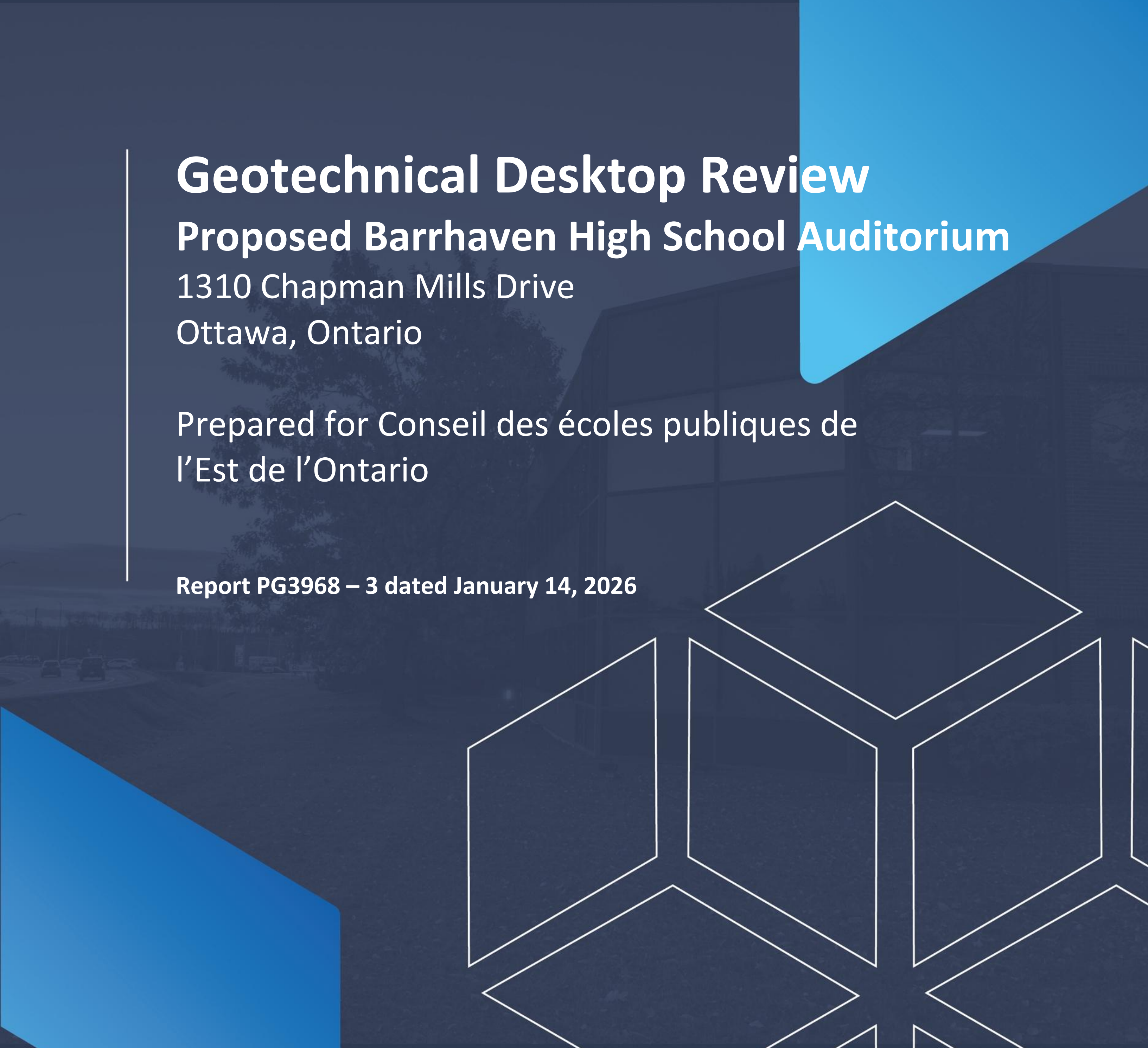
Geotechnical Desktop Review

Proposed Barrhaven High School Auditorium

1310 Chapman Mills Drive
Ottawa, Ontario

Prepared for Conseil des écoles publiques de l'Est de l'Ontario

Report PG3968 – 3 dated January 14, 2026



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Conseil des Écoles Publiques de l'Est de l'Ontario (CEPEO) to prepare a geotechnical desktop review for the proposed Barrhaven high school auditorium addition to be located at 1310 Chapman Mills Drive in Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical desktop review are to:

☐ Determine the subsoil and groundwater conditions at this site by means of existing test holes.

