patersongroup

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation
Avalon Aquaview Stage 2 Residential Development
Aquaview Drive at Lakepointe Drive
Ottawa (Cumberland), Ontario

Prepared For

Minto Communities Inc.

April 2, 2018

Report: PG4444-2

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

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

Table of Contents

Page

1.0 Introduction 1

2.0 Proposed Development 1

3.0 Method of Investigation
- 3.1 Field Investigation 2
- 3.2 Field Survey 4
- 3.3 Laboratory Testing 4

4.0 Observations
- 4.1 Surface Conditions 4
- 4.2 Subsurface Profile 5
- 4.3 Groundwater 7

5.0 Discussion and Recommendations
- 5.1 Geotechnical Assessment 8
- 5.2 Site Grading and Preparation 9
- 5.3 Foundation Design 11
- 5.4 Recommended Review of Grading Plans 18
- 5.5 Design for Earthquakes 19
- 5.6 Basement Floor Slab 20
- 5.7 Pavement Structures 20

6.0 Design and Construction Precautions
- 6.1 Foundation Drainage and Backfill 22
- 6.2 Protection Against Frost Action 22
- 6.3 Excavation Side Slopes 23
- 6.4 Pipe Bedding and Backfill 23
- 6.5 Groundwater Control 27
- 6.6 Winter Construction 27
- 6.7 Landscaping Considerations 28
- 6.8 Corrosion Potential and Sulphate 29
- 6.9 Supplementary Structures and Additions 29

7.0 Field Review and Materials Testing Services 33

Table of Contents cont'd

Page

8.0 Statement of Limitations 34

Appendices:

Appendix 1:	Soil Profile and Test Data Sheets Symbols and Terms Consolidation Test Results Atterberg Limits Results Analytical Test Results
Appendix 2:	Table 1: Summary of Subsurface Information - Aquaview Stage 2 Table 2: Summary of Consolidation Test Results - Aquaview Stage 2 Figure 2: Undrained Shear Strength Profile - Aquaview Stage 2
Appendix 3:	Figure 1: Key Plan Drawing PG4444-1: Test Hole Location Plan - Aquaview Stages 1 & 2 Drawing PG4444-2: Permissible Grade Raise Plan - Stages 1 & 2

1.0 Introduction

Paterson Group Inc. (Paterson) was commissioned by Minto Communities Inc. (Minto) to conduct a geotechnical investigation for the Avalon Aquaview Stage 2 development, located on Aquaview Drive at Lakepointe Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 3). The geotechnical investigation update for Avalon Aquaview Stage 1 (also referenced in the Key Plan) has been reported under separate cover in our Report PG4444-1.

The purpose of the current investigation has been to determine the subsoil and groundwater conditions at the proposed Avalon Aquaview Stage 2 development site by means of twelve (12) boreholes and, based on the results of the boreholes, to provide geotechnical recommendations pertaining to the currently proposed development.

The geotechnical investigation has been completed in general accordance with the requirements of the City of Ottawa's *Geotechnical Investigation and Reporting Guidelines for Development Applications*.

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 work of this present investigation. Therefore, the present report does not address environmental issues. A Phase I - Environmental Site Assessment has been prepared by Paterson for the development, and reported under separate cover.

2.0 Proposed Development

The subject development property is referenced as Block 205, Registered Plan 4M-1172, and is located directly to the west/northwest of the EUC Neighbourhood 2 (Avalon North) stormwater management pond (SWMP). The site is relatively flat, except along the south and southeast sides where berms of fill are present. The subject Stage 2 property is generally covered with a layer of fill of variable thickness.

Residential development, consisting of executive, rear lane, and back-to-back town home designs, is presently proposed for this stage of development. Local roadways and municipal services will also be required to service the subject development.

3.0 Method of Investigation

3.1 Field Investigation

The investigation fieldwork program was scheduled by Paterson after clearing underground services. The fieldwork was conducted over the interim of February 23 to March 1, 2018, and consisted of the putting down of a total of twelve (12) boreholes. The applicable boreholes are numbered BH 1 to BH 12, inclusive. The locations of the test holes are shown on the Stage 2 Parcel on the Test Hole Location Plan, Drawing No. PG4444-1, in Appendix 3.

The boreholes were put down using a track-mounted auger drill rig operated by a two-men crew. 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.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using a 50 mm diameter split-spoon sampler or 73 mm diameter thin walled Shelby tubes in combination with a piston sampler. A field vane was used to determine the shear strength of the cohesive soils encountered. Auger cuttings samples were recovered of surficial soils.

The split-spoon and auger samples were classified on site and placed in sealed plastic bags. The Shelby tubes were sealed at both ends. All samples were transported to our laboratory. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the boreholes are shown as AU, SS and TW, respectively, on the applicable Soil Profile and Test Data (SPTD) sheets in Appendix 1. Transportation of the samples was completed in general accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soil Samples.

Vane testing is more appropriate for the evaluation of the strength of cohesive soils than split spoon sampling in conjunction with conducting the Standard Penetration Test. However, it was important to recover samples of the soil as well as determining its strength. As such, in many cases, therefore, after conducting a vane test, the spit spoon sample was then pushed through a 0.6 m long increment of depth including the vane test location to recover a sample of the soil. In these locations, the SS sample is noted, but the value for the SPT "N" value is shown as "P", for "pushed". For BH 1 and BH 12, SS samples were obtained throughout the sampled borehole depth.

Except as noted above, 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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

The thickness of the silty clay layer and the inferred bedrock depth were evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at five (5) of the boreholes. 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.

Due to the low resistance exerted by the silty clay in BHs 5 and 10, the cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The hammer was then used to further advance the cone to practical refusal. In BHs 1, 7 and 12, the DCPT was conducted for the full penetration of the cone between the sampled depth and the practical refusal of the cone.

Undrained shear strength testing was conducted in the cohesive soils encountered in the boreholes using a field shear vane apparatus. This testing was done in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil. In some locations, the soil through which the vane test was conducted was then sampled by following with the split spoon sampler.

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

Groundwater

Flexible stand pipes were installed in all twelve (12) boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples from the current investigation will be stored in our 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 locations of the test holes in the field and the ground surface elevation at the test hole locations were determined by Stantec Geomatics (Stantec). It is understood that the ground elevations are referenced to Geodetic datum.

The locations and ground surface elevations of the test holes are presented on Drawing No. PG4444-1, included in Appendix 3.

3.3 Laboratory Testing

The soil samples recovered from the boreholes were examined in our laboratory to review the results of the field logging. From the twelve (12) boreholes completed with this investigation, twenty-four (24) Shelby tube samples were recovered using a piston sampler. Of these samples, nine (9) Shelby tube samples were submitted for unidimensional consolidation testing and three (3) of these samples were subjected to Atterberg Limits testing.

In addition to the three (3) Shelby tube samples noted above, eight (8) split spoon samples from shallower depths were also subjected to Atterberg Limits testing. A shrinkage test was also conducted. Water content testing was conducted of all samples recovered from BHs 1 and 12.

The results of the consolidation and Atterberg Limits testing are presented on the Consolidation Test and Atterberg Limits' Results sheets, respectively, in Appendix 1. The consolidation test results are also summarized in Table 2, in Appendix 2, and are further discussed under Sections 4 and 5 of this report.

4.0 Observations

4.1 Surface Conditions

The subject Aquaview Stage 2 development property is located west and northwest of the existing Avalon North (EUC Neighbourhood 2) SWMP. The site is relatively flat, except along the south and southeast sides where berms of fill are present. The subject Stage 2 property is generally covered with a layer of fill of variable thickness. Perusal of historical aerial photos indicate that there was site activity when the SWMP was being constructed, and several berms or windrows of apparent fill were present after completion of the pond. As such, it is our interpretation that existing fill, and some areas of disturbed surficial soils, cover much of the property and that the original ground surface elevation is generally below the present ground surface levels.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the test holes locations consist of fill deposits, overlying a deep deposit of sensitive silty clay. The bottom part of the overburden is inferred from the DCPT results to consist of heavily overconsolidated very stiff silty clay and/or a layer of glacial till. The bedrock surface, inferred from the depths of practical cone refusal, ranges from 24.4 to more than 36.3 m depth.

Reference should be made to the Soil Profile and Test Data (SPTD) sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location. A summary of subsurface information is provided in Table 1, in Appendix 2.

Existing Fill and Topsoil

Existing fill deposits were encountered overlying the native soils at all test holes. As previously noted, the site is located adjacent to the Avalon North (EUC Neighbourhood 2) SWMP and the fill is interpreted to consist primarily of excavated clay from the large pond that was stockpiled and spread over the site. The existing fill therefore primarily consists of clean site-excavated silty clay with some topsoil/organics. It is also expected that there will be localized areas where the previous site activities, such as hauling by off-road vehicles, may have disturbed the upper native soils, to shallow depth, making them look like clean native fill.

Part of the subject site had been stripped to partly stripped prior to the placing of the existing fill materials. It was not easy to establish the native soil levels in the boreholes, as sampling was not continuous and the fill is clean and not easily differentiated from the in situ soil. As such, the thickness of the fill deposits was interpreted from the perusal of the recovered soil samples and review of a few boreholes in the general area of the site from 1992 and 1999 investigation phases that pre-dated the site activities associated with the SWMP.

Silty Clay

Silty clay was encountered beneath the remnant topsoil and/or fill at all test hole locations. Based on DCPTs, the overburden soils, of which the silty clay is the predominant soil, has been inferred to extend to depths of between 24.4 to more than 36.3 m within the Stage 2 development area.

The upper portion of the silty clay has been weathered to a brown to grey desiccated “crust” at all test hole locations and extends to depths of between 2.3 and 3.4 m (mean 2.8 m) below the original ground surface. In situ shear vane field tests carried out within the silty clay crust yielded peak undrained shear strength values in excess of 50 kPa in the stiff to very stiff ranges. The SPT N values measured in the crust were generally in excess of 7. These values reflect a stiff to very stiff consistency in the silty clay crust. The crust strength was generally characterized with vane testing and then samples were recovered where necessary by following with the split spoon sampler.

The thickness of the “crust” and the elevation of the underside of the crust, as interpreted at each test hole location, are summarized in Table 1, in Appendix 2.

Grey silty clay was encountered below the brown silty clay crust in all test holes. In situ shear vane field testing conducted within the grey silty clay layer yielded undrained shear strength values generally ranging from 25 to 50 kPa. These values are indicative of a firm to stiff consistency. A few results were in the 20 to 25 kPa range indicating a soft consistency. The results of the DCPTs indicate that the shear strength of the grey silty clay increases gradually with depths into the stiff and very stiff range.

Nine (9) Shelby tube samples of the grey silty clay, of the 24 Shelby tube samples collected from the boreholes in this investigation, were subjected to unidimensional consolidation testing. These results are presented in Appendix 1. The consolidation test results are also summarized in Table 2, in Appendix 2.

The natural water content of the silty clay materials, was tested for all samples recovered from BH 1 and BH 12 to provide a profile of the water content of the soil. Water contents were also measured in the consolidation test samples. The water contents in the brown silty clay crust generally range from 35 to 45%. The water contents in the grey silty clay range from 52 to 96%, with a mean of 68%. The sensitivity of the silty clay can generally be classified as within the “sensitive” range.

The results of Atterberg Limits tests conducted on eight (8) samples from relatively shallow depth within the brown crust, and on three (3) selected samples of the grey silty clay, are presented on separate Atterberg Limits’ Results sheets in Appendix 1. All the tested silty clay samples classify as clay of high plasticity (CH) in accordance with the Unified Soil Classification System. Two (2) of the samples of the grey silty clay have plasticity indices less than 40% and all the other results are in excess of 40%. Sample SS3 from BH 6 has a shrinkage limit of 19% and a shrinkage ratio of 1.82.

Glacial Till

A glacial till layer could be inferred, from the results of the DCPTs, to be present below the silty clay in several deep boreholes. This could also be heavily overconsolidated very stiff silty clay. Glacial till typically consists of a fine soil matrix mixed with gravel, cobbles and boulders, and generally overlies bedrock directly.

Practical Refusal to DCPT and Augering

Practical refusal to DCPT penetration was observed in four (4) of the boreholes where a cone was driven. In BH 10, the DCPT did not encounter refusal before the drill ran out of drill rods at 36.3 m depth. This information is shown on the SPTDs, in Appendix 1 and is also summarized in Table 1, in Appendix 3.

Based on available geological mapping, the bedrock in this area mostly consists of interbedded limestone and shale of the Lindsay formation. The bedrock is expected to be encountered at depths ranging from 24.4 to in excess of 36.3 m.

4.3 Groundwater

The groundwater levels, measured in the boreholes relatively soon after the completion of the current 2018 fieldwork program, are presented in Table 3, below.

Table 3: Summary of Groundwater Level Readings in Boreholes

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Remarks
		Depth	Elevation	
BH 1	89.18	0.44	88.74	
BH 2	90.50	4.66	85.84	GWL in BH not stabilized
BH 3	89.08	0.86	88.22	
BH 4	88.97	0.00	88.97	Water ponded at surface
BH 5	89.62	3.16	86.46	GWL in BH not stabilized
BH 6	89.38	1.82	87.56	
BH 7	88.92	0.85	88.07	
BH 8	89.08	1.40	87.68	
BH 9	89.18	0.36	88.82	
BH 10	89.00	Dry	N/A	BH dry and/or blocked
BH 11	89.18	0.44	88.74	
BH 12	89.77	0.50	89.27	

Note: 1. Readings were taken in the standpipe tubing in the boreholes on March 28, 2018.

The measured groundwater levels range from ground surface to 4.66 m depth, but the deeper measured levels are interpreted to be due to “lag” time in the stabilization of the groundwater level, and ponded water influenced BH 4. As such, the groundwater level is expected to be located between 0.4 and 1.8 m depth.

It is our experience that the groundwater levels in the area of the development typically are found at shallow depth, perched in the drier and fissured weathered stiff crust over the grey unweathered silty clay near the base of the crust.

Based on the depth of development of the stiff clay “crust”, the long-term low pre-development groundwater levels are expected to be about 0.5 m above the base of the crust, or between approximately 1.8 and 2.9 m (mean 2.3 m) below the original ground surface.

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

5.0 Discussion and Recommendations

5.1 Geotechnical Assessment

The residential development is presently proposed to consist of a combination of executive town homes, rear lane town homes and back-to-back town homes; all to be founded on conventional footing foundations. Local roadways and municipal services are also required to service the subject development.

Generally, the subsoil conditions at the test hole locations consist of existing fill of variable thickness and a thin and discontinuous topsoil layer overlying a thick sensitive silty clay deposit. The subsurface conditions are favourable for shallow foundation design and light residential structure types, such as one to three storey wood-frame residential structures.

Due to the presence of the sensitive silty clay layer, the subject site will be subjected to grade raise restrictions. As part of the preparation of this report, permissible grade raises have been evaluated at each site-specific borehole location, and a permissible grade raise plan (in conjunction with Aquaview Stage 1) has been prepared.

Two cases should be considered when determining the allowable bearing pressures for the design of shallow footings placed within the silty clay. Namely, the shear failure case and the settlement (serviceability) case. For design purposes and using the serviceability case, footings for conventional housing can be designed using the preliminary bearing resistance values presented in Subsection 5.3, to be confirmed by the geotechnical consultant during construction as part of the field review program.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and any existing non-specified fill 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 buildings, between footings and foundation walls, and under the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or 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 buildings and below the subgrade level of paved areas should be compacted to at least 95% of its standard Proctor maximum dry density (SPMDD), although stricter compaction requirements are applicable for fill directly below footings and within the base and subbase layers of pavements.

The zone of influence of a footing is considered to be the area beneath the footing limited sideways by planes extending out from the bottom edges of the footing, at a slope of 1H:1V, and down to the undisturbed in situ soil (below any fill or organic matter). Throughout the zones of influence of the footings, the engineered fill should consist of OPSS Granular A crushed stone or Granular B Type II materials. As a bedding for subfooting XPS or EPS insulation, where used, or during cold weather placing conditions, the engineered fill can consist of clean asphalt coarse aggregate.

The above-noted fill materials 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 within the zones of influence of the footings should be compacted to at least 98% of the material's 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. There is no specific compaction requirement in soft landscaped areas, but these materials can be spread in thin lifts and compacted/consolidated by the tracks of the spreading equipment to minimize voids.

If these fill materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Based on our experience with Avalon, occasional shallow ditches (not visible now because of the existing fill) can be expected to cross the development site. Where ditches or other fill or disturbed soil features extend below the proposed footing level, the applicable structures could be structurally supported by the use of engineered granular fill, as noted above, to fill the portion of ditch channel to re-establish the footing level. This method can be used where the thickness of the fill required below the footing level does not exceed 200 mm. However, for deeper ditch/sub-excavation locations, should they be encountered, it is recommended that site excavated native stiff to very stiff brown clay be used as the engineered fill material, as described below.

This will provide conditions within the area to be treated that are similar to the native soil conditions, and reduce the propensity for additional differential settlement due to the weight of the engineered fill. The brown clay, not the grey clay, should be used in a controlled backfilling program to fill deep ditches or sub-excavations, extending below the footing levels of structures, if required.

After preparing undisturbed surfaces within the applicable portion of the existing drain profile, or sub-excavation, the brown clay should be placed in thin lifts and compacted, using padfoot compaction equipment, to densities equivalent to 100% of the material's standard Proctor density at in situ water content. The allowable thickness of the lifts will be dependent on the compaction equipment used, and should be thin enough to ensure that the voids are removed from the entire thickness of the lift to create a homogeneous material.

The need or not to "cap off" the native fill with a thin engineered granular fill layer under footings will be assessed as part of the observational program by the geotechnical consultant during construction. Where weather conditions prohibit the use of native fill, 10 mm clear crushed stone can also be used as a lighter weight alternative to graded granular materials, if suitably compacted and encapsulated in a non-woven geotextile.

5.3 Foundation Design

Limit States Design

The Ontario Building Code (OBC 2012) Part 9 residential structures that are proposed for the development will be founded on sensitive silty clay. As such, it is a requirement that the foundations for the proposed structures be designed according to the requirements of Part 4 of the OBC 2012.

Limit States Design is the only design method permitted under Part 4 of OBC 2012. As such, the footing foundations for the structures are required to be designed for both the Bearing Resistance at Serviceability Limit States (SLS) and the Factored Bearing Resistance at Ultimate Limit States (ULS).

The bearing resistance at SLS pertains to the permissible (geotechnical) serviceability or deformation-related bearing resistance. Unfactored foundation loads are used in conjunction with the bearing resistance at SLS values, other than reducing live loads to represent “sustained loading” conditions when undertaking consolidation settlement analyses. At this development, for the types of structures presently proposed, the bearing resistance at SLS will generally govern the foundation design for structures to be supported on footings.

The factored bearing resistance at ULS pertains to the ultimate (geotechnical) capacity of the bearing medium, reduced by a geotechnical resistance factor. The geotechnical resistance factor is 0.5 for footing foundations. Factored loads are used in conjunction with the factored bearing resistance at ULS values.

Bearing Resistance at SLS

Founding conditions at this site are favourable for the construction of the light residential structures that are expected to be constructed, provided that the grade raise is within an acceptable range. Based on the subsurface profile encountered, it is expected that firm to stiff silty clay will generally be encountered at the founding levels of conventional full-basement structures or slab-on-grade back-to-back town home structures. An interpretation of the base of “crust” level has been provided, for each bore hole location, in Table 1, in Appendix 2.

At this development, for the types of structures presently proposed, the bearing resistance at SLS (equivalent to the allowable bearing pressure) will generally govern the foundation design for structures to be supported on footings.

Footings for structures with up to 60 kN/m full (unfactored) foundation wall loads and 100 kN full (unfactored and not including footing weight) column loads can generally be designed using a **bearing resistance at SLS value of 70 kPa**. A **factored bearing resistance at ULS value of 105 kPa** (incorporating a geotechnical resistance factor of 0.5) can be used for the above foundation loading cases. These ranges of foundation loads are typical of one to two-storey wood-frame executive town home (and/or singles) structures.

Footings for structures with up to 120 kN/m full (unfactored) foundation wall loads and 160 kN full (unfactored and not including footing weight) column loads can generally be designed using a **bearing resistance at SLS value of 60 kPa**. A **factored bearing resistance at ULS value of 90 kPa** (incorporating a geotechnical resistance factor of 0.5) can be used for the above foundation loading cases. These ranges of foundation loads are typical of wood-frame stacked units, such as slab-on-grade back-to-back town home or semi-basement terrace homes, that do not have an elevator core or underground parking.

Depending on the structure configuration, such as garages, porches, slabs-on-grade, and the grade raise as compared to permissible grades (Table 4), lightweight fill (LWF) materials may be recommended for the above-noted bearing resistance values to be applicable, especially where the grade raise levels approach the maximum permissible, and as the depth of foundations approaches the underside of stiff crust levels.

