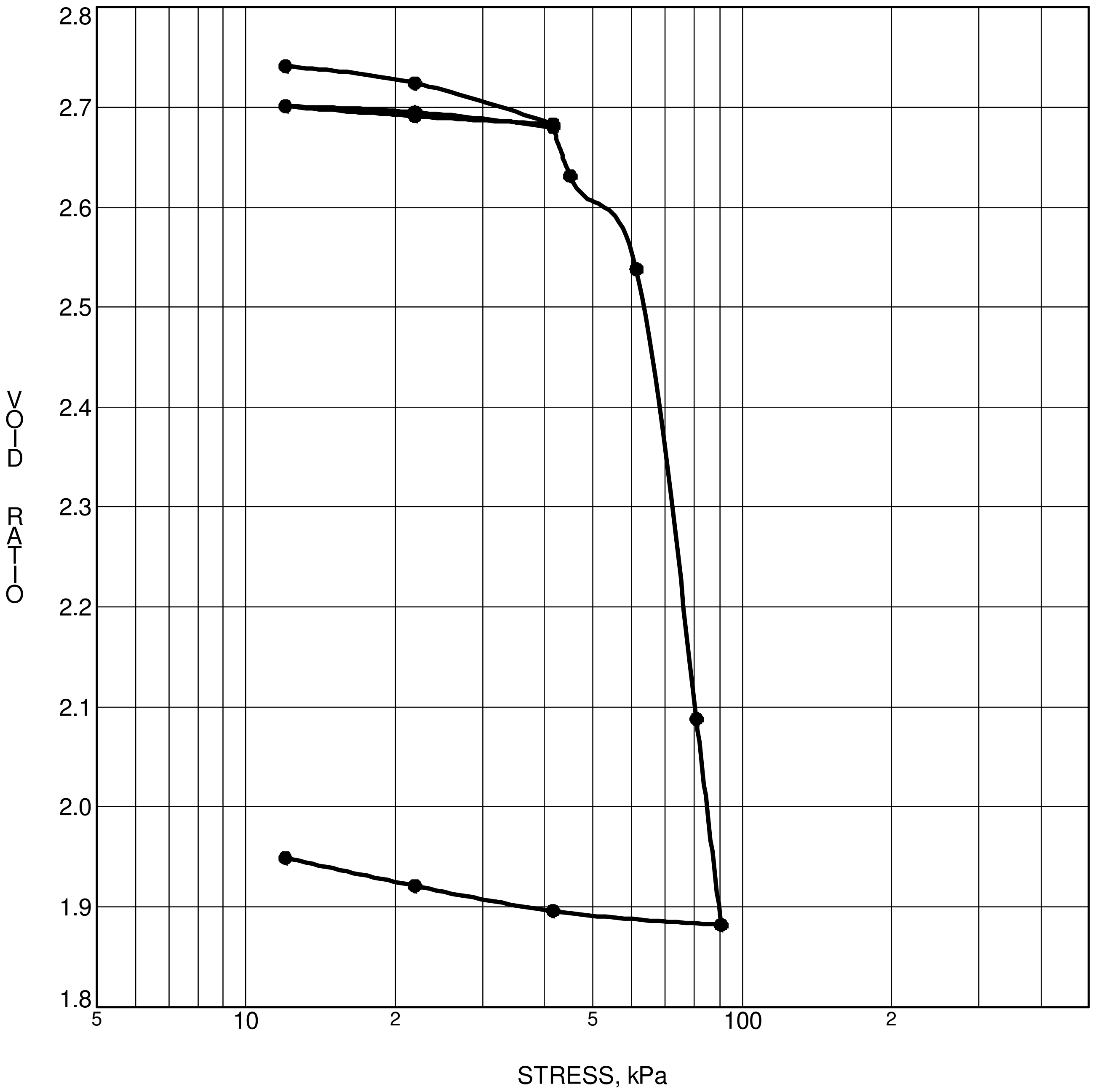
PROJECT **Geotechnical Investigation - Prop. Residential Dev.-Eastboro Phase 2-Navan Road**

DATE **23/06/2014**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY							
Borehole No.	**BH22**	p'$_o$	**38**	kPa	Ccr	**0.038**	
Sample No.	**TW 5**	p'$_c$	**60**	kPa	Cc	**4.296**	
Sample Depth	**3.56** m	OC Ratio	**1.6**		Wo	**100.4**	%
Sample Elev.	**82.52** m	Void Ratio	**2.76**		Unit Wt.	**14.8**	kN/m^3