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

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

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

2.0 Proposed Development

Based on our review of available drawings, it is understood that the proposed project will consist of slab on grade proposed auditorium addition to the existing high school building. It is understood that the proposed finished floor geodetic elevation is 94.6 m. Proposed soft and hard landscaping is also anticipated surrounding the new auditorium addition.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Previous geotechnical field programs for the overall Barrhaven High School development were completed by this firm in 2011, 2015, 2016 and 2019. During the previous investigations, a total of twenty (20) boreholes were advanced within, or in proximity to, the subject site to maximum depths of 17.7 m.

The location of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG3968-2 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance drill rig operated by a two-person crew. All fieldwork by Paterson was conducted under the full-time supervision of our personnel under the direction of a senior engineer from the geotechnical division.

The drilling procedure, for the 2019 and 2011 boreholes, consisted of augering to the required depths at the selected locations and sampling and testing the soils encountered. Hollow-stem augers allowed for the use of the augers as casing.

The drilling procedure, for the 2015 and 2016 boreholes, consisted of using “wash boring” rotary drilling methods to advance an open casing. This method reduces the sampling disturbance, as compared to the hollow-stem augering method, but is slower and requires the use of drilling water, to be supplied by a water truck. However, the wash boring method has proven to provide the conditions to obtain the most accurate in situ shear strength test results and the best quality “undisturbed” soil samples possible.

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. The split spoon soil samples were classified on site and placed in sealed plastic bags. The Shelby tubes were recovered from the borehole using a piston sampler, were sealed with caps at both ends and protected from disturbance during the entire process. Auger samples were recovered from the upper part of the boreholes. 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 samples, respectively, on the Soil Profile and Test Data 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.

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.

Undrained shear strength testing was carried out in the cohesive soils encountered in the boreholes, using 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.

The depth to inferred bedrock was evaluated during the course of the previous investigation stages by conducting dynamic cone penetration testing (DCPT) to practical refusal. The DCPT was completed at eleven (11) of the previous 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. In general, the cone was pushed without driving through part of the clay prior to starting the DCPT. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment of penetration.

For the 2019 boreholes, the evaluation of the inferred bedrock surface was considered to be important, as the use of a deep foundation alternative was known to be required. As such, it was initially planned to drill to practical auger refusal on the assumption that it would be more reliable than the DCPT. However, other than BH 5-19, the auger refusal was encountered at too shallow a depth to be considered accurate. At BH 4-19 and BH 6-19, the augers had deflected at the depth of refusal, and it was not considered to be safe to pull the plug and drive a cone. At BH 2-19 and BH 3-19 a cone was driven to practical refusal below the depth of auger refusal and was able to penetrate further through overburden. At BH 1-19, a cone was driven to practical refusal below the sampling depth without achieving prior auger refusal.

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

Groundwater

Flexible standpipes were installed in each of the boreholes to permit monitoring of the groundwater levels within the silty clay subsequent to the completion of the sampling program for each stage of investigation. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

All the previous test hole locations were referenced accurately in the field and the ground surface elevation at the time of the fieldwork was referenced to geodetic datum. **Note that for the older investigation stages, the recorded ground elevation may have changed due to subsequent site activities, such as placement and spreading of fill materials.**

The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG3968-2 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the previous investigation and were visually examined in our laboratory to review the results of the field logging. Shelby tube samples were saved (at the applicable investigation time) and stored for future testing purposes.

The Shelby tube samples that were used for consolidation testing were processed to determine the water content, as well as to record a description of the soil. Atterberg Limits testing was conducted on several of the tested samples to determine the plasticity characteristics of the soil.

To provide additional geotechnical data pertaining to the permissible grade raises for the site and to provide information for settlement analyses, selected undisturbed samples of the silty clay stratum, recovered using a piston sampler in Shelby tubes, were subjected to consolidation testing in our laboratory. Eighteen (18) Shelby tube samples from the previous (2011, 2015 and 2016) boreholes were submitted for unidimensional consolidation testing.