Bearing Medium Review Observations

On-site bearing medium assessment observations are recommended as part of the geotechnical field review, and this has been an important facet of the “sensitive soil protocol” used by Paterson on other development in this area. The results of shear strength testing conducted from the excavation level is used to assess the bearing resistance at SLS, for the sizing of footings, based on footing matrices prepared by the structural engineer and reviewed, in consideration of the (approved) proposed grading by the Paterson geotechnical project manager.

As such, for one to two-storey wood-frame full basement town homes(and/or singles) and slab-on-grade back-to-back town homes within the subject development, the bearing resistance at SLS value can be confirmed on a town home block-by-block or a lot-by-lot basis, based on traditional shear strength testing within the house excavations at the time of construction.

The above-noted bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil or suitably prepared engineered fill bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil, unspecified fill, 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.

Considering that the footing levels are expected to vary, based on the finished grading, the thickness of remnant stiff silty clay “crust” under the USF is expected to vary. As such, it may be prudent, as has been done on similar projects, for the structural engineer to prepare footing size matrices for various bearing resistance at SLS values. Typical bearing resistance at SLS values would be 100, 85, 70, 65 and 55 kPa.

Where fill is required to raise the grade below the footing level, or to replace unsuitable material, the fill located within the zones of influence of the footings should consist of engineered fill, as described under Subsection 5.2. The bearing resistance at SLS (allowable bearing pressure) values for footings placed on engineered fill should be equivalent to the above-noted values for footings on native soil.

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 stiff to firm silty clay 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

In addition to the shear failure case, consideration must be given to potential post construction settlements when determining the bearing resistance at SLS (allowable bearing pressure) values. The potential settlements can occur due to the compression of the deep silty clay deposit under the loads from the footings, the grade raise fill pressures and groundwater lowering effects.

The foundation loads to be considered for the settlement analyses are the continuously applied loads, which can be taken as the unfactored dead loads and a portion of the unfactored live loads. We have conservatively considered 50% of the live load as part of the continuously applied loads, for the residential structures anticipated at this site.

Settlement analyses carried out to estimate the potential post construction differential and total settlements at this site consider a continuously applied wall load of between 30 and 45 kN/m on footings designed based on the shear strength. Actual footing widths for construction will generally be based on the results of the individual lot assessments at the time of construction, using the full live and dead loads (wall load of 60 kN/m and column load of 100 kN) and the shear strength profile of the bearing medium soil.

Acceptable Total and Differential Settlements

The following discussion is based on the assumption that acceptable total and differential settlements for the proposed structures are 25 and 20 mm, respectively.

Consolidation Testing

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. The results of the consolidation tests are presented in Appendix 1 and are summarized in Table 2, in Appendix 2.

Value p'_c is the preconsolidation pressure of the sample and p'_o is the effective overburden pressure. The difference between these values is the available overconsolidation. 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 overconsolidation if unacceptable settlements are to be avoided.

The values C_{cr} and C_c are the recompression and compression indices, respectively, and are a measure of the compressibility of the soil 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.

It should be noted that the values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering practices and are estimates only. In addition, natural variations within the soil deposit would also affect the results. Furthermore, the p'_o parameter is directly influenced by the groundwater level. While the groundwater levels were measured at the time of the fieldwork, the stabilized levels are difficult to determine and these values have a direct impact on the available overconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available overconsolidation. 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 site-specific investigation, as summarized in Table 2, are based on the interpreted original ground surface, a 2.8 m thick crust, and a pre-development seasonal low groundwater level being 2.3 m below the original ground surface.

The shear strength tests conducted in the boreholes included in this report have been plotted against elevation and this information is provided graphically on Figure 2, in Appendix 2.

The total and differential settlements will be dependent of the characteristics of the buildings. For design purposes, the total and different settlements are estimated to be 25 and 20 mm, respectively, for the expected grade raises and for total foundation wall loads not exceeding 60 kN/m and total column loads not exceeding 100 kN (excluding pad footing weight). A post-development groundwater lowering of 0.5 m was assumed. As such, larger and heavier stacked units, may require lesser grade raises than the tabulated maximum permissible, or may require more extensive use of LWF materials.

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

Table 4, on the following page, presents permissible grade raises at the borehole locations. This information is also plotted on the Permissible Grade Raise Plan, Drawing No. PG4444-2, in Appendix 3.

Our grading analyses considered a long-term groundwater level drawdown of 0.5 m, used approximately 80% of the estimated soil overconsolidation, continuously applied foundation wall loads of 50 kPa on a 0.9 m wide footing (i.e. continuously applied wall loads of 45 kN/m), and conventional slab-on-fill garage construction. The foundation load represents full dead load and 50% of live load, as discussed previously. Note that these foundation loads are typical of one to three storey wood-frame structures.

Analyses were also conducted for conditions where lightweight fill (LWF) material is used under the garage (and porch) slab-on-grade of conventional residential singles (and town homes). The LWF, as described later in this section, is used to reduce the weight of the garage fill and, thereby, reduce the estimated settlement of the garage footings, which are the limiting serviceability design case for a conventional house.

Based on geotechnical considerations, and the above-noted criteria, permissible grade raises for conventional (i.e. no LWF) construction, and construction using LWF have been determined at each borehole location and are summarized in Table 4. For the review of the grading plans, under subsection 5.4, the permissible grade raises thickness values have also been expressed as finished grades, for easier application to the design of the site grading. Note that the finished grades provided are to be measured at the front of the garage.

TABLE 4: Permissible Grade Raise at Borehole Locations

Borehole Number	Original Ground Elev. (m)	Permissible Grade Raise - No LWF		Permissible Grade Raise - LWF	
		Raise (m)	Fin. Grade (m)	Raise (m)	Fin. Grade (m)
BH 1	88.80	0.80	89.60	1.20	90.00
BH 2	88.80	0.60	89.40	1.00	89.80
BH 3	88.80	0.70	89.50	1.10	89.90
BH 4	88.90	0.80	89.70	1.20	90.10
BH 5	89.00	0.70	89.70	1.10	90.10
BH 6	88.80	0.70	89.50	1.10	89.90
BH 7	88.70	0.60	89.30	1.00	89.70
BH 8	88.80	0.80	89.60	1.20	90.00
BH 9	89.00	0.50	89.50	0.90	89.90
BH 10	88.60	0.90	89.50	1.30	89.90
BH 11	88.80	0.70	89.50	1.10	89.90
BH 12	89.10	0.60	89.70	1.00	90.10

Notes:
1. “Permissible Grade Raises - No LWF” are based on conventional wood-frame town home (or single home) housing construction with normal weight fill within garage and porch slabs-on-grade.
2. “Permissible Grade Raises - LWF” are based on installing EPS LWF in garage and porch cavities. Higher grade raises may be possible with exterior EPS LWF.

The basic grade raise limits are referenced with respect to the garage, as this is the most critical grading condition for the structures, in terms of differential settlement, for conventional slab-on-fill garage construction. The basic permissible grade raise is limited by settlement of the garage and, as such, can be exceeded by up to 0.4 m to 0.5 m if LWF is used to reduce the settlement of the garage (and slab-on-fill porch) portion of the house, as described elsewhere in this report.

To reduce potential long term liabilities, consideration should be given to accounting for groundwater lowering and to providing means to reduce long-term groundwater lowering (e.g. clay dykes, restriction on planting around dwellings, etc). It should be noted that building on silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in all foundations, especially if placed at key structural locations, will tend to reduce foundation cracking as compared to unreinforced foundations. It should be noted that building on thick silty clay deposits increases the likelihood of house movements and therefore of cracking.

Means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the dwellings, etc) should be implemented for the proposed development. It is not possible to economically prevent potential cracking of foundation walls and slabs in residential construction using standard construction practices. The use of properly installed reinforcement in foundations will tend to reduce foundation cracking as compared to unreinforced foundations. Note that building on thick silty clay deposits increases the potential for house movements and therefore of cracking.

Sensitive Soil Foundation Design and Field Review Protocol

The reader should be aware that the City of Ottawa Building Services Branch has recommended a sensitive soils foundation design and field review protocol that has been fully implemented by Minto and their geotechnical and structural engineering consultants, and is considered to be applicable to the subject development.

The sensitive soils protocol is a comprehensive and holistic methodology that ensures that the structural design incorporates the geotechnical design elements. In addition, the construction-related elements, such as field measured shear strengths, underside of footing levels, grade raises, are checked or reviewed during construction and compared with the design assumptions, so modifications can be made where the results of the field observations warrant, and otherwise, as-built conditions can be confirmed to be according to design assumptions.

Bearing resistance at SLS values for footing designs should be confirmed on a lot-by-lot (or town home block-by-block) basis at the time of construction, as part of the protocol (medium evaluation), to refine the recommended design values provided in this report. The bearing resistance values provided earlier in this report are preliminary in nature and are subject to being confirmed at the bearing medium evaluation stage.

Use of Lightweight Fill

Lightweight fill (LWF), consisting of EPS (expanded polystyrene foam) Type 1 blocks can be used where the permissible grade raises are exceeded, but the structure can still be designed for footing foundations. LWF is also recommended where permissible grade raises are not exceeded, but either the thickness of the remaining crust under the footing level is considered to be insufficient to support the garage and porch fill loads and/or the foundation loads in the garage and/or porch exceed “design” values.

Use of EPS LWF within the interior of the garage and porch areas of conventional full basement singles (or town home) structures to reduce the fill-related loads, can allow the permissible grade raises noted in the right columns of Table 4.

5.4 Recommended Review of Grading Plans

Atrel has prepared a Macro Grading Plan for the proposed development lands, as detailed on Atrel Drawing No. 171203-GRM, as part of their Assessment of Adequacy of Public Services, Project No: 171203. Paterson has reviewed the Macro Grading Plan, from a geotechnical perspective, with reference to the recommended permissible grade raise (see below). In our opinion, the macro grading of the roads is consistent with the permissible grading. Residential structures within portions of the Stage 2 site will require the use of LWF (lightweight fill), however the macro grading achieves the goal of enhancing site cut-fill balance and the grading requirement, such as proper overland flow of storm water.

With reference to our review of the macro grading, the permissible grade raises provided in Table 4 are based on the finished grade at the front of the garages of the residential structures. When preparing master or macro grading of the streets, a permissible centreline of road grade value of 0.3 to 0.4 m below the tabulated values should be used to account for the houses being graded above the road grades.

Paterson Group should be consulted to provide geotechnical review of the comprehensive grading plans to ensure the proposed site grading continues to conform to the intent of our geotechnical recommendations concerning grading. Our review of the grading plans will be based on an interpretation of the proposed grades and underside of footing levels to ensure our foundation design soil parameters for bearing and settlement (25 mm total and 20 mm differential) are valid.

5.5 Design for Earthquakes

Near surface seismic reflection-refraction testing was carried out by Paterson, in December of 2009, on the nearby Aquaview Stage 1 property, to estimate the shear wave velocity. The test findings indicated that the average shear wave velocity, using the OBC 2012 averaging formula, of the upper 30 m of the profile, V_{S30}, is less than 180 m/s. The average shear wave velocity, V_S, of the soil overburden portion of the profile was measured to be 133 m/s and the bedrock was interpreted to have V_S of 1950 m/s. Using the OBC averaging formula, where the depth to bedrock is greater than 21.6 m, the value of V_{S30} will be less than 180 m/s, as shown in the formula, below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\sum\left(\frac{21.6m}{133m/s} + \frac{8.4m}{1{,}950m/s}\right)}$$

$$V_{s30} = 180m/s$$

The depth to inferred bedrock is in excess of 21.6 m at all five (5) boreholes in Stage 2 where DCPT testing was conducted, and therefore, the OBC averaging formula would indicate $V_{S30} < 180$ m/s. As such, the geological conditions at the subject site indicate that Site Class E ($V_{S30} < 180$ m/s) is appropriate for seismic design at the subject development site. It is our recommendation that **Site Class E** be used for foundation design for the Aquaview - Stage 2 development.

With respect to seismic liquefaction of the silty clay deposits, the Bray criteria states that soils with a plasticity index, PI, less than 20% may have reduced strength following the strong cyclic loading resulting from a severe earthquake. If reference is made to the Atterberg Limits test results, in Appendix 1, all of the tested samples tested have plasticity indices greater than 20, and therefore no strength reduction is to be expected during the strong ground motions of an earthquake.

As such, the soils underlying the Aquaview Stage 2 site are not susceptible to seismic liquefaction.

5.6 Basement Floor Slab

With the removal of all topsoil and fill, if any, within the footprint of the proposed residential buildings, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type I or 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 consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

It should be noted that, due to the sensitive silty clay encountered at this site**,** the subgrade is expected to be susceptible disturbance by construction traffic (workmen and equipment).

5.7 Pavement Structures

For design purposes, the pavement structures presented in the tables below, and on the following page, could be used for the design of car parking areas (Table 5), access lanes/local residential streets (Table 6) and collector roads with bus traffic (Table 7).

Table 5: Recommended Pavement Structure
Car Parking Areas

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

Table 6: Recommended Pavement Structure
Access Lanes/Local Subdivision Streets

Thickness mm	Material Description
40	**Wear Course -** SP 12.5 Asphaltic Concrete
50	**Binder Course -** SP 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
375	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either suitable existing fill, in situ silty clay or OPSS Granular B Type I or II material placed over in situ soil or suitable fill.	

Table 7: Recommended Pavement Structure
Collector Roads - Bus Traffic

Thickness mm	Material Description
50	**Wear Course -** SP 12.5 Asphaltic Concrete
100	**Binder Course -** SP 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either suitable existing fill, in situ silty clay or OPSS Granular B Type I or II material placed over in situ soil or suitable 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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, a biaxial geogrid, 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

It is recommended that the road structure granular layers be protected from surface water. 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.

The pavement structure should maintain a suitable crown to shed water towards the available storm sewer catch basins. Consideration should be given to the placement of subdrains along the pavement edge for major roads, or “stubby” drains, leading into the catch basins at the subgrade level.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for each of the proposed structures. The system should consist of a 100 mm to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials, such as clean sand or OPSS Granular B Type I material. The greater part of the site excavated materials will consist of frost susceptible fine-grained soils and, as such, are not recommended for re-use as backfill against the foundations unless a composite drainage system (such as system Platon or Miradrain 6000 or G100N), connected to a foundation drainage system, is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures, including attached garages, 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 and wing walls may require more soil cover or a combination of soil cover and XPS or EPS insulation.

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through 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. The lowermost 1.2 m can be vertical provided the material consists of stiff in situ silty clay only. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not practical, temporary shoring should be used.

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.

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

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

6.4 Pipe Bedding and Backfill

General Recommendations

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. Trench details should be as per Detail Drawing Nos. W17, S6 and S7. Further guidelines concerning trench excavations are provided later in this section.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. Where the invert of the excavation is within grey silty clay the thickness of the bedding should be increased to 300 mm. The bedding should extend to the spring line of the pipe. Cover material, from the /spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

It should generally be possible to re-use the upper portion of the silty clay above the cover material if excavation and filling operations are conducted in dry weather. Due to its high natural water content, the wet grey silty clay will be difficult, if not impractical, to compact without an extensive drying period. Native trench backfill that is difficult to compact may be placed and consolidated in layers using ramping techniques.

Where hard surface areas will be located 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.

Seepage Barriers - Clay Seals

In order to reduce the potential consolidation of the compressible clay deposit, it is very important that no long-term groundwater lowering occur. To prevent the granular pipe bedding and pipe cover from acting as a "french" drain, it is recommended that clay dykes or seals be installed along service trenches situated below the water table.

The clay seals should be as per Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. The seals should be at least 1.5 m long (in the trench direction), as compared to the 1 m minimum in the detail, and should extend from trench wall to trench wall.

Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations, but no more than 60 m intervals apart, in the service trenches.

Trench Support

The installation of the proposed sewers in soils can be carried out safely within the confines of a trench box or in an open cut. An open cut in overburden materials will require that all side slopes be cut back at 1H:1V or shallower to maintain stability. If a shoring system (i.e. trench box) is used to support the walls of the cut, the trench box design should, for safety purposes, allow for additional surcharge pressures associated with construction equipment and stockpiled fill materials above the cut, although stockpiling of materials above excavations is strongly discouraged, considering the presence of deep deposits of sensitive silty clay.

The interpretation of the soil descriptions in the Occupational Health and Safety Act, Regulations for Construction Projects, for purposes such as trench box design, should be undertaken by experienced geotechnical personnel. The information provided in the Soil Profile and Test Data sheets in Appendix 1 can be consulted for this purpose, with due consideration that the information is only accurate at the applicable test hole locations, and in consideration that the contractor's equipment and methods, as well as the depth of the excavation, can have a significant effect on the actual earth pressures in force at the time of construction.

Basal Stability

There is a potential for basal heave to occur in deep excavations in firm to soft clay at the site. Our calculations indicate that there is a factor of safety against base heave of 2.0 for cuts of up to 5.5 metres in depth, and a factor of safety of 1.5 for cuts of 7.5 metres in depth for clays with a shear strength of 25 kPa. Note that where higher or lower shear strengths are encountered at the above-noted trench depths, the applicable factors of safety against base heave, would be higher or lower by a proportional amount to the increase or reduction in shear strength from 25 kPa.

Deeper cuts, or cuts intercepting soft clay will tend to have lower factors of safety and, therefore, increasing basal instability. Trenches with factors of safety of less than 2.0 against basal heave tend to be problematic with respect to squeezing of the excavation base and sides.

Improved basal stability can be provided by keeping excavation lengths shorter and trench widths narrower. Cutting back the sides of the excavation at shallower slopes and/or "benching" the top of the excavation sides, also provides increased stability in this regard. The beneficial effects of the benching are improved by widening the benches and increasing the depth of the benches. Excavated materials can exert a surcharge and should not be placed beside the top of the trench cut. These materials should be placed a lateral distance equivalent to a minimum of 1.5 times the trench depth away from the side of the trench in order to minimize their surcharge effects. Where the work area for the shovel is weak, the use of steel plates, beams and/or wood timbers, under the front of its tracks should be considered.

One area of the site was encountered with a measured shear strength below 25 kPa. This area may require more attention to sloping the excavation sides, or benching of the excavation side slopes, depending on the depth at which the lower strengths were encountered and the depth of the excavation. For the greater part, minimum measured shear strengths are in excess of 25 kPa.

Trenching, Supporting and Backfilling Procedures

Native trench fill materials that have just been placed, unless they have been thoroughly compacted using padfoot compaction equipment (which is generally not the case) can be considered to be very weak. As such, the backfilling of the trench with difficult to compact grey clay native fill materials should be accomplished by ramping with a small dozer or loader, working back and forth on a shallow ramp, placing the native fill in thin lifts.

It is recommended that the bedding and granular cover material be of a uniform density to provide optimum support to the pipe. Compaction of the bedding and cover materials will also enhance the stability of the trench during backfilling.

Improved performance of the pipe installation can be effected by using a combination of the following techniques:

1. Employ benching techniques to reduce the effective trench depth.

2. Properly support the excavator with plates and/or beams to distribute equipment surcharges beyond just the trench heading to the sides of the trench.

3. Keep unsupported trench lengths as short as practical.

4. Use the drier, upper, soils for trench backfilling and discard the wetter, lower, grey soils for use as general fill in landscaped areas, such as the boulevards. This will provide time for the wetter soil to dry out, while the better soil is used immediately in the trenches.

5. Use layered ramping techniques, with a loader or dozer, supplemented with a padfoot compactor, to backfill the trench. Follow-up as close as practical with the backfilling to the completion of the pipe installation and cover and the moving of the trench box.

6. Bulk up granular cover material against the sides of the trench box to provide granular material to fill the voids created by the walls of the trench box as it is moved forward.

7. Take particular care when moving away from drainage structures, such as manholes, to backfill carefully behind the trench box with the excavator, as backfilling by ramping will not be practical.

Trench Dewatering

Low to moderate rates of groundwater flow into excavations below the water table should be expected. The contractor should be prepared to pump water from the excavation to enable the installation of services to be carried out in the dry. It is expected that routine pumping from within the confines of the excavation will suffice where excavations are in clay. If onerous groundwater conditions develop, more elaborate dewatering may be required to deal with localized problems.

6.5 Groundwater Control

Due to the relatively low permeability of the silty clay material, 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. Where deep excavations are required, other dewatering means or cutoff barriers could be required.

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

The developer may need to register with the Ontario Ministry of Environment and Climate Change's (MOECC's) Environmental Activity and Sector Registry (EASR) process for this project if more than 50,000 L/day (and less than 400,000 L/day) are to be pumped during the construction phase (routine flows). Paterson can assist with this process.

Pumping of more than 400,000 L/day requires a temporary MOECC permit to take water (PTTW). At lead time of 4 to 6 months should be allowed for completion of the application and the review and issuance of the permit by the MOECC.

6.6 Winter Construction

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

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

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 Landscaping Considerations

The subject site is located in an area of sensitive silty clay deposits for tree planting. For the proposed development, it is expected that final grade raises will be approximately 0.7 to 1.4 m above existing grades. Therefore, it is expected that the combination of the proposed finished grades and the thickness of the underlying weathered clay crust will provide approximately 3 to 4 m thick buffer to the underlying firm to soft grey silty clay deposit.