CLIENT **Ashcroft Homes**

FILE NO. **PG2444**

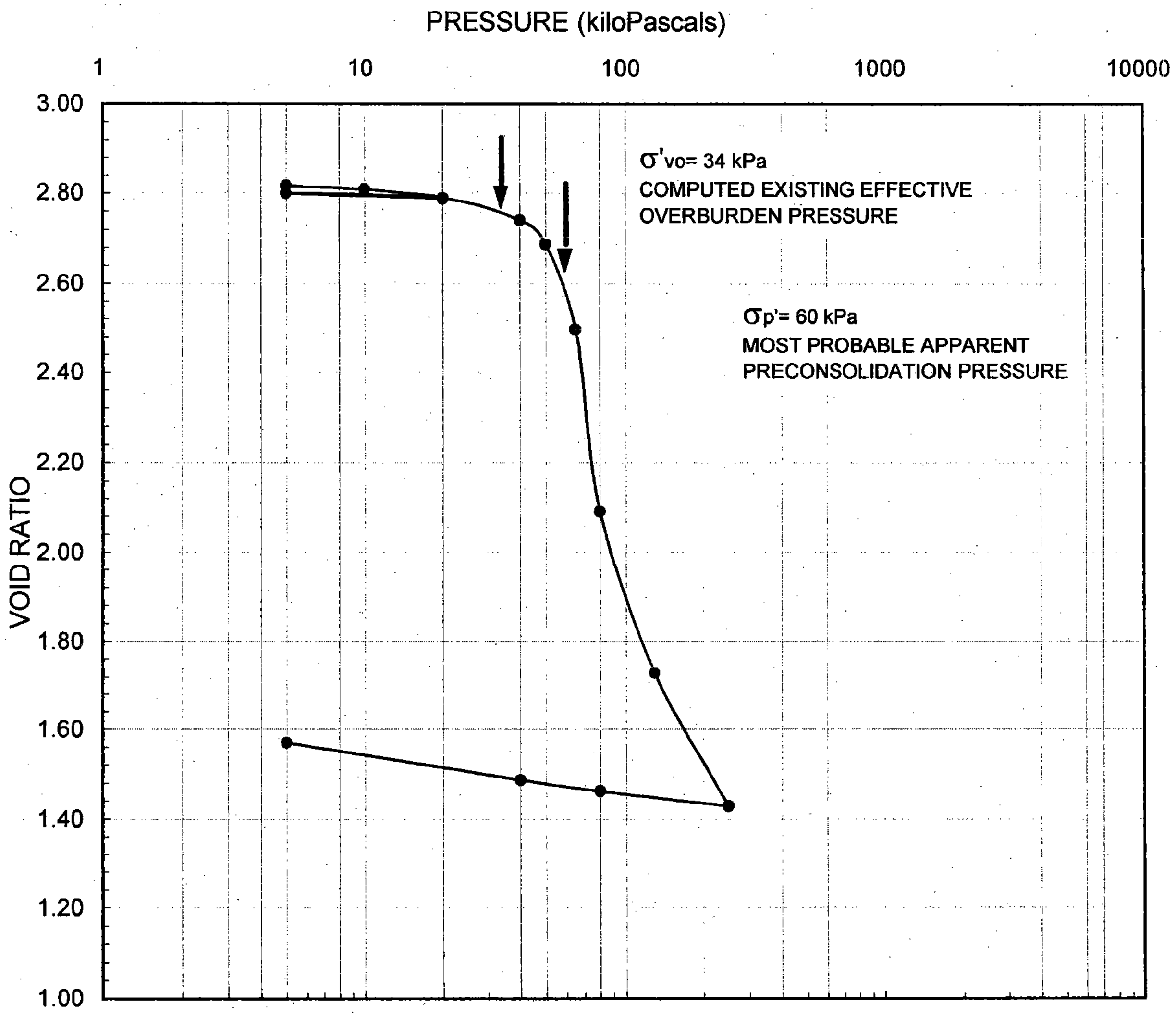
PROJECT **Geotechnical Investigation - Prop. Residential**
Dev.-Eastboro Phase 2-Navan Road