Atterberg limits testing was conducted within the activity zone for tree roots on the Aquaview Stage 2 parcel and indicates the silty clay soils have plasticity indices ranging from 43 to 50%, indicating the soil underlying the site has a high volume change potential and has an elevated risk for shrinkage related to tree roots.

In our opinion, tree planting for this subject development should be limited to low to moderate water demand trees. Low water demand species include beech, birch, mulberry, cedar, fir, pine and spruce. Moderate water demand trees include ash, cherry, hawthorn, hornbeam, sugar and red maples, and mountain ash.

The minimum permissible distance from the foundation to the tree will depend on the nature of the tree, the depth of the clay crust and the final grade raise in relation to the permissible grade raise. In our opinion, the development, should be provided with a minimum tree to foundation clearance of 6.0 metres. In critical areas, the minimum permissible tree planting distance can be improved by installing various tree damage preventative measures such as:

- ❑ Exfiltration trenches with a moisture retention barrier
- ❑ Root barrier systems with water delivery systems
- ❑ Separation barriers
- ❑ Additional foundation reinforcement and support

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

6.8 Corrosion Potential and Sulphate

The results of analytical testing conducted on three (3) soil samples recovered during the site-specific investigation are provided in Appendix 1. The test results show that the sulphate content is less than 0.1%. This result is indicative that GU (general use) Portland cement, formerly Type 10 cement, would be appropriate for buried concrete structures at this site.

The chloride content and the pH of the tested samples 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 a moderate to very aggressive corrosive environment.

6.9 Supplementary Structures and Additions

Swimming Pools

Both in-ground and above-ground swimming pools are permitted, from a geotechnical standpoint, in the Aquaview Stage 2 development, with some precautions, as described below.

Installation of an in-ground pool will generally not result in a net increase in load to the soil provided the lot grading is not raised as part of the pool installation. The main issue for in-ground pools is that most of the soil removed from the pool excavation should be removed from the site. Any regrading of the ground surface around the pool is required to conform with the lot grading plan. The lot grading should not be significantly altered, other than to get level grading for the pool deck slab. As such, placing of large quantities of imported fill materials should also not be permitted, without prior review and written approval of the Geotechnical Consultant (i.e. Paterson Group). The normal building permit minimum clearances from structures are applicable for in-ground pools.

Installation of an above-ground pool will result in a net increase in load to the soil. To address concerns regarding the weight of a pool, we have undertaken settlement analyses to assess the loading effects of a 54" (1.4 m) high above-ground pool, located a minimum of 2.0 m away from the house foundation in the rear or a side of a town house or single house, with the pool located adjacent to the basement, NOT on a side adjacent to a garage. The grading for the analyses was assumed to meet, or be lower than, the permitted grade raise (i.e. per the eventual City-approved grading plan).

The pool water loading has been determined to result in negligible settlement effects on the foundation. For the pool itself, the estimated differential settlement between the perimeter and middle of the pool related to native silty clay consolidation (i.e. not including the settlement or compression of the landscaping fill layer above the original ground surface) is less than 15 mm, which is expected to be tolerable for a typical liner-type pool. The magnitude of the total and differential compression of any existing fill materials left in place under the pool will depend on the state of compaction and the uniformity of the fill and whether any organics are present.

We suggest placing a requirement for a minimum 2.0 m clearance from the foundation wall to the edge of an above-ground pool in the Aquaview Stage 2 development, in order to situate the pool outside the routine house excavation limits, and to satisfy the conditions used in our analyses. Regrading associated with above-ground pools should, like for the in-ground pools, respect the intent of the grading plan and be for the purpose of providing a level base for the pool structure and liner, rather than raising the general grading significantly.

Similar clearance restrictions should be applied to exterior hot tubs or spas, as for above-ground pools. These structures are generally required to be placed on a concrete slab or patio stone pad, so location outside the backfill zone from the foundations is important. This should generally be achieved with the requirement for a minimum 2.0 m clearance from the foundation wall to the edge of the hot tub. These structures are relatively small, so loading effects on the existing foundations are no worse than for above-ground pools.

For both in-ground and above-ground pool types (as well as hot tubs) the pool constructor is responsible for ensuring that the support conditions are adequate for the applicable pool structure. This is particularly important for above-ground pools, as the surficial soil is often clay fill of variable state of compaction. Granular fill can be added to provide level conditions for the base of above-ground pools. All site materials, including stripped topsoil and soil excavated from in-ground pool excavations will be frost susceptible and that should be considered with regard to its potential re-use for minor re-grading around the pool.

As far as the presence of exterior lightweight fill (LWF) is concerned, this is not expected to be required in Avalon Aquaview Stage 2. The LWF will generally only be installed in the garages and porches of the units and, therefore, should not be of concern for the pool issue.

Where/if an above-ground pool is proposed to be installed adjacent to a garage, a geotechnical review would be in order and a greater clearance to the garage may be in order than the 2.0 m. This situation would probably only be possible for end units of town homes that are on corner lots, so it is not considered to be a frequently imposed restriction.

Deck Structures

With regard to deck structures, the deck constructor is responsible for ensuring that the support conditions are adequate for the applicable deck structure. These structures are normally relatively lightly loaded and the foundation loads associated with their installation are not of particular concern to the existing foundations. As noted for pools, the upper part of the soil underlying the ground surface generally consists of clayey fill material of variable state of compaction, so the use of shallow-surface deck piers should be limited to independently supported structures that can tolerate potential differential settlement and frost action.

Some geotechnical or structural review may be warranted with regard to decks that are supported in part by attachment to the existing foundation wall. Proposed attached decks for houses or town home units that have lightweight fill and/or an allowable bearing pressure for footing design that is less than 75 kPa, should be required to be reviewed by the Geotechnical Consultant (i.e. Paterson Group) to ensure that our geotechnical recommendations are followed.

Proposed attached decks, where the foundations for the house have been designed to an allowable bearing pressure of at least 75 kPa, with no lightweight fill, will not generally be of geotechnical concern, but we cannot comment on potential structural concerns.

The City’s normal policy for review of the requirements for pier or post-type of foundations should be applied and will generally be adequate. The lower the design allowable bearing pressure for the house, however, the greater the tendency for weaker bearing conditions at the toe of post or pier-type foundations. Adfreeze bond breaks, such as the use of a “sonotube” form or polyethylene wrap should be used for augered piers to reduce the propensity for frost action.

Additions and/or Basement Walkouts

Proposed additions to houses or town home units that have lightweight fill and/or an allowable bearing pressure for footing design that is less than 75 kPa, should be required to be reviewed by Paterson to ensure that our geotechnical recommendations are followed. In our opinion, review by the structural engineer of record should also be required for structures that have lightweight fill and/or an allowable bearing pressure that is less than 75 kPa.

In the case of basement walkouts, foundation insulation is generally required, so geotechnical review is recommended to ensure that the existing and new foundations, as well as the foundation drain, will be adequately protected. Structural review is also recommended to ensure that the opening in the foundation wall will be placed in a suitable location and that appropriate measures are taken to ensure adequate support. The design of a wing wall to provide grade separation may also be required.

Note that these recommendations are guidelines, and can be reviewed on a case by case basis, at the cost of the applicant, if the need arises.

7.0 Field Review and Materials Testing Services

A sensitive soils foundation design and field review protocol should be implemented by Minto and their geotechnical and structural engineering consultants on this project to meet City of Ottawa requirements. The sensitive soils inspection protocol is a comprehensive and holistic methodology that ensures that the structural design incorporates the geotechnical design elements. In addition, the construction-related elements, such as field measured shear strengths, underside of footing levels, grade raises, are checked or reviewed during construction and compared with the design assumptions, so modifications can be made where the results of the field observations warrant, and otherwise, as-built conditions can be confirmed to be according to design.

As such, the following field review and materials testing program should be performed by the geotechnical consultant:

❏ Geotechnical review of all grading plans, with consideration of the structures proposed in each development area.

❏ Evaluation of all bearing media, including the putting down of a hand auger and shear vane hole below the footing level, prior to the forming of footings.

❏ Observation of all bearing surfaces prior to the concreting of footings.

❏ Inspection of the placement of lightweight fill (LWF) materials, where required.

❏ Sampling and testing of the concrete and granular fill materials used.

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3.0 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling and follow-up 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 our recommendations could be issued, upon demand, based on the completion of a satisfactory materials testing and field review observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We have reviewed the present grading plans, and provided comments in this report.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and 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, we request that we be notified immediately in order to permit reassessment of our recommendations.

The preliminary 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 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 Minto Communities Inc. 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.

AJ Tovell

LICENSED PROFESSIONAL ENGINEER
Apr. 02-18
A. J. TOVELL
46812509
PROVINCE OF ONTARIO

Andrew J. Tovell, P.Eng.

Report Distribution:

☐ Minto Communities Inc. (1 copy)
☐ WSP (4 copies)
☐ Atrel Engineering Ltd. (1 copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TEST RESULTS

ATTERBERG LIMITS RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

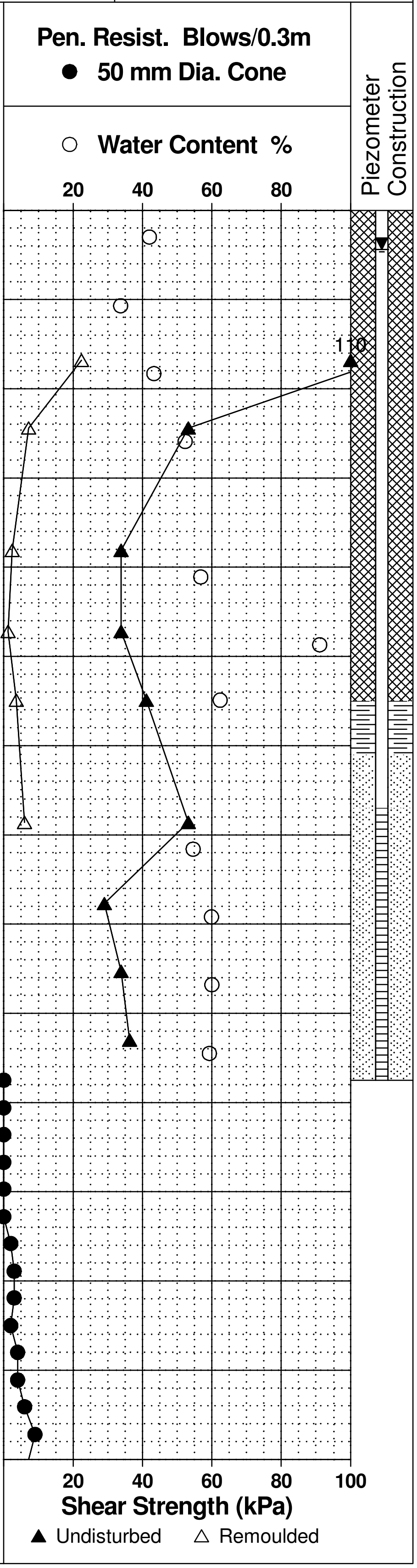
DATE March 1, 2018

FILE NO. PG4444

HOLE NO. BH 1

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty clay, some organics 0.70	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	54	7
	SS	3	83	P
2.70	SS	4	92	P
Firm, grey **SILTY CLAY**	TW	5	92	
	SS	6	100	P
	SS	7	100	P
	SS	8	100	P
	TW	9	94	
	SS	10	100	P
	SS	11	100	P
	SS	12	100	P
9.75	SS	13	100	P
Dynamic Cone Penetration Test commenced at 9.75m depth.				
Inferred firm to stiff **SILTY CLAY**				

DEPTH (m)	ELEV. (m)
0	89.18
1	88.18
2	87.18
3	86.18
4	85.18
5	84.18
6	83.18
7	82.18
8	81.18
9	80.18
10	79.18
11	78.18
12	77.18
13	76.18
14	75.18



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

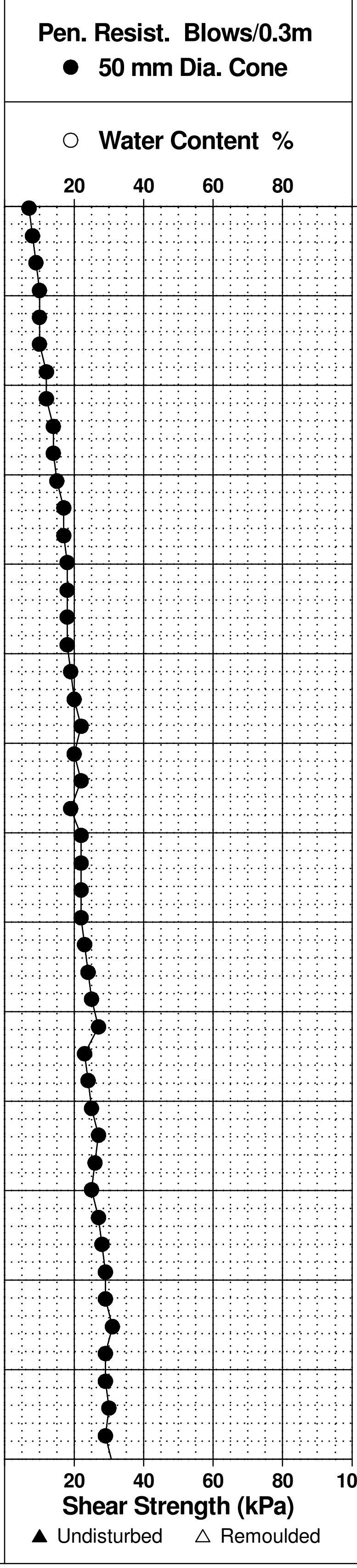
BORINGS BY CME 55 Power Auger

DATE March 1, 2018

FILE NO. PG4444

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								
	▨					14	75.18	
	▨					15	74.18	
	▨					16	73.18	
	▨					17	72.18	
	▨					18	71.18	
	▨					19	70.18	
Inferred stiff to very stiff **SILTY CLAY**	▨					20	69.18	
	▨					21	68.18	
	▨					22	67.18	
	▨					23	66.18	
	▨					24	65.18	
	▨					25	64.18	
	▨					26	63.18	
	▨					27	62.18	
						28	61.18	



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 1, 2018

FILE NO. PG4444

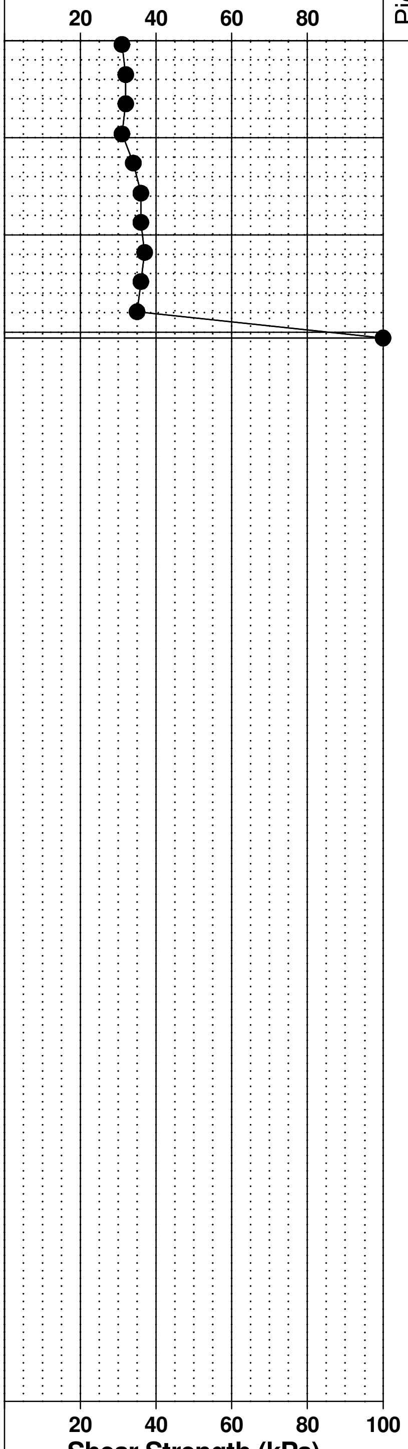
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Inferred very stiff **SILTY CLAY** and/or dense **GLACIAL TILL**	▨					28	61.18
	▨					29	60.18
	▨					30	59.18
31.06	▨					31	58.18
End of Borehole Practical DCPT refusal at 31.06m depth. (GWL @ 0.44m - March 28, 2018)							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. **PG4444**

REMARKS

HOLE NO. **BH 2**

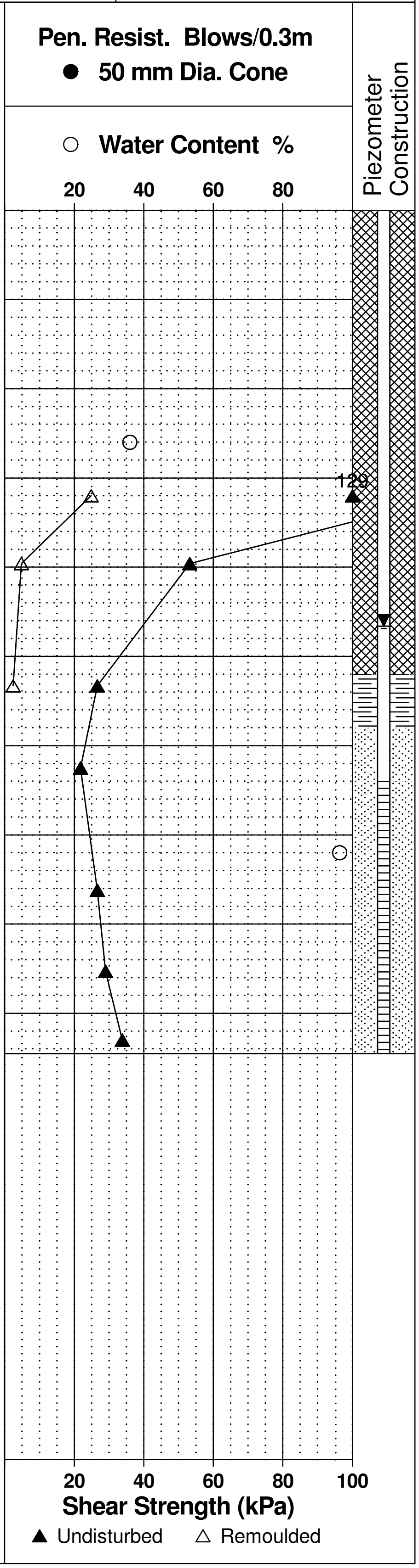
BORINGS BY CME 55 Power Auger

DATE March 1, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.50
FILL: Brown silty clay, trace sand, organics and topsoil	AU	1				
	SS	2	33	5	1	89.50
	SS	3	75	11	2	88.50
2.60	SS	4	71	9		
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	5	100	P	3	87.50
4.20					4	86.50
Firm, grey **SILTY CLAY**	TW	6	22		5	85.50
					6	84.50
	TW	7	98		7	83.50
					8	82.50
					9	81.50
9.45						

End of Borehole

(GWL @ 4.66m - March 28, 2018)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

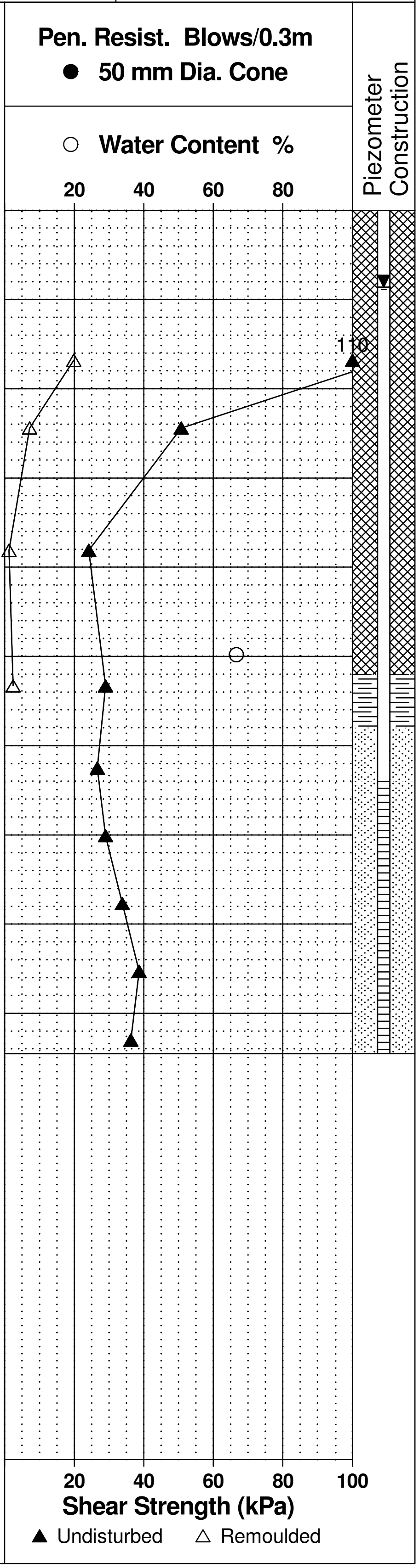
DATE February 28, 2018

FILE NO. PG4444

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay, some organics 0.60		AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	75	7
		SS	3	92	P
2.60		SS	4	100	P
Frim, grey **SILTY CLAY**		TW	5	88	
		TW	6	96	
9.45					
End of Borehole (GWL @ 0.86m - March 28, 2018)					