DATE **12/12/2011**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



LEGEND

Borehole: 08-4 w_i = 101.0% S_o = 99%
Sample: 3 w_f = 58.1% C_c = 4.50
Depth (m): 3.05-3.66 w_l = 86.0% C_r = 0.048
γ_m = 14.22 kN/m3 w_p = 26.3% e_o = 2.83

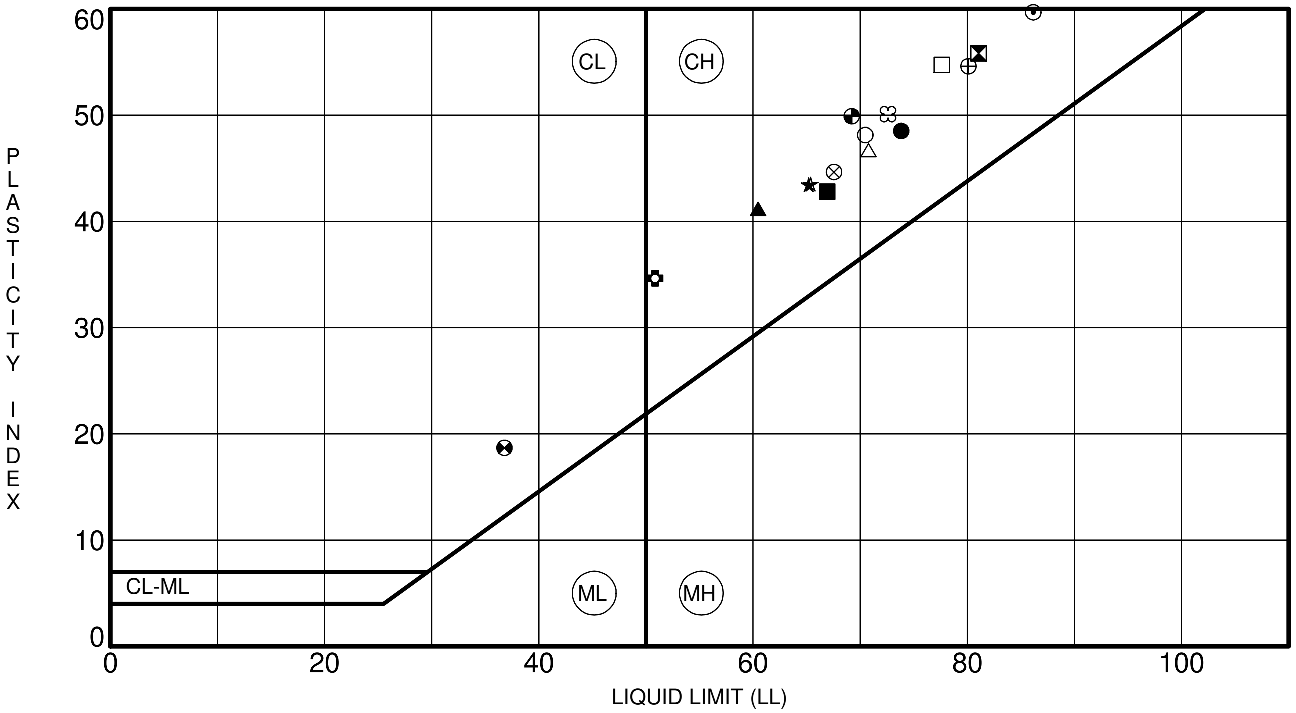
Golder Associates

SCALE AS SHOWN
DATE 01/22/09
DESIGN NA
CADD NA
CHECK
REVIEW

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 0711210129 REV. 0
FIGURE 7

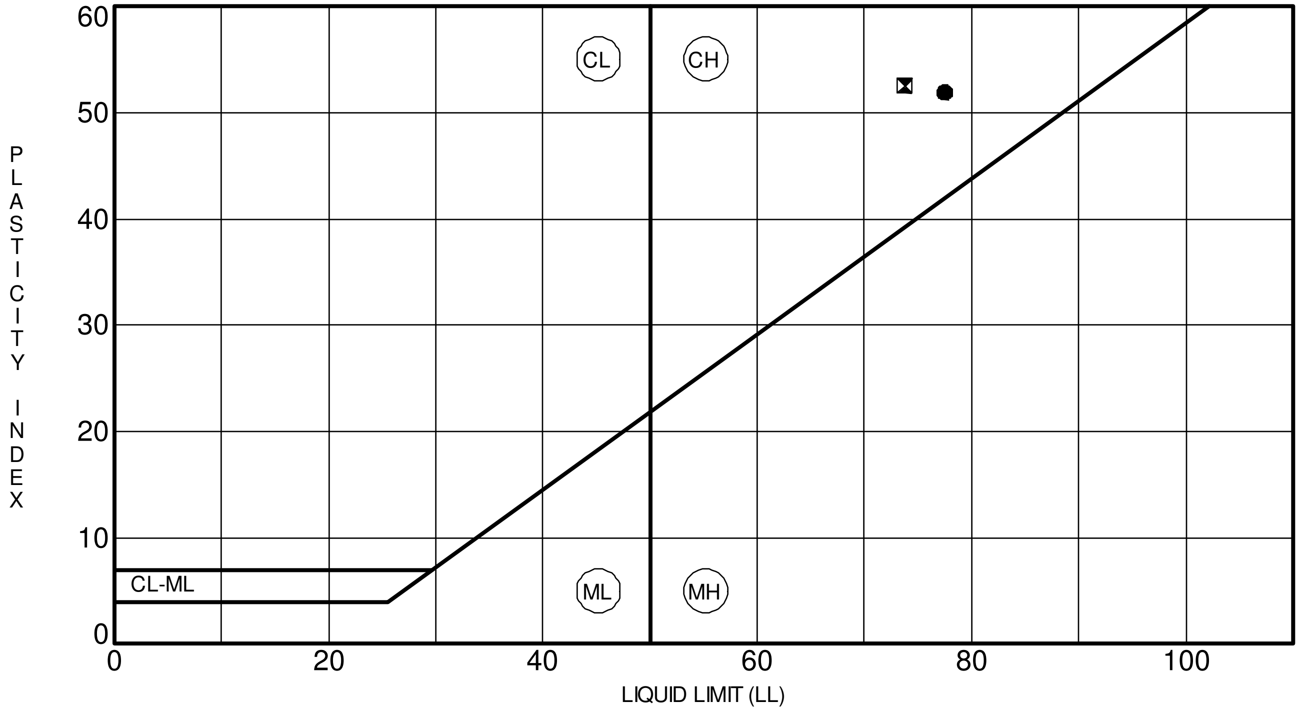


Specimen Identification			LL	PL	PI	Fines	Classification
●	BH 1-22	SS4	74	25	49		CH - Inorganic clays of high plasticity
⧗	BH 2-22	SS3	81	25	56		CH - Inorganic clays of high plasticity
▲	BH 3-22	SS2	60	19	41		CH - Inorganic clays of high plasticity
★	BH 4-22	SS4	65	22	43		CH - Inorganic clays of high plasticity
⊙	BH 5-22	SS5	86	26	60		CH - Inorganic clays of high plasticity
✿	BH 6-22	SS3	51	16	35		CH - Inorganic clays of high plasticity
○	BH 7-22	SS5	70	22	48		CH - Inorganic clays of high plasticity
△	BH 8-22	SS3	71	24	47		CH - Inorganic clays of high plasticity