DEPTH (m)	ELEV. (m)
0	89.08
1	88.08
2	87.08
3	86.08
4	85.08
5	84.08
6	83.08
7	82.08
8	81.08
9	80.08



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. **PG4444**

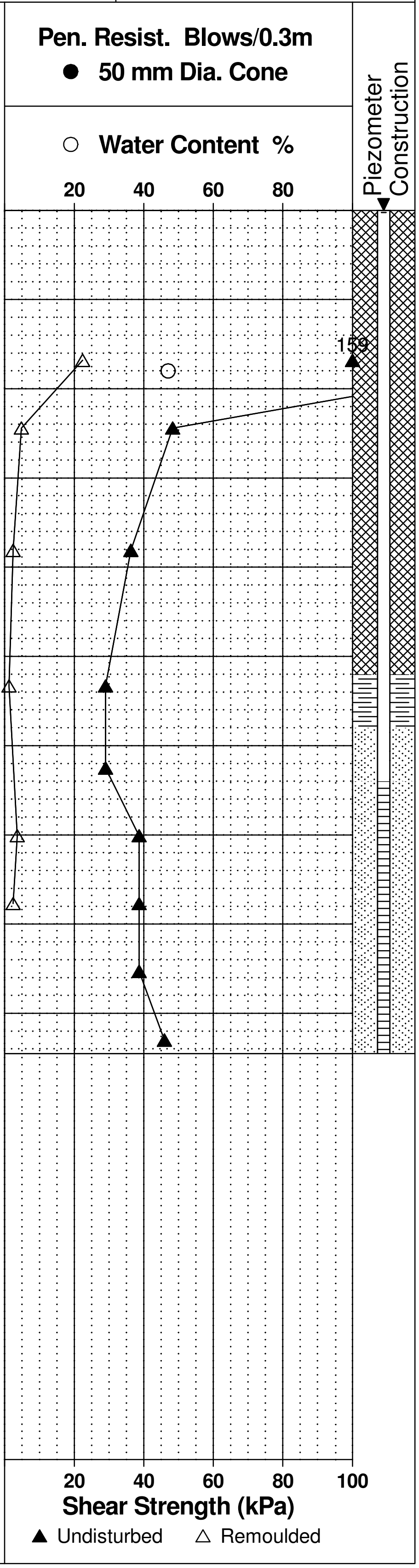
REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger **DATE** March 1, 2018

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay, some organics	0.38	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	75	6
	2.60	SS	3	100	P
Firm, grey **SILTY CLAY**		TW	4	85	
		TW	5	100	
	9.45	SS	6	100	P
End of Borehole (GWL @ ground surface - March 28, 2018)					

DEPTH (m)	ELEV. (m)
0	88.97
1	87.97
2	86.97
3	85.97
4	84.97
5	83.97
6	82.97
7	81.97
8	80.97
9	79.97



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG4444

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

DATE February 28, 2018

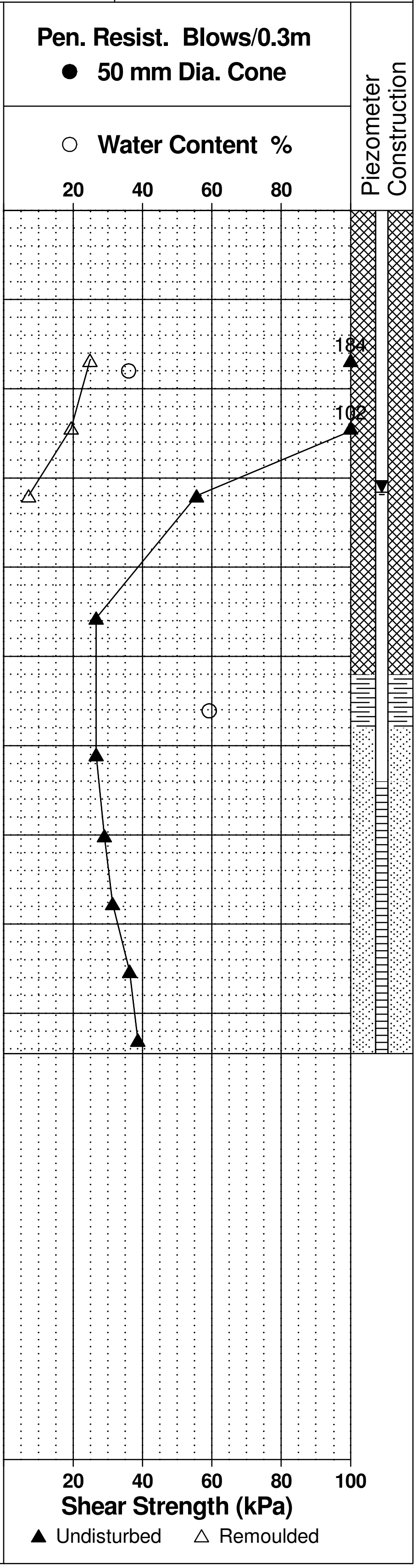
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty clay, some organics	0.76
TOPSOIL	0.96
Very stiff to stiff, brown **SILTY CLAY,** trace sand	3.40
Firm, grey **SILTY CLAY**	9.45

Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed and then driven to practical refusal at 28.24m depth.

(GWL @ 3.16m - March 28, 2018)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	9
SS	3	83	P
SS	4	100	P
TW	5	94	
TW	6	98	

DEPTH (m)	ELEV. (m)
0	89.62
1	88.62
2	87.62
3	86.62
4	85.62
5	84.62
6	83.62
7	82.62
8	81.62
9	80.62



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. **PG4444**

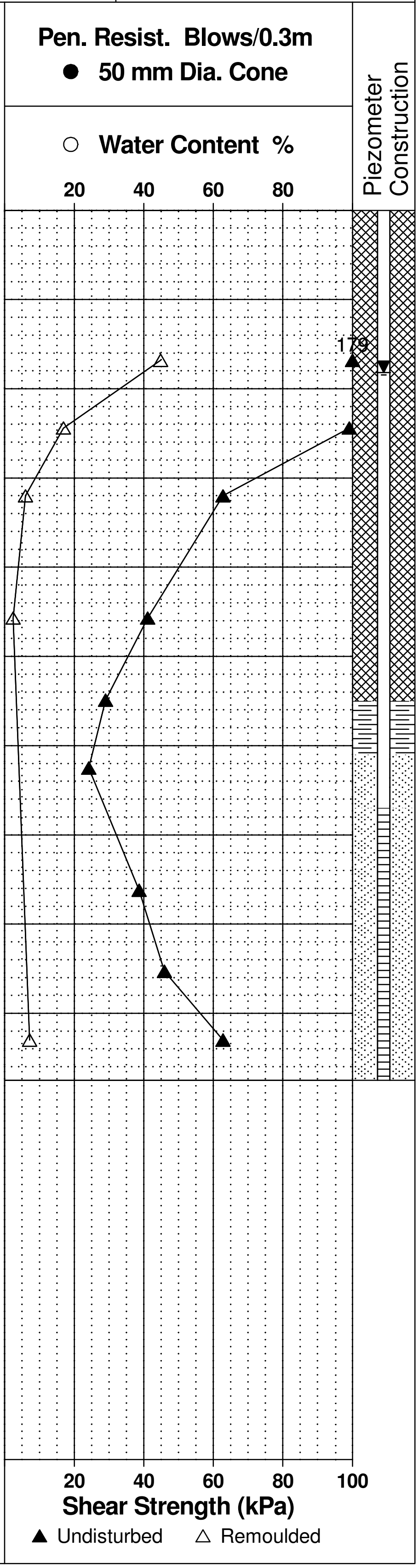
REMARKS

HOLE NO. **BH 6**

BORINGS BY CME 55 Power Auger

DATE February 28, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	89.38
FILL: Brown silty clay, some organics	AU	1				
1.05	SS	2	46	8	1	88.38
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	3	92	P	2	87.38
					3	86.38
4.00	TW	4	92		4	85.38
					5	84.38
					6	83.38
Firm to stiff, grey **SILTY CLAY**	TW	5	98		7	82.38
					8	81.38
					9	80.38
9.75						
End of Borehole						
(GWL @ 1.82m - March 28, 2018)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 28, 2018

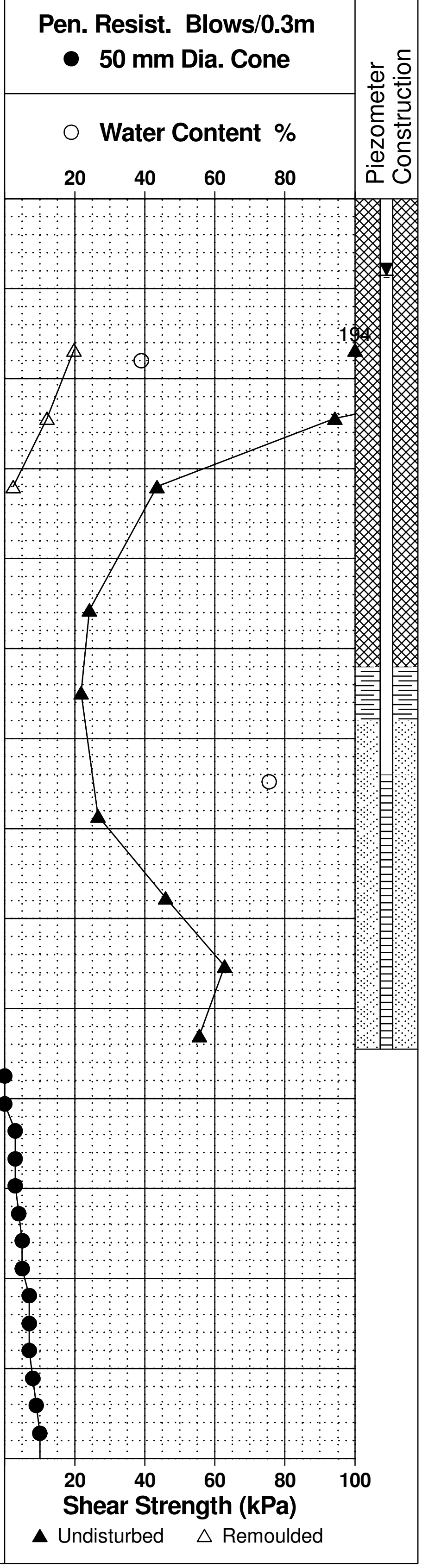
FILE NO. PG4444

HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty clay, some organics	0.86
Very stiff to stiff, brown **SILTY CLAY,** trace sand	3.10
Firm to stiff, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth.	
Inferred stiff **SILTY CLAY**	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	6
SS	3	100	P
TW	4	90	
SS	5	100	P
TW	6	96	

DEPTH (m)	ELEV. (m)
0	88.92
1	87.92
2	86.92
3	85.92
4	84.92
5	83.92
6	82.92
7	81.92
8	80.92
9	79.92
10	78.92
11	77.92
12	76.92
13	75.92
14	74.92



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

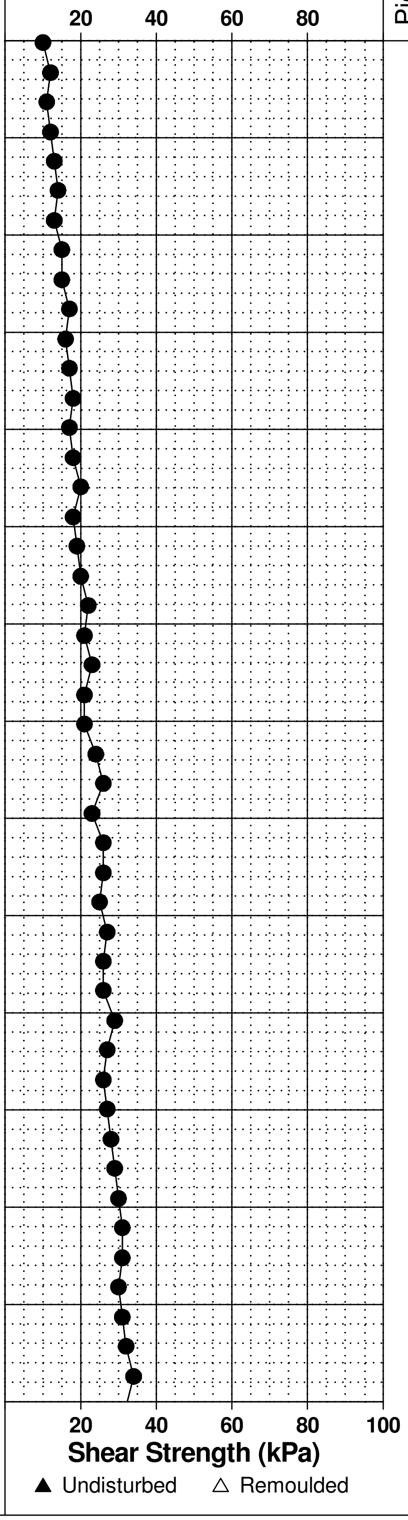
BORINGS BY CME 55 Power Auger

DATE February 28, 2018

FILE NO. PG4444

HOLE NO. BH 7

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									
Inferred stiff to very stiff **SILTY CLAY**	▨					14	74.92		
						15	73.92		
						16	72.92		
						17	71.92		
						18	70.92		
						19	69.92		
						20	68.92		
						21	67.92		
						22	66.92		
						23	65.92		
						24	64.92		
						25	63.92		
						26	62.92		
						27	61.92		
						28	60.92		



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. **PG4444**

REMARKS

HOLE NO. **BH 7**

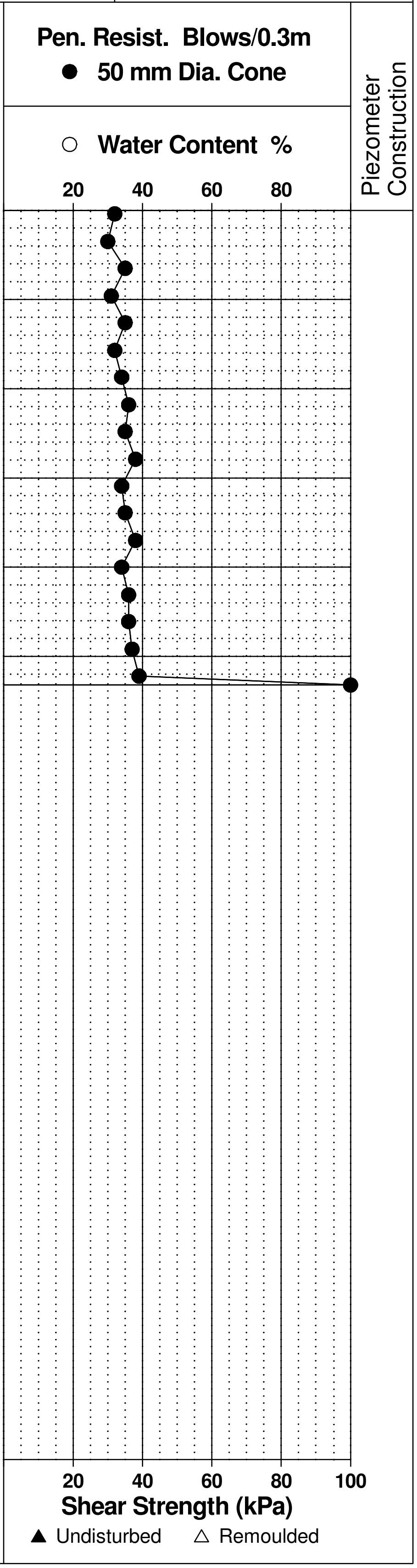
BORINGS BY CME 55 Power Auger **DATE** February 28, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						28	60.92
	▨					29	59.92
Inferred very stiff **SILTY CLAY** and/or dense **GLACIAL TILL**	▨					30	58.92
	▨					31	57.92
	▨					32	56.92
33.32	▨					33	55.92

End of Borehole

Practical DCPT refusal at 33.32m depth

(GWL @ 0.85m - March 28, 2018)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG4444

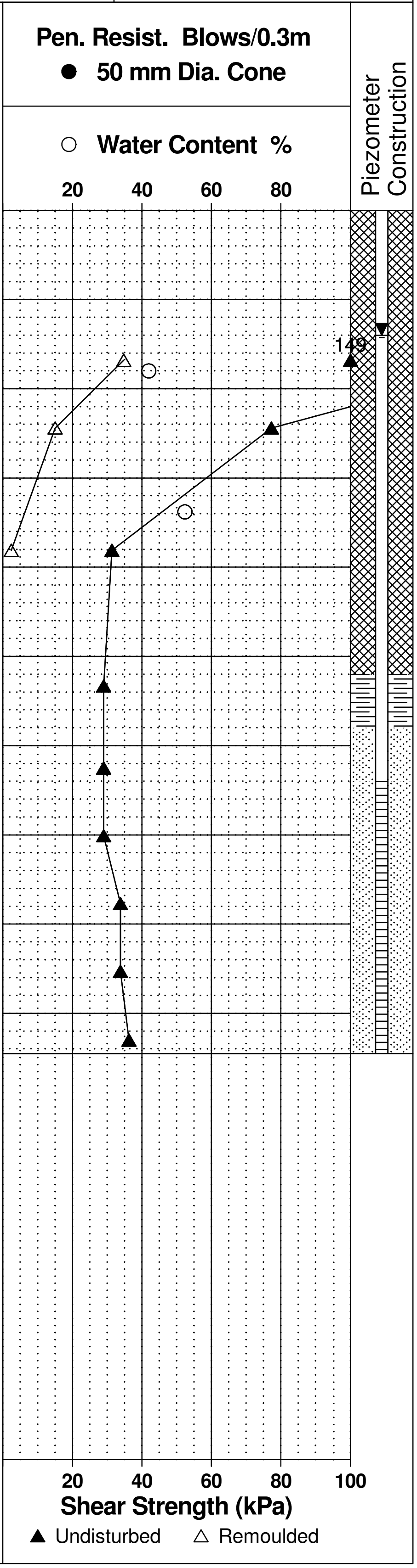
REMARKS

HOLE NO. BH 8

BORINGS BY CME 55 Power Auger

DATE February 26, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.08
FILL: Brown silty clay, some organics		AU	1				
0.60							
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	54	7	1	88.08
		SS	3	100	P	2	87.08
						3	86.08
3.20							
Firm, grey **SILTY CLAY**		TW	4	90		4	85.08
		TW	5	100		5	84.08
						6	83.08
		SS	6	100	P	7	82.08
						8	81.08
						9	80.08
9.45							
End of Borehole (GWL @ 1.40m - March 28, 2018)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. **PG4444**

REMARKS

HOLE NO. **BH 9**

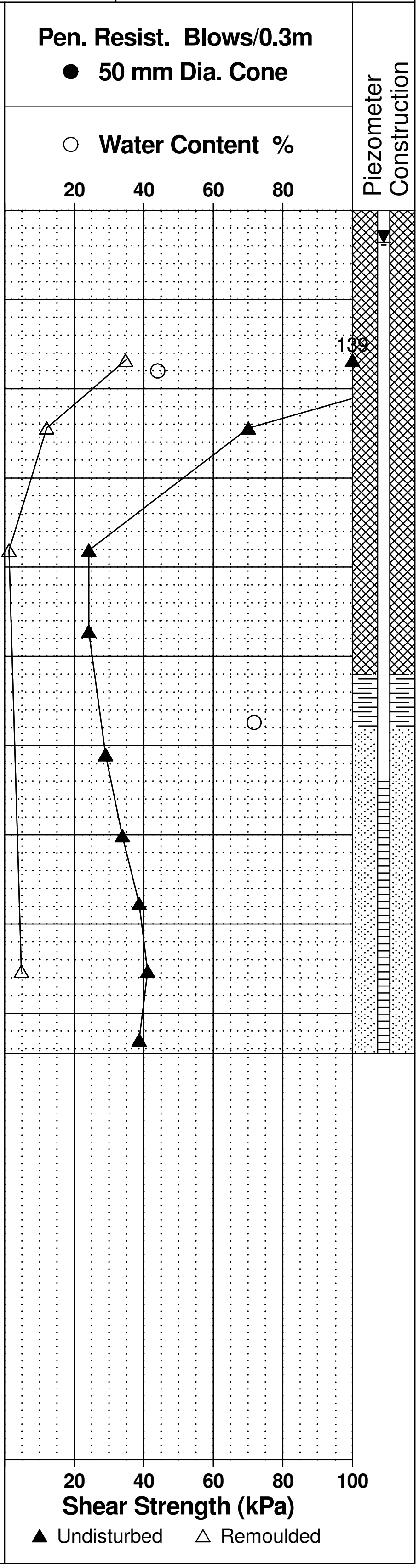
BORINGS BY CME 55 Power Auger

DATE February 27, 2018

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty clay, some organics	0.50
Very stiff to stiff, brown **SILTY CLAY,** trace sand	3.00
Soft to firm, grey **SILTY CLAY**	9.45
End of Borehole (GWL @ 0.36m - March 28, 2018)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	5
SS	3	83	P
TW	4	88	
TW	5	98	
SS	6	100	P