⊗	BH 9-22	SS3	68	23	45		CH - Inorganic clays of high plasticity
⊕	BH10-22	SS3	80	25	55		CH - Inorganic clays of high plasticity
□	BH11-22	SS3	78	23	55		CH - Inorganic clays of high plasticity
⊗	BH12-22	SS4	37	18	19		CL - Inorganic clays of low plasticity
◕	BH13-22	SS3	69	19	50		CH - Inorganic clays of high plasticity
☆	BH14-22	SS7	65	22	44		CH - Inorganic clays of high plasticity
✣	BH15-22	SS4	73	22	50		CH - Inorganic clays of high plasticity
■	BH16-22	SS2	67	24	43		CH - Inorganic clays of high plasticity

CLIENT **Ashcroft Homes**
FILE NO. **PG2444**
PROJECT **Supplemental Geotechnical Investigation - Prop. Residential Development - Eastboro Phase 2**
DATE **25 May 22**

patersongroup **Consulting Engineers**
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 1-14 TW 3**	**77**	**26**	**52**		**CH - High plasticity inorganic clay**
⧗	**BH 4-14 TW 2**	**74**	**21**	**53**		**CH - High plasticity inorganic clay**

CLIENT **Ashcroft Homes**

PROJECT **Geotechnical Investigation - Prop. Residential**
Dev.-Eastboro Phase 2-Navan Road

FILE NO. **PG2444**

DATE **13 Jun 14**

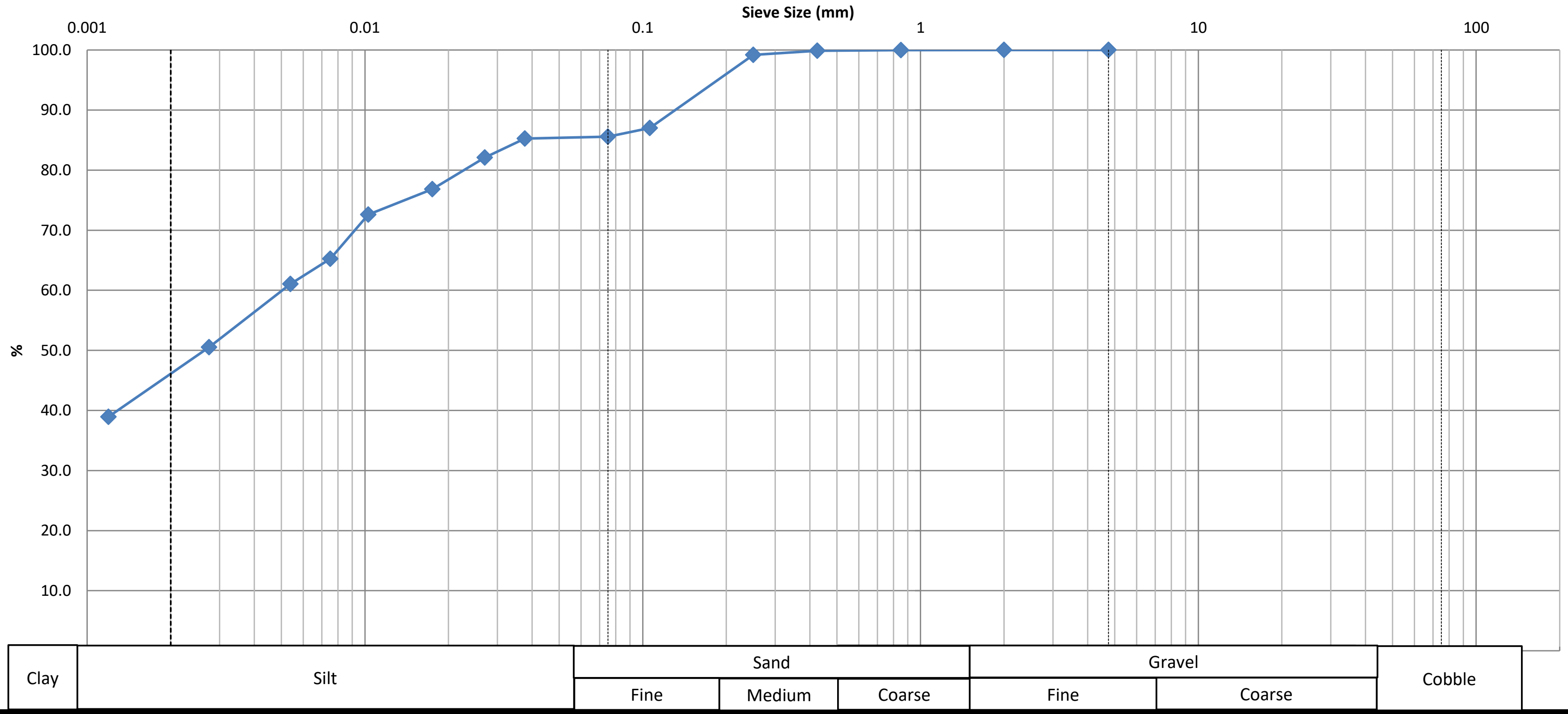
patersongroup **Consulting Engineers**
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Ashcroft Homes	DEPTH:	7'-6" to 9'-6"	FILE NO:	PG2444
CONTRACT NO.:		BH OR TP No.:	BH13-22-PH2 SS4	LAB NO:	34115
PROJECT:	Eastboro - Phase 2			DATE RECEIVED:	30-May-22
				DATE TESTED:	31-May-22
DATE SAMPLED:	May 19-25			DATE REPORTED:	10-Jun-22
SAMPLED BY:	N.S			TESTED BY:	DK/CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					46.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					0.0	14.4		39.1		46.5