DEPTH (m)	ELEV. (m)
0	89.18
1	88.18
2	87.18
3	86.18
4	85.18
5	84.18
6	83.18
7	82.18
8	81.18
9	80.18



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG4444

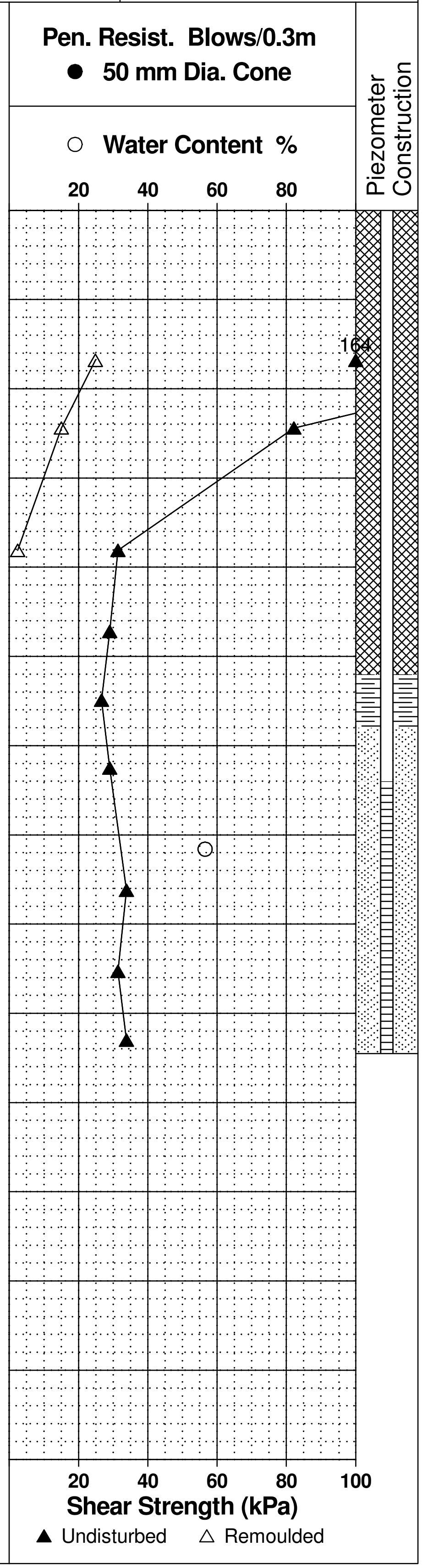
REMARKS

HOLE NO. BH10

BORINGS BY CME 55 Power Auger **DATE** February 27, 2018

SOIL DESCRIPTION	Depth to base	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay, some organics	0.69	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	50	7
		SS	3	3	P
	3.30				
Firm, grey **SILTY CLAY**		TW	4	90	
		SS	5	100	P
		TW	6	90	
	9.45				
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 30.5m depth. Cone pushed through inferred firm to stiff **SILTY CLAY**					

DEPTH (m)	ELEV. (m)
0	89.00
1	88.00
2	87.00
3	86.00
4	85.00
5	84.00
6	83.00
7	82.00
8	81.00
9	80.00
10	79.00
11	78.00
12	77.00
13	76.00
14	75.00



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

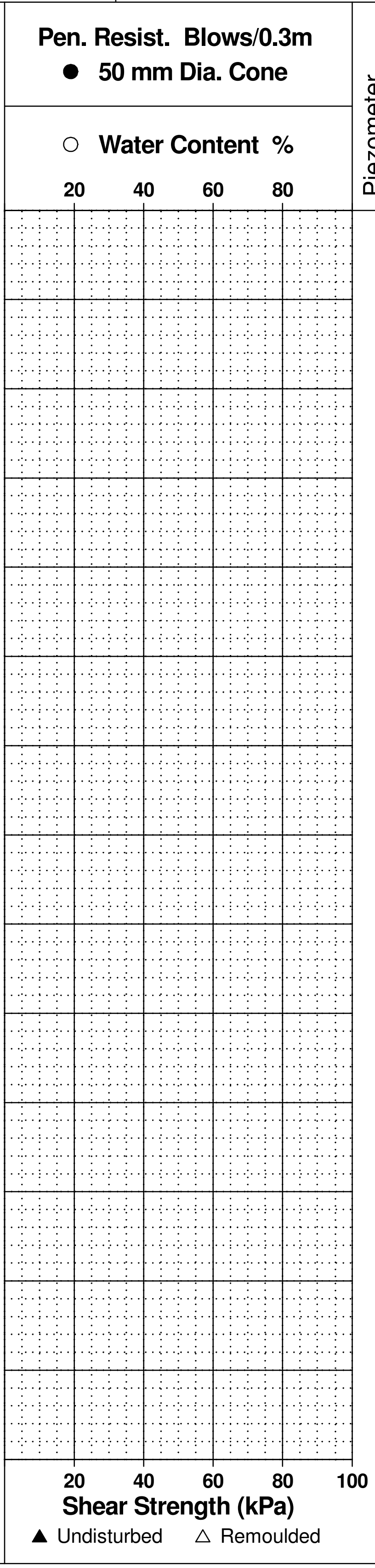
BORINGS BY CME 55 Power Auger

DATE February 27, 2018

FILE NO. PG4444

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
	▨					14	75.00	
	▨					15	74.00	
	▨					16	73.00	
	▨					17	72.00	
	▨					18	71.00	
Cone pushed through inferred stiff **SILTY CLAY**	▨					19	70.00	
	▨					20	69.00	
	▨					21	68.00	
	▨					22	67.00	
	▨					23	66.00	
	▨					24	65.00	
	▨					25	64.00	
	▨					26	63.00	
	▨					27	62.00	
						28	61.00	

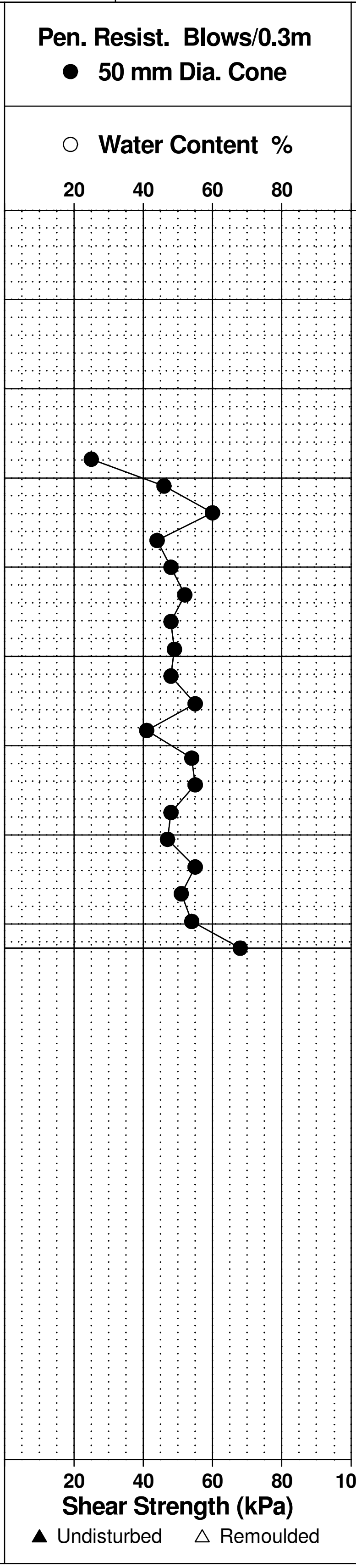


SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.
REMARKS
BORINGS BY CME 55 Power Auger
DATE February 27, 2018
FILE NO. PG4444
HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						28	61.00
	▨					29	60.00
	▨					30	59.00
Inferred very stiff **SILTY CLAY** and/or dense **GLACIAL TILL**	▨					31	58.00
	▨					32	57.00
	▨					33	56.00
	▨					34	55.00
	▨					35	54.00
36.27	▨					36	53.00
End of Borehole (BH dry - March 28, 2018)							



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 26, 2018

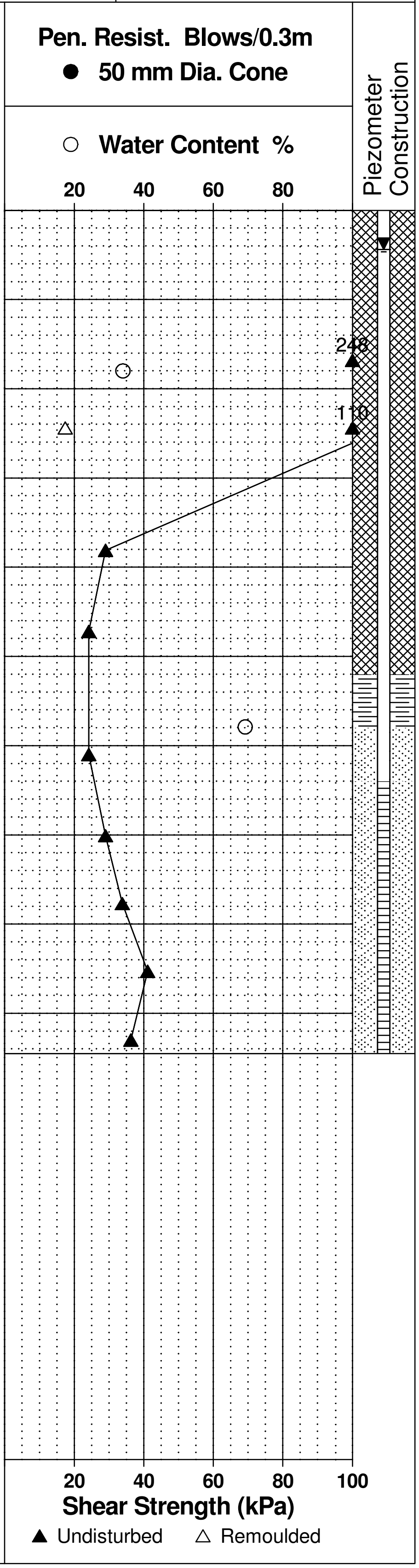
FILE NO. PG4444

HOLE NO. BH11

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty clay, some organics (to 0.84)	AU	1		
Hard to stiff, brown **SILTY CLAY,** trace sand (0.84 to 3.40)	SS	2	38	6
	SS	3	75	P
	TW	4	88	
Firm, grey **SILTY CLAY** (3.40 to 9.45)	TW	5	100	
	SS	6	100	P
End of Borehole (9.45)				

(GWL @ 0.44m - March 28, 2018)

DEPTH (m)	ELEV. (m)
0	89.18
1	88.18
2	87.18
3	86.18
4	85.18
5	84.18
6	83.18
7	82.18
8	81.18
9	80.18



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

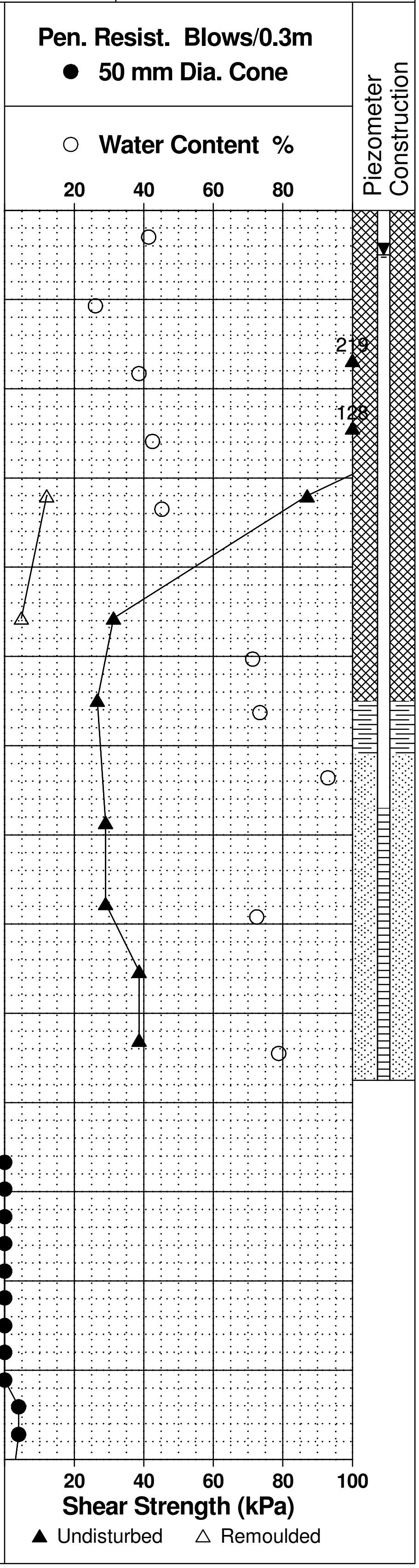
BORINGS BY CME 55 Power Auger

DATE February 26, 2018

FILE NO. PG4444

HOLE NO. BH12

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	89.77
FILL: Brown silty clay, some organics — 0.80	AU	1				
TOPSOIL — 0.95	SS	2	58	7	1	88.77
Hard to stiff, brown **SILTY CLAY,** trace sand	SS	3	62	P	2	87.77
	SS	4	100	P	3	86.77
	SS	5	100	P		
— 4.10	TW	6	79		4	85.77
Stiff to firm, grey **SILTY CLAY**	SS	7	100	P	5	84.77
	SS	8	100	P	6	83.77
	TW	9	100		7	82.77
	SS	10	100	P	8	81.77
— 9.75	SS	11	100	P	9	80.77
Dynamic Cone Penetration Test commenced at 9.75m depth. Cone pushed to 10.4m depth.					10	79.77
					11	78.77
					12	77.77
Inferred firm **SILTY CLAY**					13	76.77
					14	75.77



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Aquaview Stage 2 - 10th Line Road & Lakepointe Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG4444

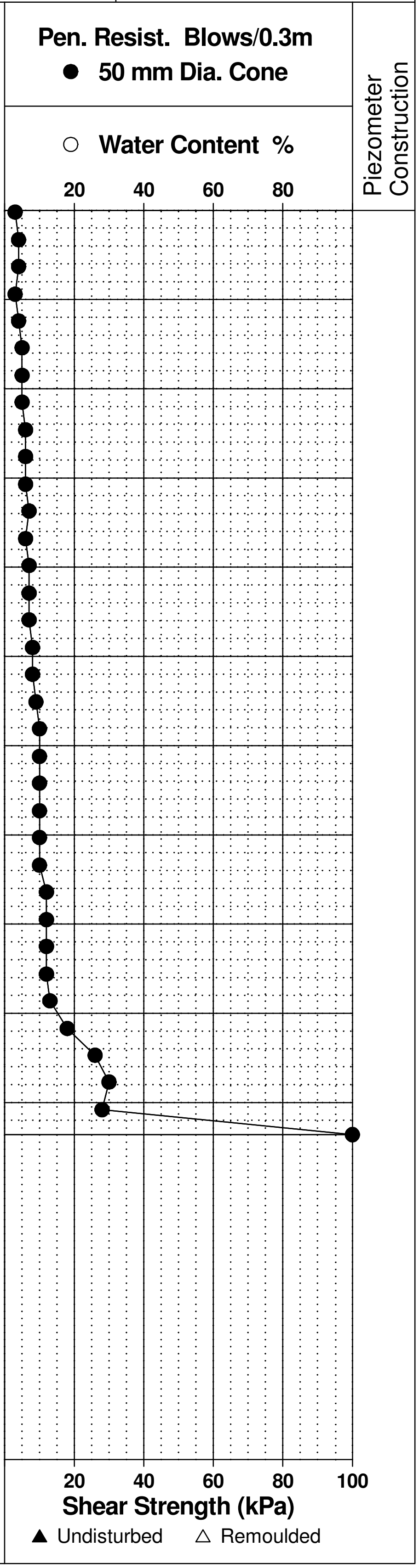
REMARKS

HOLE NO. BH12

BORINGS BY CME 55 Power Auger

DATE February 26, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						14	75.77
						15	74.77
						16	73.77
						17	72.77
Inferred stiff to very stiff **SILTY CLAY**						18	71.77
						19	70.77
						20	69.77
						21	68.77
						22	67.77
						23	66.77
						24	65.77
24.36							
End of Borehole Practical DCPT refusal at 24.36m depth (GWL @ 0.50m - March 28, 2018)							



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)

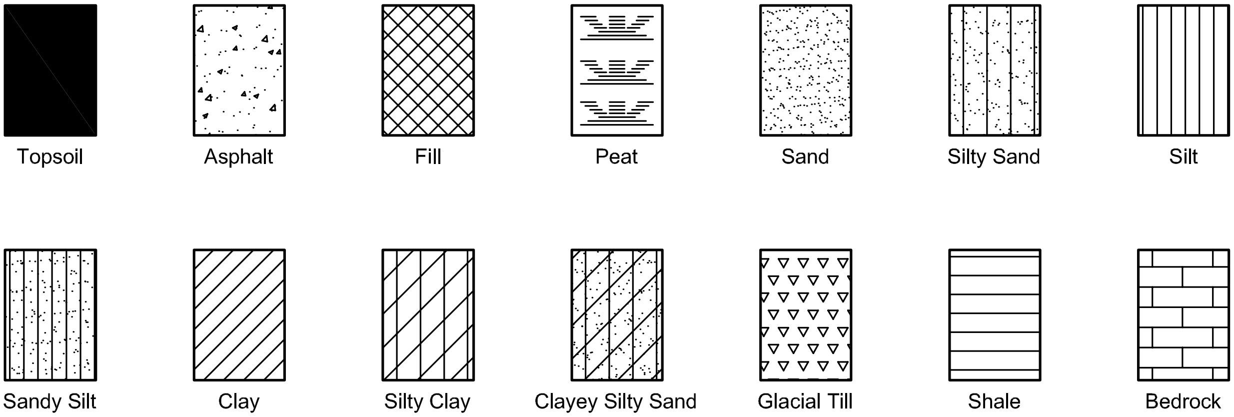
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

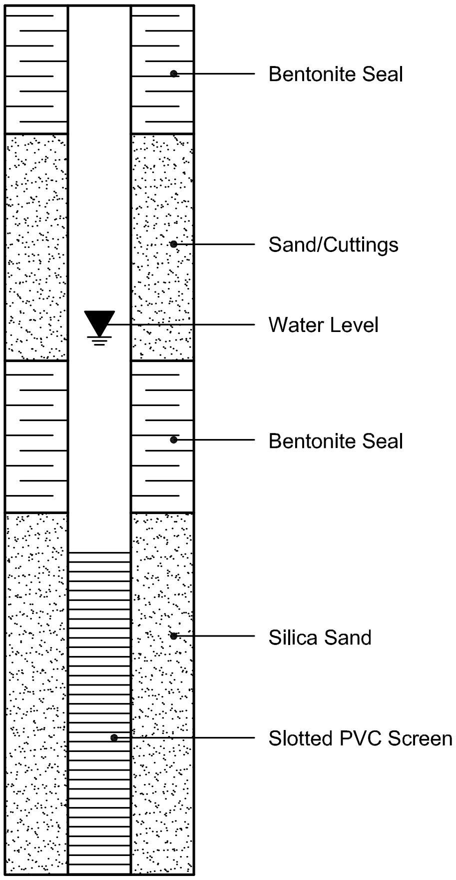
SYMBOLS AND TERMS (continued)

STRATA PLOT

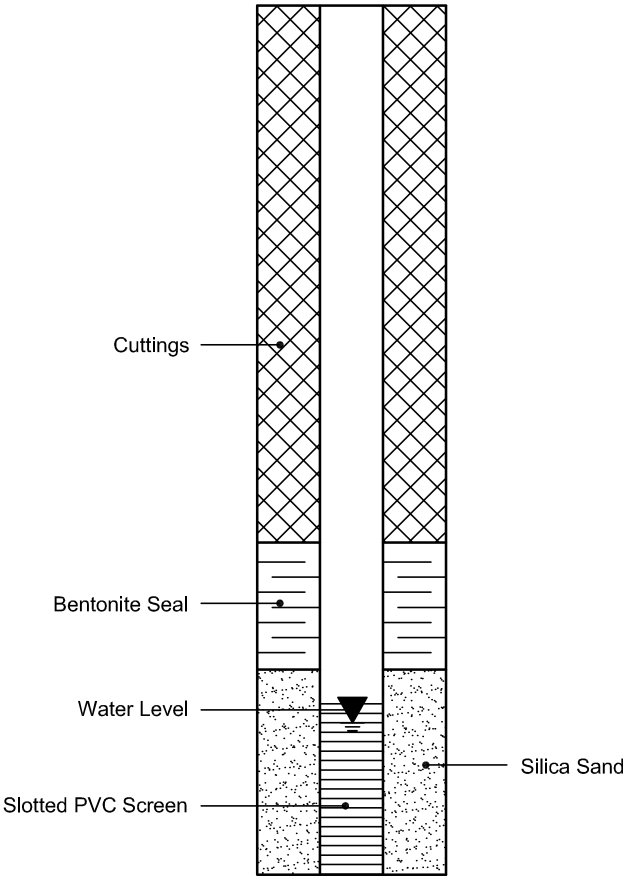


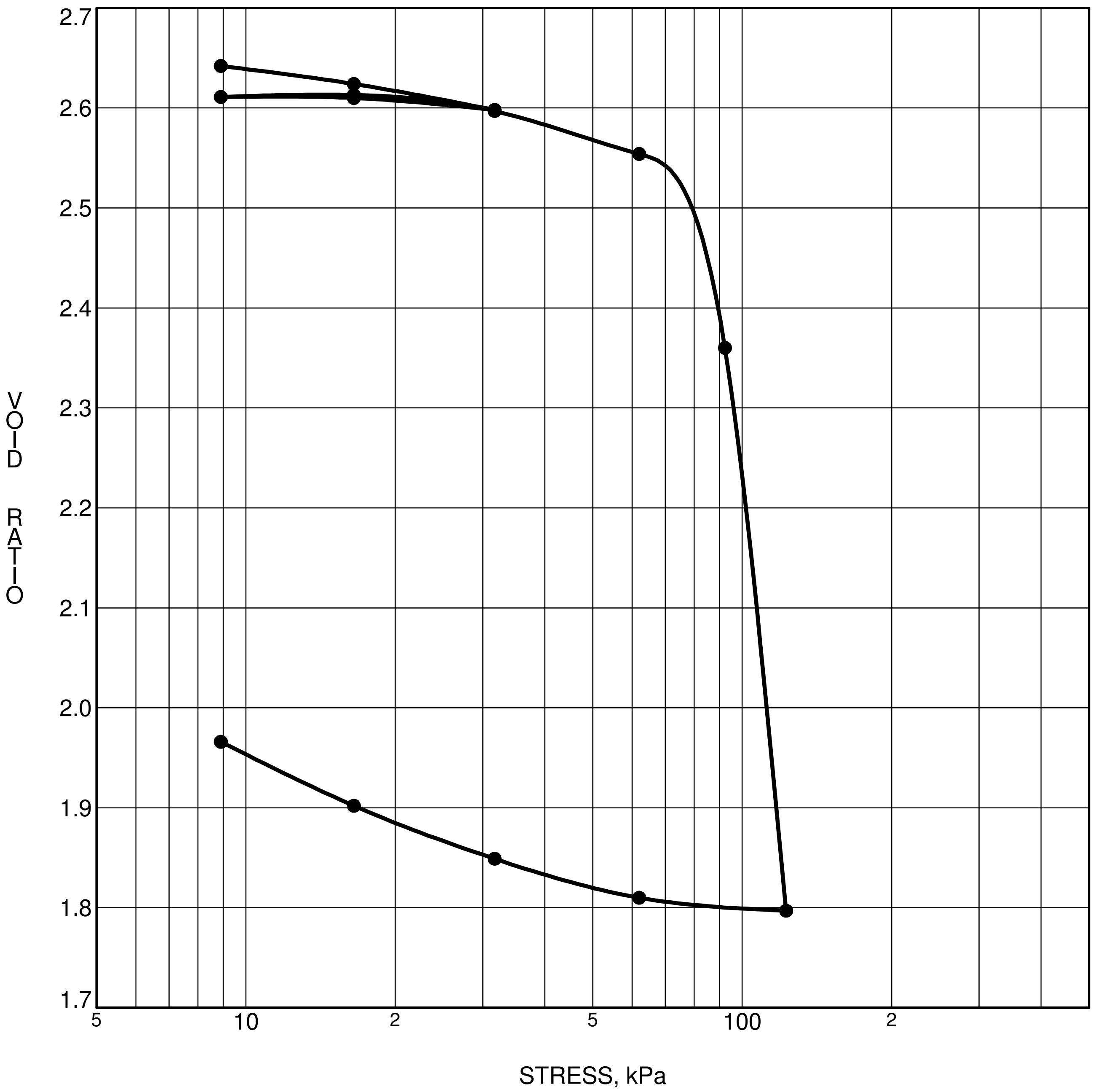
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2**	p'_o	**62** kPa	Ccr	**0.047**
Sample No.	**TW 7**	p'_c	**85** kPa	Cc	**4.587**
Sample Depth	**7.20** m	OC Ratio	**1.4**	Wo	**96.3** %
Sample Elev.	**83.30** m	Void Ratio	**2.649**	Unit Wt.	**14.5** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG4444**

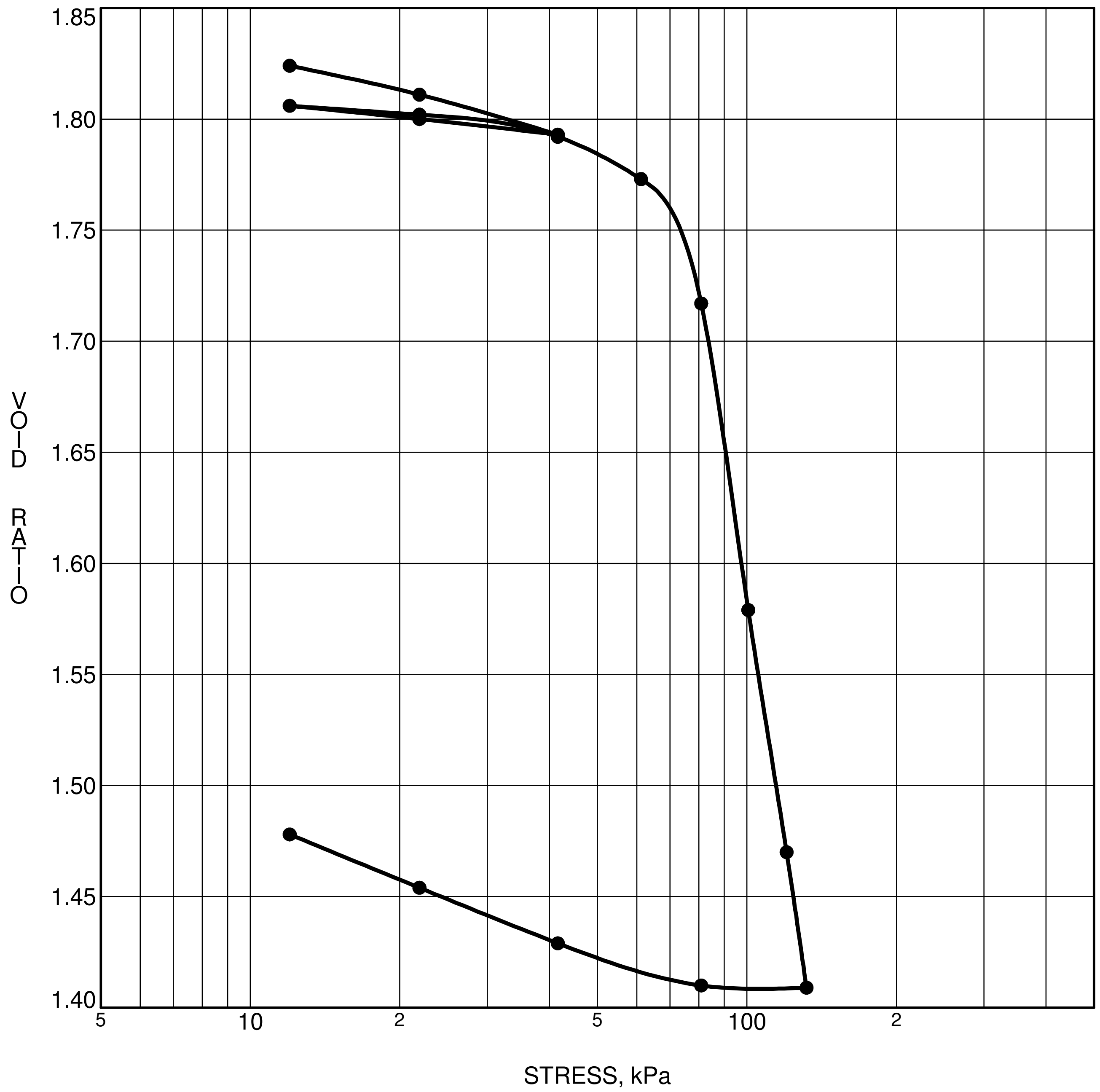
PROJECT **Geotechnical Investigation - Aquaview Stage 2 -**
10th Line Road & Lakepointe Drive

DATE **27/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3**	p'o	**58** kPa	Ccr	**0.028**
Sample No.	**TW 6**	p'c	**76** kPa	Cc	**1.476**
Sample Depth	**4.98** m	OC Ratio	**1.3**	Wo	**66.6** %
Sample Elev.	**84.10** m	Void Ratio	**1.833**	Unit Wt.	**15.9** kN/m ^{3}

CLIENT **Minto Communities Inc.**

PROJECT **Geotechnical Investigation - Aquaview Stage 2 - 10th Line Road & Lakepointe Drive**

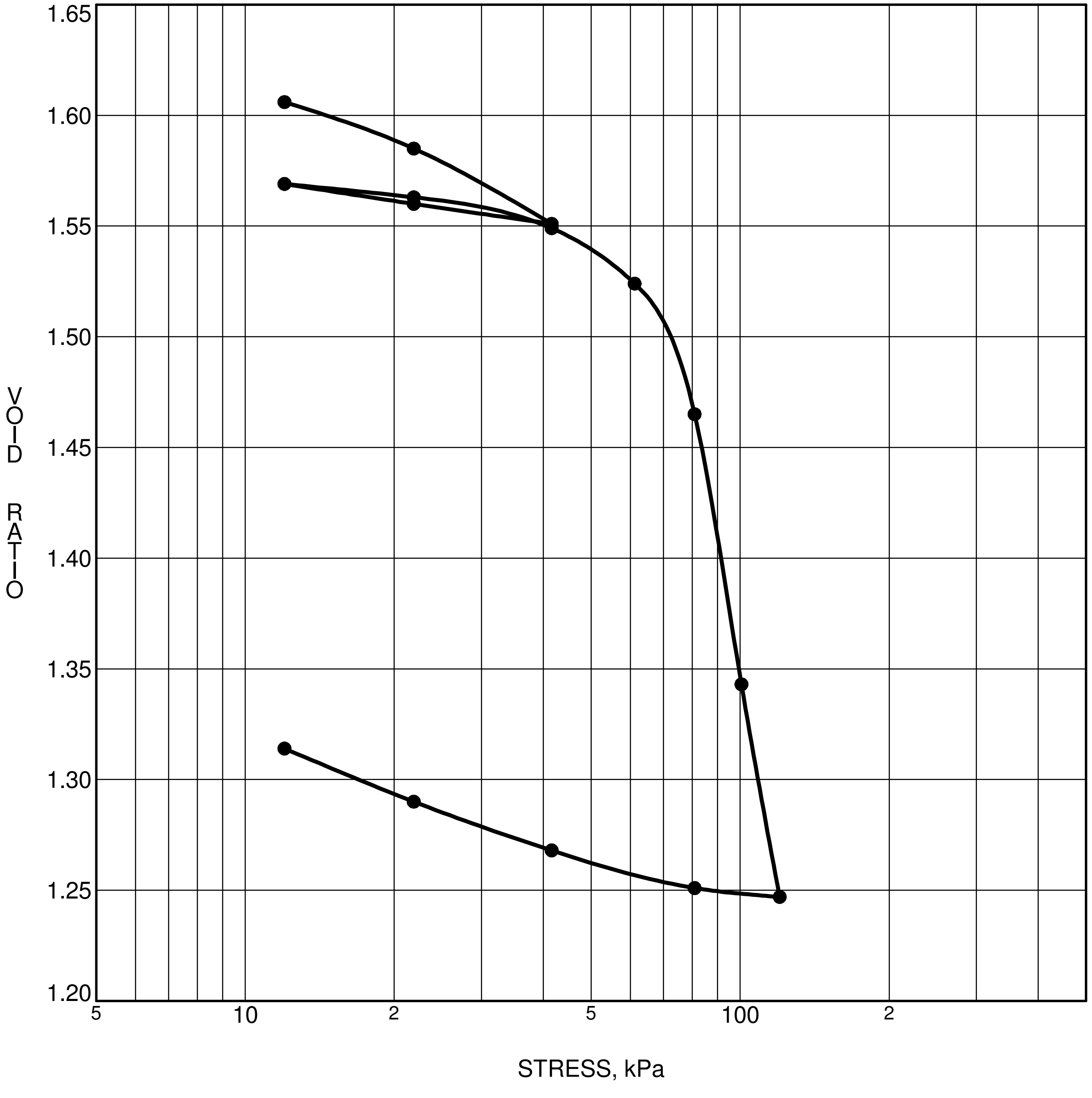
FILE NO. **PG4444**

DATE **9/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 5**	p'o	**59** kPa	Ccr	**0.040**
Sample No.	**TW 6**	p'c	**75** kPa	Cc	**1.260**
Sample Depth	**5.61** m	OC Ratio	**1.3**	Wo	**59.2** %
Sample Elev.	**84.01** m	Void Ratio	**1.629**	Unit Wt.	**16.3** kN/m ^{3}

CLIENT **Minto Communities Inc.**

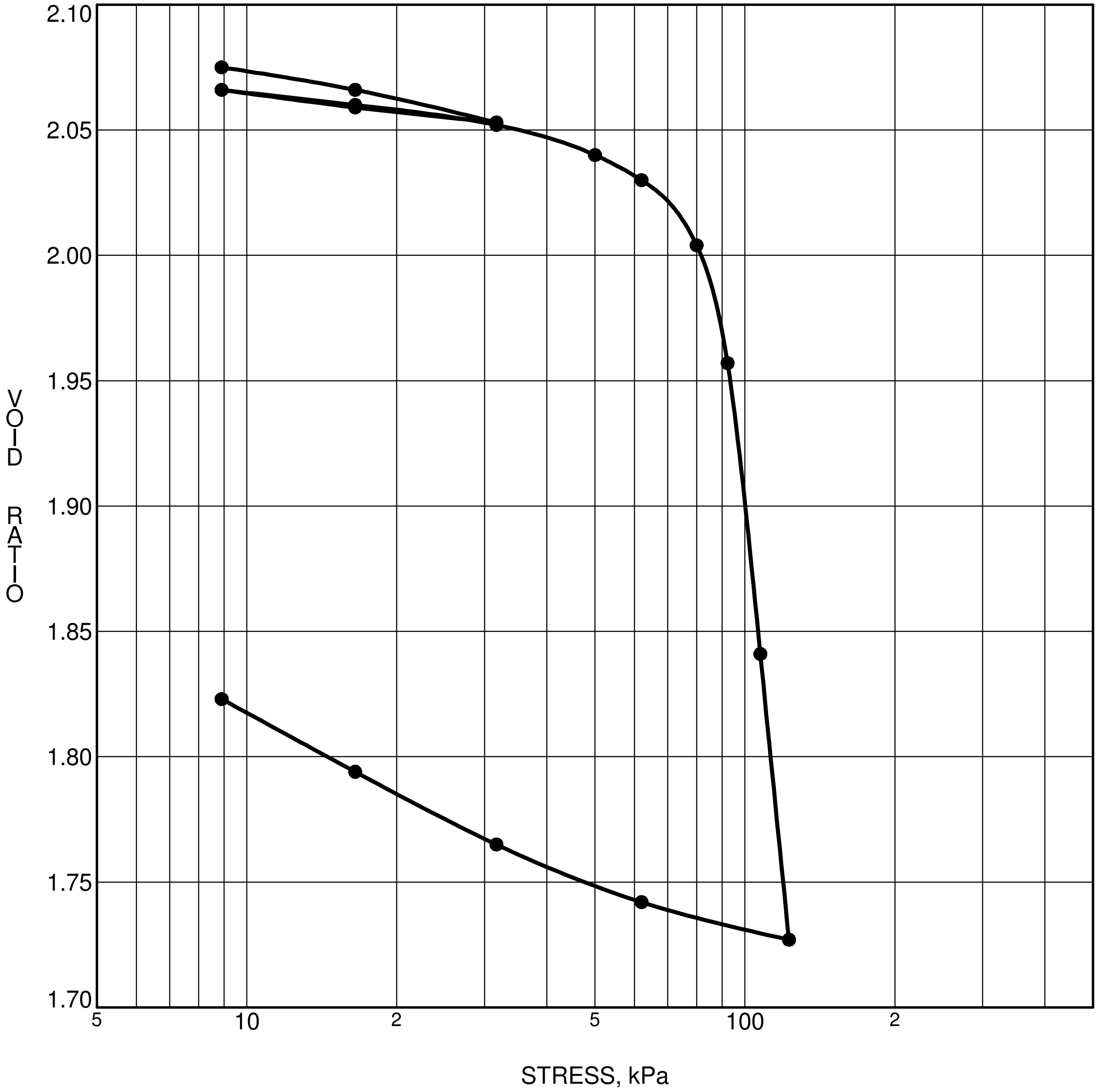
FILE NO. **PG4444**

PROJECT **Geotechnical Investigation - Aquaview Stage 2 -**
10th Line Road & Lakepointe Drive

DATE **23/03/2018**

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

CONSOLIDATION TEST



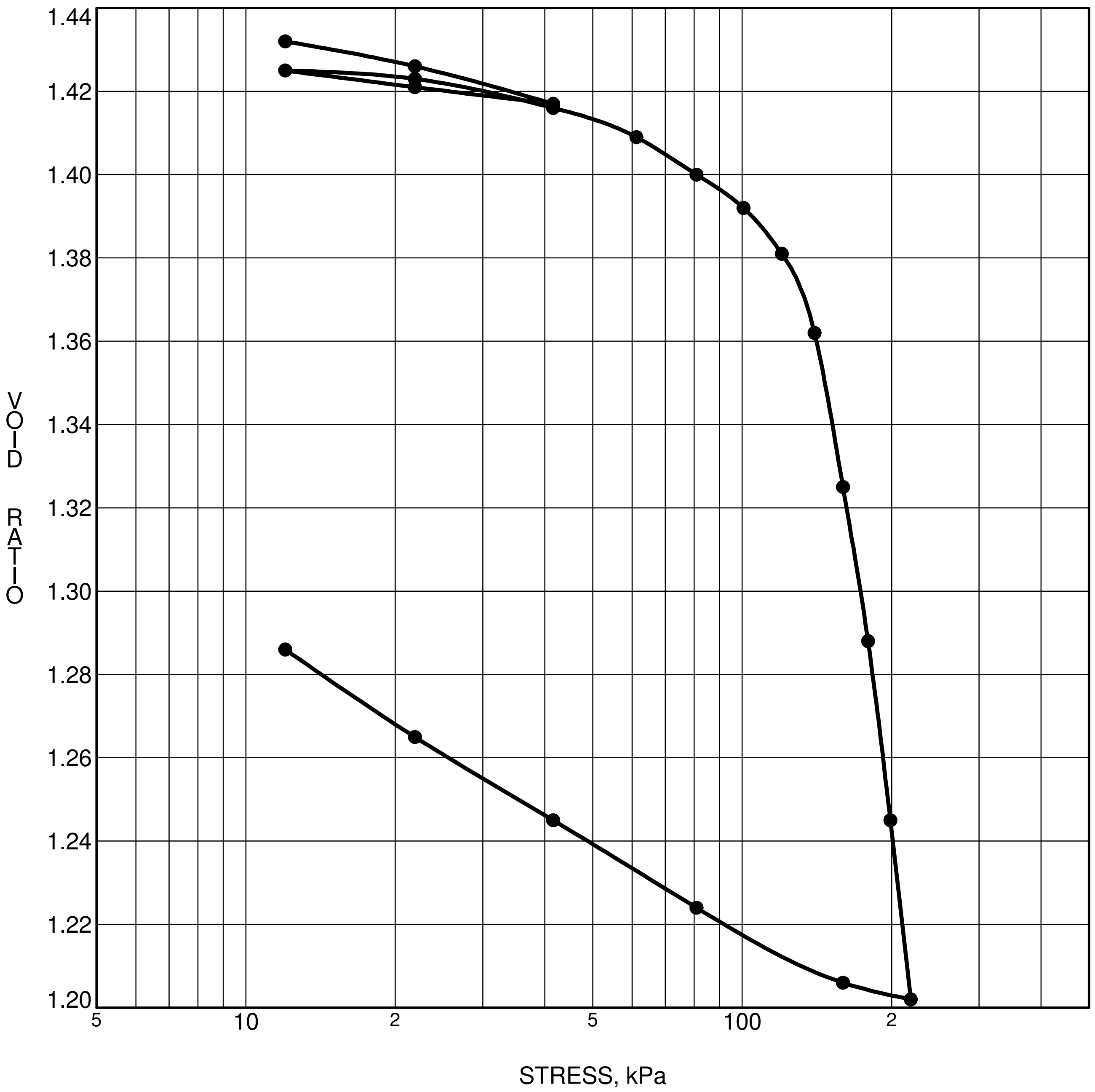
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 7**	p'_o	**67** kPa	Ccr	**0.025**
Sample No.	**TW 6**	p'_c	**88** kPa	Cc	**2.020**
Sample Depth	**6.48** m	OC Ratio	**1.3**	Wo	**75.5** %
Sample Elev.	**82.44** m	Void Ratio	**2.076**	Unit Wt.	**15.4** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG4444**
PROJECT	**Geotechnical Investigation - Aquaview Stage 2 -** **10th Line Road & Lakepointe Drive**	DATE	**16/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST