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

patersongroup
consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ashcroft Homes	DEPTH:	7'-6" to 9'-6"	FILE NO.:	PG2444
PROJECT:	Eastboro - Phase 2	BH OR TP No.:	BH13-22-PH2 SS4	DATE SAMPLEI	May 19-25
LAB No. :	34115	TESTED BY:	DK/CS	DATE RECEIVE	30-May-22
SAMPLED BY:	N.S	DATE REPT'D:	10-Jun-22	DATE TESTED:	31-May-22

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
114.6	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	33.38	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	7.29	AIR DRY	127.90	77.90
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	102.00	52.00
		CORRECTED	0.668	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)		WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5				
19				
13.2				
9.5				
4.75		**0.00**	0.0	**100.0**
2.0		**0.00**	0.0	**100.0**
Pan		**102.00**		
0.850		**0.01**	0.0	**100.0**
0.425		**0.06**	0.1	**99.9**
0.250		**0.41**	0.8	**99.2**
0.106		**6.49**	13.0	**87.0**
0.075		**7.22**	14.4	**85.6**
Pan		**7.29**		
SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:30	46.5	6.0	23.0	0.0376	85.2	**85.2**
2	8:31	45.0	6.0	23.0	0.0270	82.1	**82.1**
5	8:34	42.5	6.0	23.0	0.0175	76.8	**76.8**
15	8:44	40.5	6.0	23.0	0.0103	72.6	**72.6**
30	8:59	37.0	6.0	23.0	0.0075	65.2	**65.2**
60	9:29	35.0	6.0	23.0	0.0054	61.0	**61.0**
250	12:39	30.0	6.0	23.0	0.0028	50.5	**50.5**
1440	8:29	24.5	6.0	23.0	0.0012	38.9	**38.9**

Moisture = 46.6%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Report Date: 09-Sep-2014
Order Date:3-Sep-2014

Client: **Paterson Group Consulting Engineers**
Client PO: 16486
Project Description: PG2444

	Client ID:	BH3-14 SS6	BH5-14 SS4	-	-
	Sample Date:	16-Jun-14	16-Jun-14	-	-
	Sample ID:	1436088-01	1436088-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	62.0	78.5	-	-
General Inorganics					
pH	0.05 pH Units	7.92 [1]	6.63 [1]	-	-
Resistivity	0.10 Ohm.m	37.1	386	-	-
Anions					
Chloride	5 ug/g dry	10 [1]	<5 [1]	-	-
Sulphate	5 ug/g dry	134 [1]	<5 [1]	-	-

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OTTAWA-EAST
300-2319 St. Laurent Blvd.
Ottawa, ON K1G 4J8

OTTAWA-WEST
104-195 Stafford Rd. W.
Nepean, ON K2H 9C1

MISSISSAUGA
6645 Kitimat Rd. Unit #27
Mississauga, ON L5N 6J3

SARNIA
218-704 Mara St.
Point Edward, ON N7V 1X4

NIAGARA
360 York Rd. Unit 16B
Niagara-on-the-Lake, ON L0S 1J0

KINGSTON
1058 Gardiners Rd.
Kingston, ON K7P 1R7

Page 3 of 7

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 11127

Project Description: PG2444

Report Date: 01-Feb-2012

Order Date:30-Jan-2012

Client ID:		BH3-SS4	BH13-SS4	BH25-SS3	-
Sample Date:		13-Sep-11	16-Sep-11	21-Nov-11	-
Sample ID:		1205032-01	1205032-02	1205032-03	-
	MDL/Units	Soil	Soil	Soil	-
Physical Characteristics					
% Solids	0.1 % by Wt.	62.0	80.1	62.0	-
General Inorganics					
pH	0.05 pH Units	7.54	7.54	7.59	-
Resistivity	0.10 Ohm.m	39.0	34.7	23.0	-
Anions					
Chloride	5 ug/g dry	20 [1]	43 [1]	30 [1]	-
Sulphate	5 ug/g dry	198 [1]	134 [1]	1140 [1]	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2444-5 - TEST HOLE LOCATION PLAN