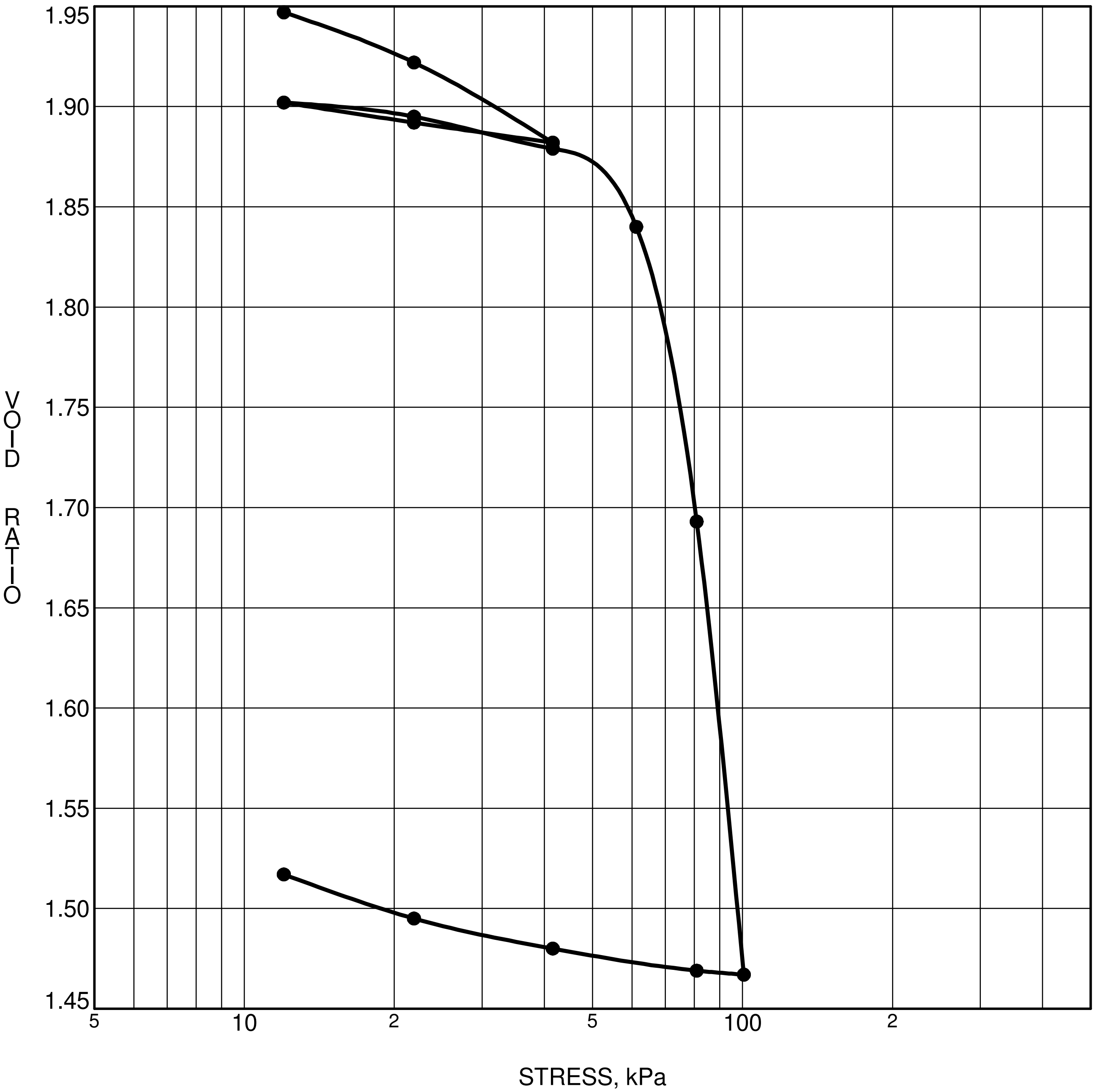


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 8**	p'$_o$	**48** kPa	Ccr	**0.017**
Sample No.	**TW 4**	p'$_c$	**148** kPa	Cc	**1.018**
Sample Depth	**3.38** m	OC Ratio	**3.1**	Wo	**52.4** %
Sample Elev.	**85.70** m	Void Ratio	**1.441**	Unit Wt.	**16.8** kN/m ^{3}

CLIENT **Minto Communities Inc.**
FILE NO. **PG4444**
PROJECT **Geotechnical Investigation - Aquaview Stage 2 - 10th Line Road & Lakepointe Drive**
DATE **16/03/2018**

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

CONSOLIDATION TEST



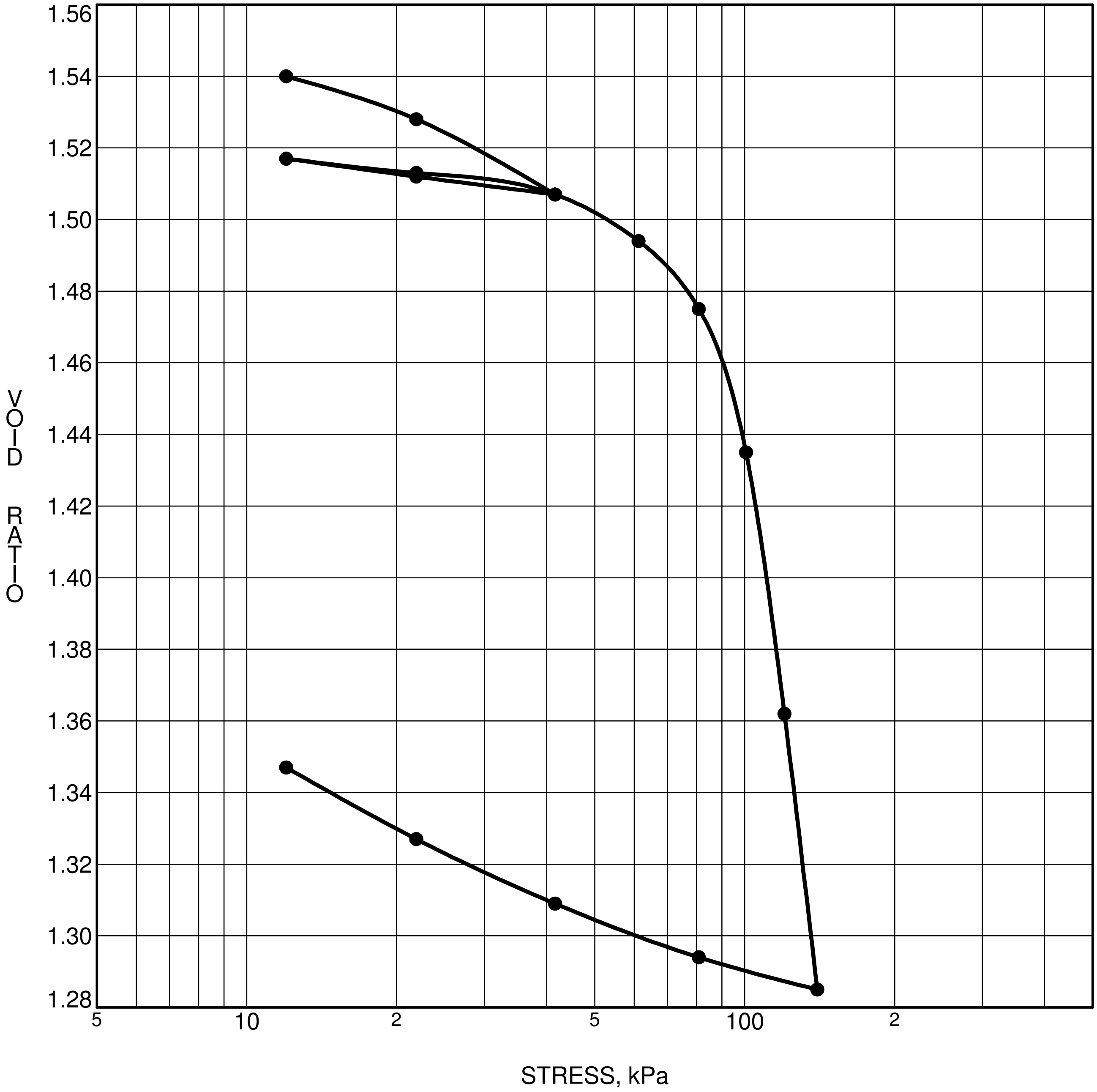
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 9**	p'o	**63** kPa	Ccr	**0.044**
Sample No.	**TW 5**	p'c	**70** kPa	Cc	**2.459**
Sample Depth	**5.74** m	OC Ratio	**1.1**	Wo	**71.7** %
Sample Elev.	**83.44** m	Void Ratio	**1.972**	Unit Wt.	**15.6** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG4444**
PROJECT	**Geotechnical Investigation - Aquaview Stage 2 - 10th Line Road & Lakepointe Drive**	DATE	**9/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



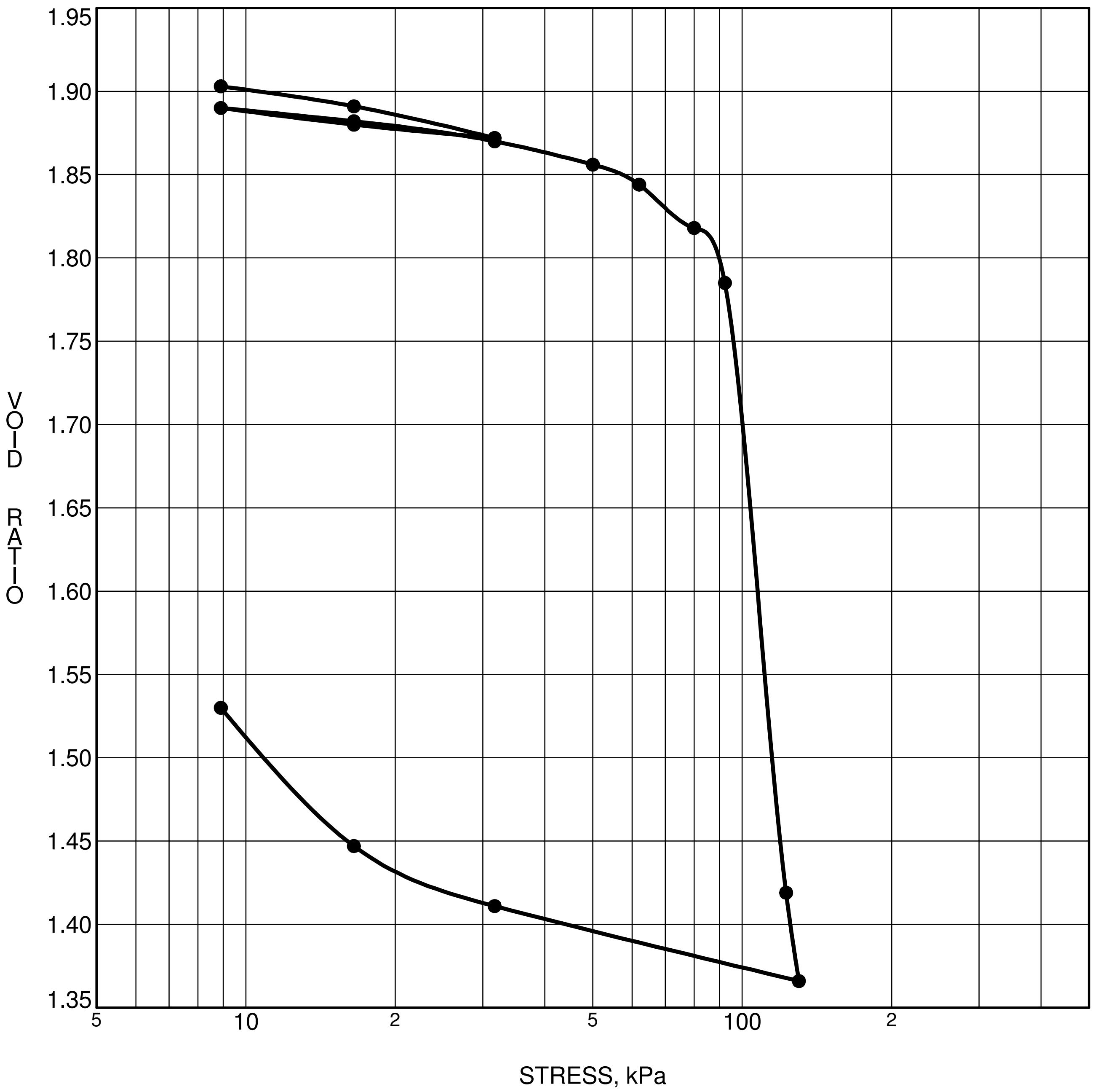
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH10**	p'o	**70** kPa	Ccr	**0.020**
Sample No.	**TW 6**	p'c	**101** kPa	Cc	**1.196**
Sample Depth	**7.16** m	OC Ratio	**1.4**	Wo	**56.5** %
Sample Elev.	**81.84** m	Void Ratio	**1.555**	Unit Wt.	**16.5** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG4444**
PROJECT	**Geotechnical Investigation - Aquaview Stage 2 -** **10th Line Road & Lakepointe Drive**	DATE	**23/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH11**	p'_o	**62** kPa	Ccr	**0.036**
Sample No.	**TW 5**	p'_c	**90** kPa	Cc	**2.775**
Sample Depth	**5.79** m	OC Ratio	**1.5**	Wo	**69.2** %
Sample Elev.	**83.39** m	Void Ratio	**1.903**	Unit Wt.	**15.7** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG4444**

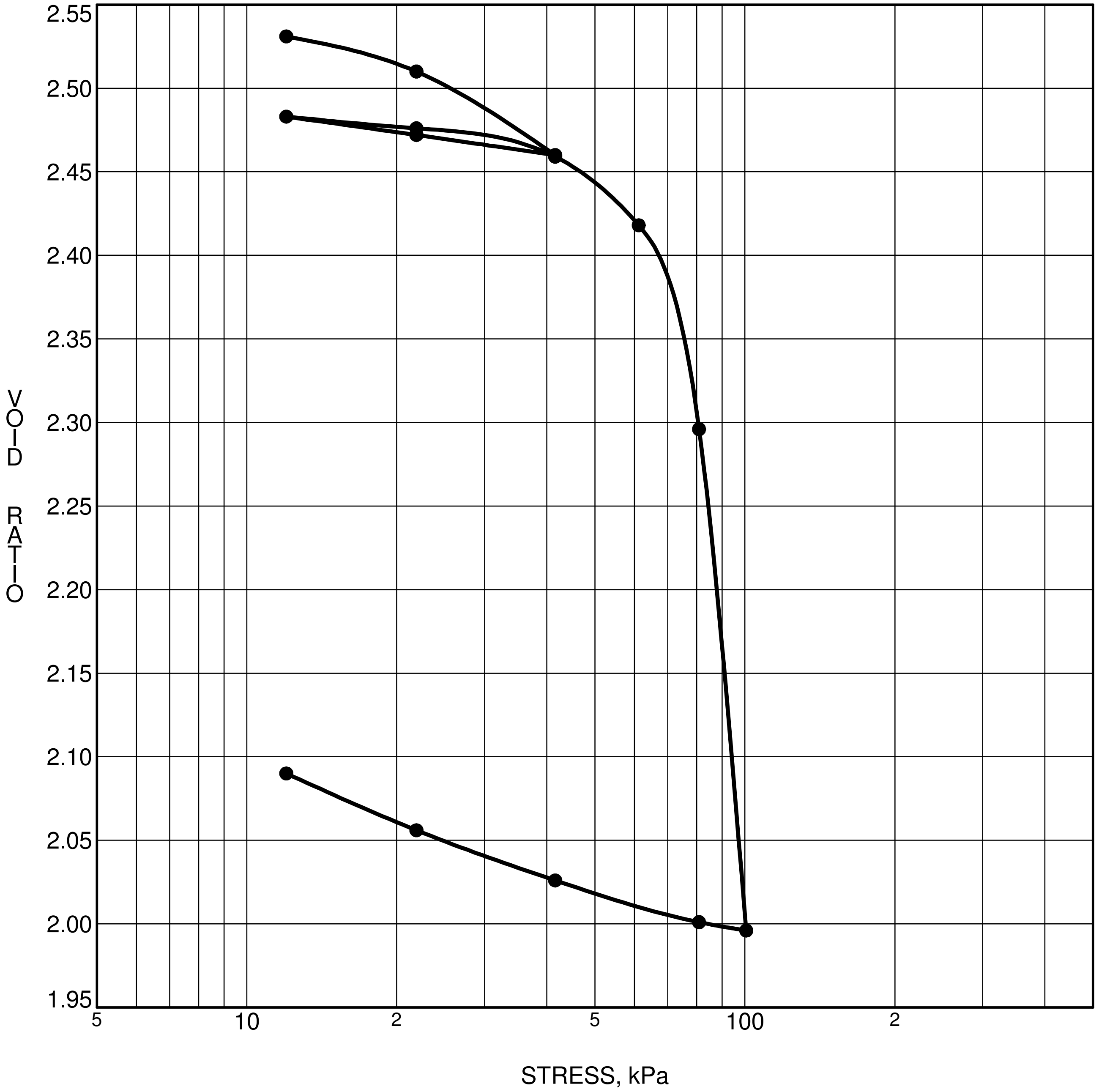
PROJECT **Geotechnical Investigation - Aquaview Stage 2 -**
10th Line Road & Lakepointe Drive

DATE **10/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH12**	p'o	**63** kPa	Ccr	**0.044**
Sample No.	**TW 9**	p'c	**74** kPa	Cc	**3.237**
Sample Depth	**6.36** m	OC Ratio	**1.2**	Wo	**93.0** %
Sample Elev.	**83.41** m	Void Ratio	**2.556**	Unit Wt.	**14.6** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG4444**

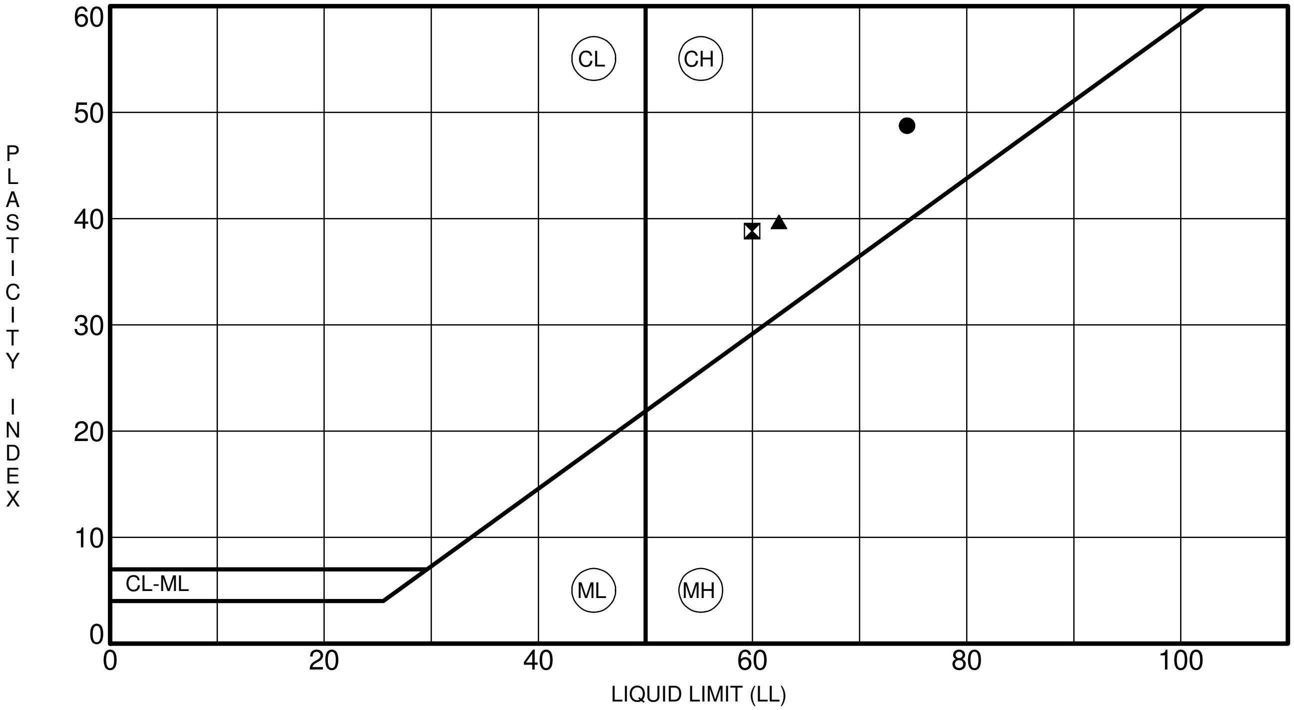
PROJECT **Geotechnical Investigation - Aquaview Stage 2 - 10th Line Road & Lakepointe Drive**