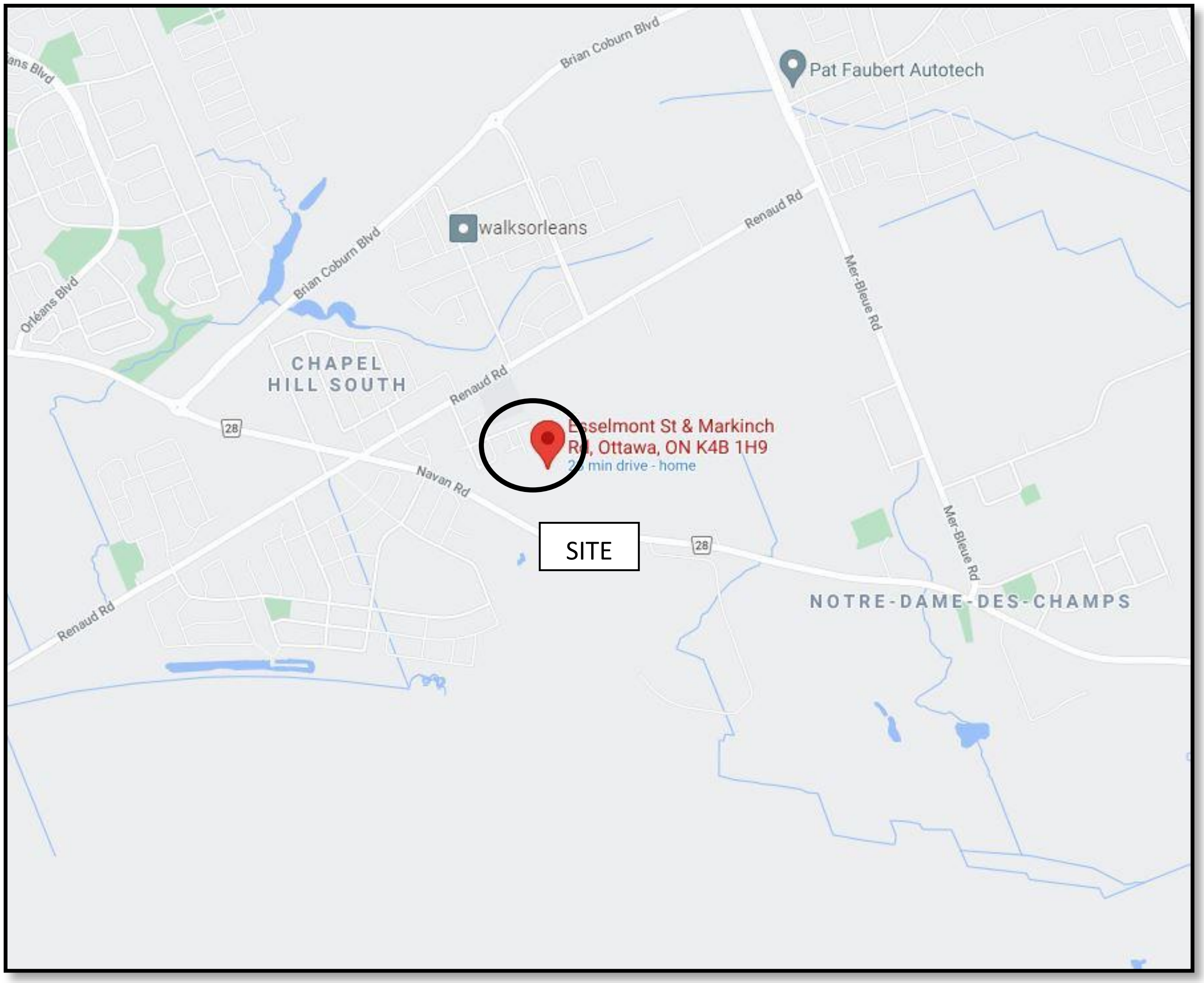
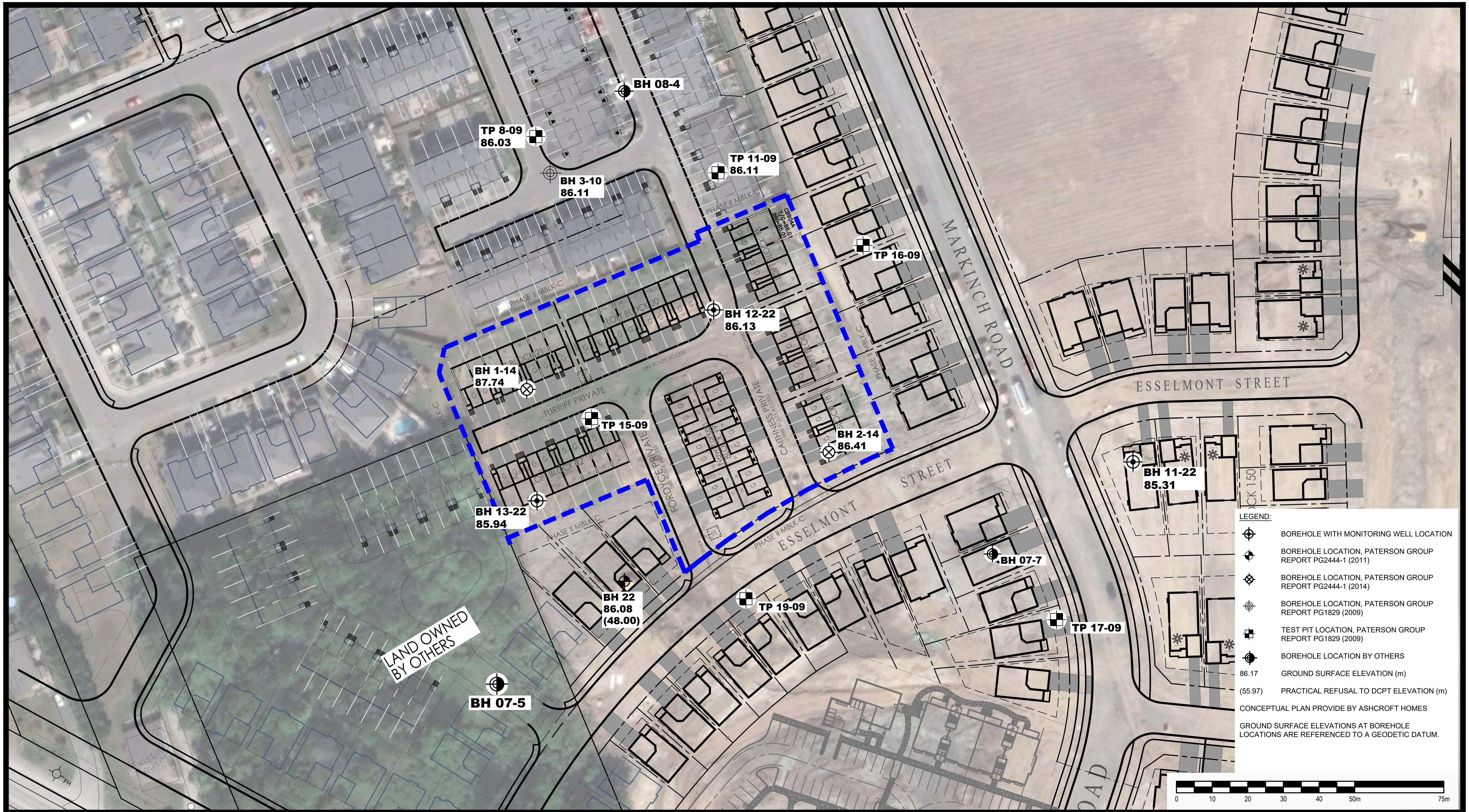


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
◕	BOREHOLE LOCATION, PATERSON GROUP REPORT PG2444-1 (2011)
⊗	BOREHOLE LOCATION, PATERSON GROUP REPORT PG2444-1 (2014)
◎	BOREHOLE LOCATION, PATERSON GROUP REPORT PG1829 (2009)
▚	TEST PIT LOCATION, PATERSON GROUP REPORT PG1829 (2009)
◉	BOREHOLE LOCATION BY OTHERS
86.17	GROUND SURFACE ELEVATION (m)
(55.97)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

CONCEPTUAL PLAN PROVIDE BY ASHCROFT HOMES

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

NO.	REVISIONS	DATE	INITIAL
1	2022 BOREHOLES ADDED AND BASEPLAN UPDATED	06/07/2022	KP

ASHCROFT HOMES
GEOTECHNICAL INVESTIGATION
EASTBORO RESIDENTIAL DEVELOPMENT
MULTIBLOCK C

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 06/2021
Drawn by: YA
Report No.: PG2444-3
Checked by: YT
Dwg. No.: PG2444-5
Approved by: FA
Revision No.: 1

c:\users\robertg\documents\pg2444 muiltblock c.dwg