DATE **17/03/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2 TW 7**	**74**	**26**	**49**		**CH - Inorganic clays of high plasticity**
⧗	**BH 8 TW 4**	**60**	**21**	**39**		**CH - Inorganic clays of high plasticity**
▲	**BH11 TW 5**	**62**	**23**	**40**		**CH - Inorganic clays of high plasticity**

CLIENT **Minto Communities Inc.**

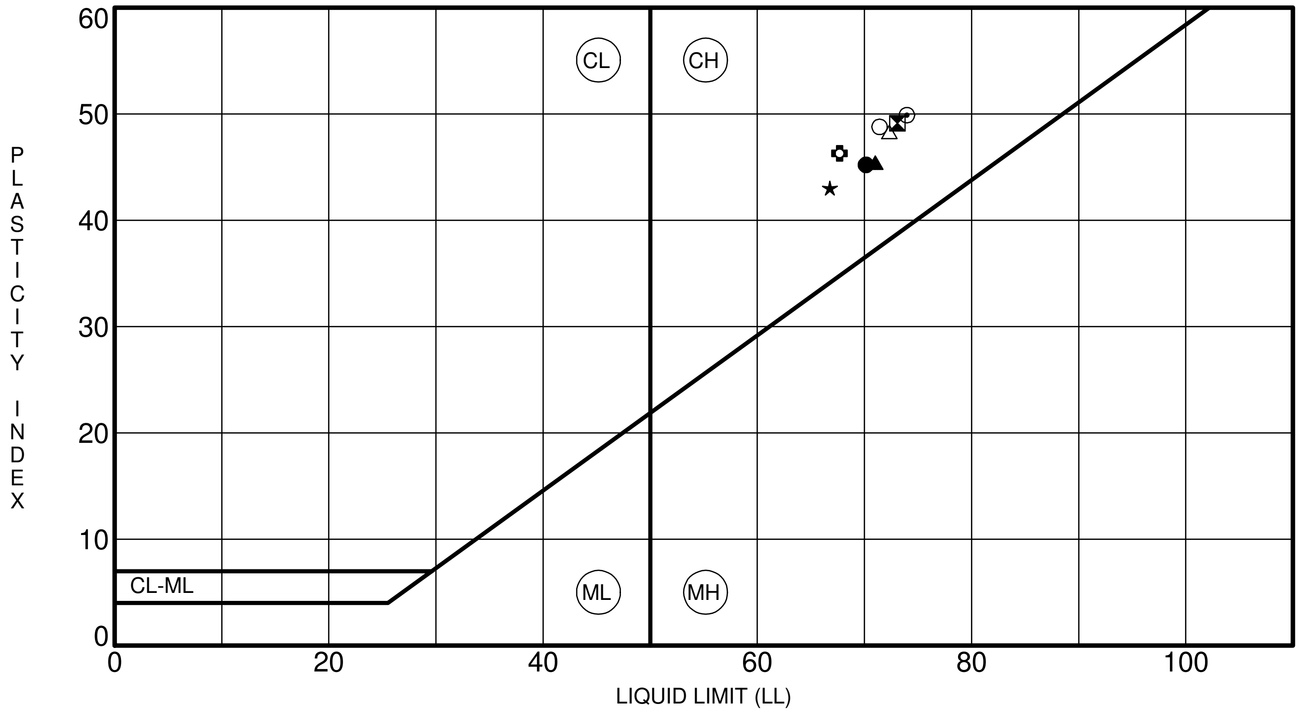
PROJECT **Geotechnical Investigation - Aquaview Stage 2 -**
10th Line Road & Lakepointe Drive

FILE NO. **PG4444**

DATE **26 Feb 18**

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

ATTERBERG LIMITS' RESULTS



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**SS 3**	**70**	**25**	**45**		**CH - Inorganic clays of high plasticity**
⧗	**BH 4**	**SS 3**	**73**	**24**	**49**		**CH - Inorganic clays of high plasticity**
▲	**BH 5**	**SS 3**	**71**	**26**	**45**		**CH - Inorganic clays of high plasticity**
★	**BH 7**	**SS 3**	**67**	**24**	**43**		**CH - Inorganic clays of high plasticity**
⊙	**BH 8**	**SS 3**	**74**	**24**	**50**		**CH - Inorganic clays of high plasticity**
✿	**BH 9**	**SS 3**	**68**	**21**	**46**		**CH - Inorganic clays of high plasticity**
○	**BH11**	**SS 3**	**71**	**23**	**49**		**CH - Inorganic clays of high plasticity**
△	**BH12**	**SS 3**	**72**	**24**	**48**		**CH - Inorganic clays of high plasticity**

CLIENT **Minto Communities Inc.**

FILE NO. **PG4444**

PROJECT **Geotechnical Investigation - Aquaview Stage 2 -**
10th Line Road & Lakepointe Drive

DATE **26 Feb 18**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 23170

Report Date: 09-Mar-2018
Order Date: 6-Mar-2018
Project Description: PG4444

	Client ID:	BH2-SS3	BH7-SS3	BH11-SS3	-
	Sample Date:	05-Mar-18	05-Mar-18	05-Mar-18	-
	Sample ID:	1810221-01	1810221-02	1810221-03	-
	MDL/Units	Soil	Soil	Soil	-
Physical Characteristics					
% Solids	0.1 % by Wt.	77.9	71.2	74.6	-
General Inorganics					
pH	0.05 pH Units	7.50	7.73	7.72	-
Resistivity	0.10 Ohm.m	24.2	39.0	27.8	-
Anions					
Chloride	5 ug/g dry	51	24	68	-
Sulphate	5 ug/g dry	59	21	30	-

APPENDIX 2

TABLE 1: SUMMARY OF SUBSURFACE INFORMATION

TABLE 2: SUMMARY OF CONSOLIDATION TEST RESULTS

FIGURE 2: UNDRAINED SHEAR STRENGTH PROFILE

TABLE 1:
SUMMARY OF SUBSURFACE INFORMATION FOR AQUAVIEW - STAGE 2
Aquaview Drive at Lakepointe Drive, Ottawa (Cumberland), Ontario

File Number	Test Hole Number	Ground Elevation (m)	Original Ground Surface Level		Underside of Stiff Clay Crust		Inferred Bedrock Surface	
			Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
PG4444-2	BH 1	89.18	0.38	88.80	2.70	86.48	31.06	58.12
	BH 2	90.50	1.70	88.80	4.20	86.30	---	---
	BH 3	89.08	0.28	88.80	2.60	86.48	---	---
	BH 4	88.97	0.07	88.90	2.60	86.37	---	---
	BH 5	89.62	0.62	89.00	3.40	86.22	28.24	61.38
	BH 6	89.38	0.58	88.80	4.00	85.38	---	---
	BH 7	88.92	0.22	88.70	3.10	85.82	33.32	55.60
	BH 8	89.08	0.28	88.80	3.20	85.88	---	---
	BH 9	89.18	0.18	89.00	3.00	86.18	---	---
	BH 10	89.00	0.40	88.60	3.30	85.70	<36.27	<52.7
	BH 11	89.18	0.38	88.80	3.40	85.78	---	---
	BH 12	89.77	0.67	89.10	4.10	85.67	24.36	65.41

Note:

1. Original ground surface (OGS) level is interpreted as approximately 0.3 m above the inorganic in situ soil levels, assuming 0.3 m of original topsoil in areas that have been stripped, partly stripped and/or covered with fill deposits. These OGS values have been rounded to the nearest 0.05 m. Guidance has also been obtained from older boreholes put down nearby in 1992 and 1999.
2. The bedrock surface levels are inferred to be at the depths of practical refusal of the dynamic cone penetration test (DCPT).

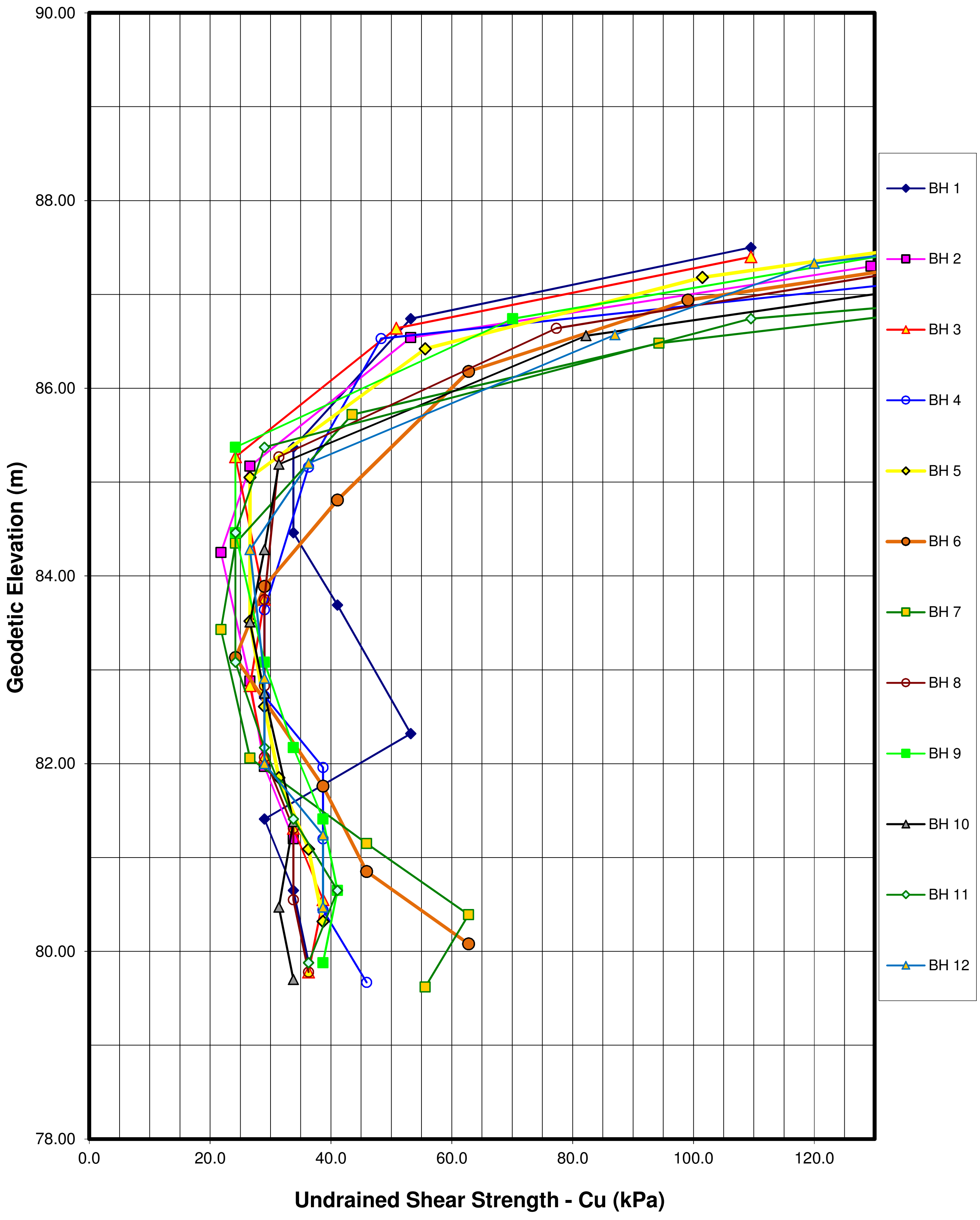
TABLE 2
SUMMARY OF CONSOLIDATION TEST RESULTS - AQUAVIEW - STAGE 2
Aquaview Drive at Lakepointe Drive, Ottawa (Cumberland), Ontario

Sample No.	Ground Elev. (m)	Depth (m)	Elevation (m)	p'_c (kPa)	p'_o (kPa)	O.C. (kPa)	Ccr	Cc	W.C. (%)	Disturbance Factor & Limits (%)
Avalon Aquaview Stage 2 - 2018 Testing Program:										
BH2 - TW7	90.50	7.20	83.30	85	62	23	0.047	4.587	96	2.0< **2.3** < 4.0 = **OK**
BH3 - TW6	89.08	4.98	84.10	76	58	18	0.028	1.476	67	2.0< **2.0** < 4.0 = **OK**
BH5 - TW6	89.62	5.61	84.01	75	59	16	0.040	1.260	59	2.0< **3.9** < 4.0 = **Poor**
BH7 - TW6	88.92	6.48	82.44	88	67	21	0.025	2.020	76	1.6< **2.0** = **V. Good**
BH8 - TW4	89.08	3.38	85.70	148	48	100	0.017	1.018	52	1.0 < **1.1** < 3.0 = **OK**
BH9 - TW5	89.18	5.74	83.44	70	63	7	0.044	2.459	72	3.0< **4.8** < 5.0 = **Poor**
BH10 - TW6	89.00	7.16	81.84	101	70	31	0.020	1.196	56	2.0< **2.8** < 4.0 = **OK**
BH11 - TW5	89.18	5.79	83.39	90	62	28	0.036	2.775	69	1.5 < **2.1** < 3.5 = **OK**
BH12 - TW9	89.77	6.36	83.41	74	63	11	0.044	3.237	93	**4.1** > 4.0 = **Disturb.±**

Notes:

1. Effective overburden pressure, p'_o, is based on groundwater depths at 0.5 m above the underside of the stiff silty clay crust. These values are also based on the inferred original ground surface without the weight of fill material and are based on the inorganic in situ soil level (see Table 1), with the weight of 0.3 m of topsoil added.
2. The last (right) column presents the disturbance ratio of the test sample (Lacasse et. al.) in bold and compares it to the acceptable range (OK samples), or the upper limit of the acceptable range (**Poor** =poor to disturbed samples close to the upper limit and **Disturb.±** = disturbed samples over the upper limit). The lower limit of the acceptable range indicates “good” to “very good” samples.

Figure 2: Shear Strength Profile - Avalon Aquaview - Stage 2



APPENDIX 3

FIGURE 1: KEY PLAN

DRAWING PG4444-1: TEST HOLE LOCATION PLAN

DRAWING PG4444-2: PERMISSIBLE GRADE RAISE PLAN

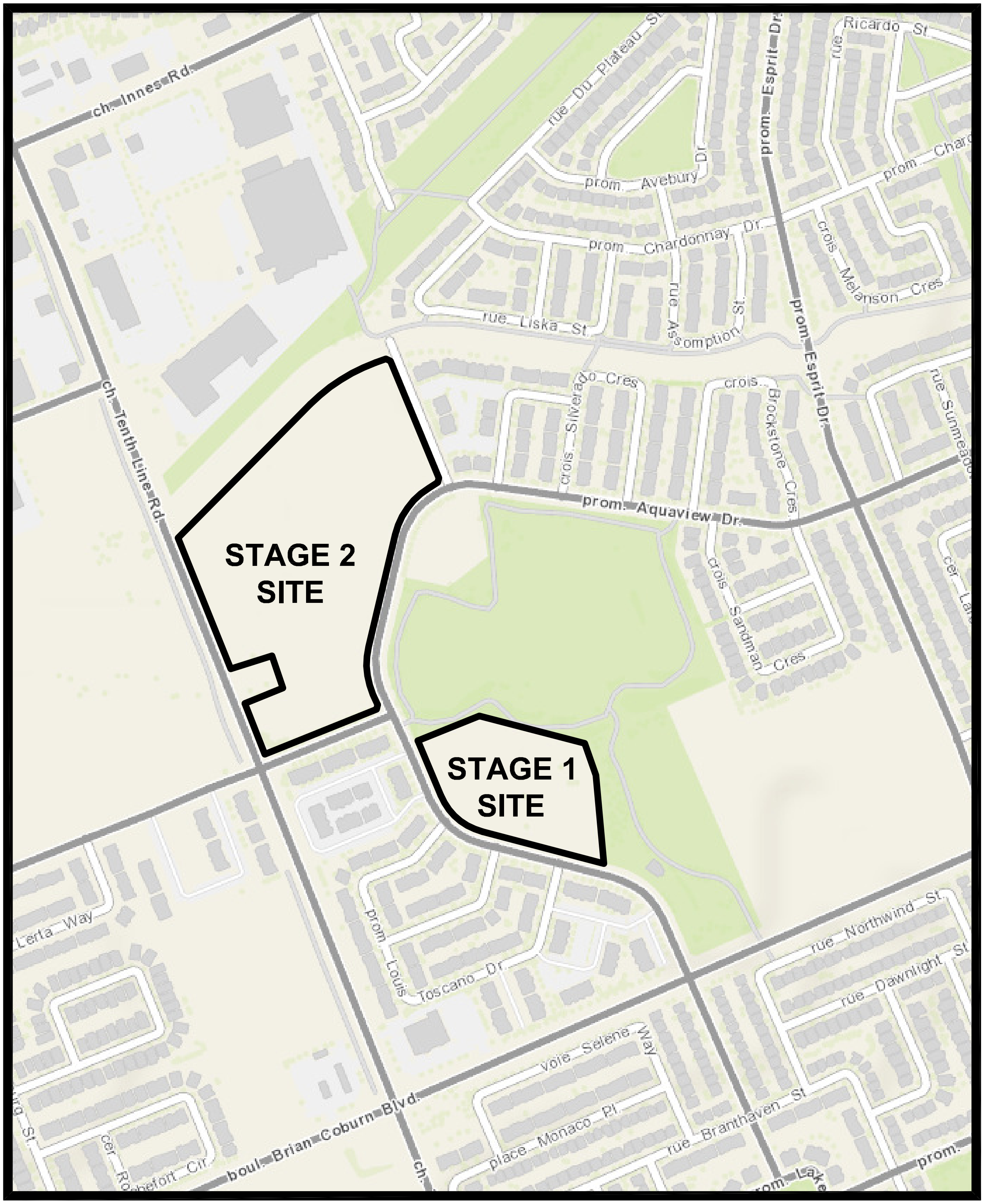


FIGURE 1

KEY PLAN

p:\autocad drawings\geotechnical\pg44xx\pg4444\pg4444 key plan.dwg

LEGEND:

⌖	BOREHOLE LOCATION, CURRENT INVESTIGATION
◆	BOREHOLE LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG1950-LET.01, JAN. 24, 2012
⊞	TEST PIT LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG1950-LET.01, JAN. 24, 2012
89.18	GROUND SURFACE ELEVATION (m)
[88.36]	INORGANIC IN SITU SOIL LEVEL (m)
(61.90)	PRACTICAL DCPT REFUSAL ELEVATION (m)

NOTES:

1. STAGE 1 - TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ATREL ENGINEERING (2009).
2. STAGE 2 - TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY STANTEC GEOMATICS LIMITED.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

MINTO COMMUNITIES INC.
GEOTECHNICAL INVESTIGATIONS
AVALON AQUAVIEW STAGES 1 & 2 - AQUAVIEW DRIVE AND TENTH LINE ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Stamp:

Scale:	1:1000
Drawn by:	MPG
Checked by:	AJT
Approved by:	AJT
Date:	03/2018
Report No.	PG4444-1
Job No.:	PG4444-1
Revision No.	0

p:\autocad drawings\geotechnical\pg44xx\pg4444\pg4444-1 ttb.dwg

LEGEND:

◈	BOREHOLE LOCATION, CURRENT INVESTIGATION
◆	BOREHOLE LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG1950-LET.01, JAN. 24, 2012
⊞	TEST PIT LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG1950-LET.01, JAN. 24, 2012
89.18	GROUND SURFACE ELEVATION (m)
[88.36]	INORGANIC IN SITU SOIL LEVEL (m)
(61.90)	PRACTICAL DCPT REFUSAL ELEVATION (m)

PERMISSIBLE GRADE RAISE LEGEND:

88.80	INTERPRETED ORIGINAL GROUND ELEVATION (m)
0.80	PERMISSIBLE GRADE RAISE (m)
89.60	PERMISSIBLE FINISHED GRADE (m)
90.00	PERMISSIBLE FINISHED GRADE (m) USING LIGHTWEIGHT FILL (LWF)

NOTES:

1. STAGE 1 - TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ATREL ENGINEERING (2009).
2. STAGE 2 - TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY STANTEC GEOMATICS LIMITED.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

MINTO COMMUNITIES INC.
GEOTECHNICAL INVESTIGATIONS
AVALON AQUAVIEW STAGES 1 & 2 - AQUAVIEW DRIVE AND TENTH LINE ROAD
OTTAWA, ONTARIO

Title:
PERMISSIBLE GRADE RAISE PLAN

Stamp:

Scale: 1:1000
Drawn by: MPG
Checked by: AJT
Approved by: AJT
Date: 03/2018
Report No. PG4444-1
Job No.:
PG4444-2
Revision No. 0

p:\autocad drawings\geotechnical\pg44xx\pg4444\pg4444-1 thlp.dwg