The consolidation test results are plotted and tabulated (Table 2) in Appendix 1 and are discussed under subsections 5.3 of this report. Atterberg Limits test results are also provided in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The overall subject site consists of the existing Barrhaven high school with an existing building on the north side of the site and a paved parking area with access lanes southeast of the existing building. The remaining southern portion of the site is mainly grass covered, with gravel pedestrian pathways, paved sport fields and modular classrooms in the western corner. The overall site is bordered to the north by Strandherd Drive and Chapman Mills Drive, to the east by Chapman Mills Drive, to the west by Strandherd Drive and to the south by residential developments and vacant land.

The ground surface along the north, northwest and northeast area relatively flat and at grade with the adjacent roads. The site slopes down slightly to the south.

The subject area of the proposed auditorium addition is located south of the eastern leg of the building and is currently occupied by a grass area with gravel pedestrian pathways.

4.2 Subsurface Profile

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile for the previous boreholes that were located on or adjacent to the Barrhaven High School parcel.

Existing fill (and remnant topsoil) deposits of between 1.2 and 2.3 m in thickness were encountered at the 2019 boreholes. The topsoil or cultivated soil layer appeared to have been stripped from the greater part of the subject parcel. The inorganic in situ soil surface levels, interpreted as the native soil below the fill and any remnant topsoil, are summarized in Table 1, in Appendix 2.

The inorganic native soil profile underlying the site consists primarily of a thick layer of sensitive silty clay that underlies the existing fill and/or a thin clayey sandy silt deposit. Based on the augering that was done below the sampled depth, the thickness of the silty clay layer is estimated to generally range from about 6 to 10 m. The silty clay is inferred to be underlain by a glacial till layer, primarily indicated by the drill “chatter” (i.e. auger encountering coarse granular material) during augering, with inferred depths indicated on the logs. The bedrock directly underlies the glacial till layer.

Silty Clay

Silty clay was encountered immediately beneath the existing fill and/or a thin clayey sandy silt layer at all test hole locations. The upper portion of the silty clay layer has been weathered to a very stiff to stiff brown to grey crust. Within the Barrhaven High School parcel, for the 2015 investigations, the crust extends to depths varying between 3.2 m (BH 3-15) and 5.2 m (BH 7-15) below the original ground surface level, with a mean underside of crust depth of 4.1 m. The interpreted level of the underside of the stiff crust is tabulated for each borehole location in Table 1, in Appendix 2.

Grey silty clay was encountered below the weathered crust at all test hole locations. In situ shear vane field testing carried out within the sampled depths of this layer yielded undrained shear strength values generally ranging from 30 kPa to more than 50 kPa. A few shear strength below 30 kPa were encountered (in the 2019 boreholes). Somewhat lower shear strengths were encountered in the previous boreholes, but these are interpreted to have been affected by the drilling method. These values are indicative of a firm (occasionally soft) to stiff consistency.

Based on Atterberg Limits testing conducted on samples of the silty clay from the previous investigations, the silty clay is somewhat variable in plasticity. The silty clay can generally be classified as a clay of low plasticity (CL), although one test each indicated a silt of low plasticity (ML) and a clay of high plasticity (CH). Results of the Atterberg Limits testing of samples recovered on and adjacent to the Barrhaven High School parcel are provided in Appendix 2.

Eighteen (18) samples of the silty clay from the previous investigations were subjected to unidimensional consolidation testing. The plotted results of all the current and previous test samples that are within or adjacent to the Barrhaven High School parcel, are presented in Appendix 2 and are discussed and interpreted under subsection 6.3. All the consolidation test results are summarized in Table 2, in Appendix 2.

Glacial Till

A glacial till deposit was encountered or inferred below the silty clay based on auger "chatter" and DCPT test results. The glacial till was sampled just below the auger refusal in BH 1-19 and the matrix at that location consists of a fine soil matrix of grey silty clay with sand and gravel. The matrix is expected to range to a silty sand and gravel with a trace of clay. Based on the extent of the auger chatter, as well as the observed auger refusal within the glacial till, it should be expected that the glacial till contains cobbles and boulders.

The results of DCPT testing conducted within the glacial till layer had N values ranging from less than 10 to over 50 blows per 300 mm increment of penetration, indicating a compactness condition ranging from loose to very dense.

Practical Refusal to Augering and DCPT- Inferred Bedrock

The depths to practical refusal to DCPT testing are shown on their respective Soil Profile and Test Data sheets in Appendix 1. This information has been used to infer the bedrock surface, as shown in parentheses on the Test Hole Location Plan, in Appendix 2, and summarized in Table 1, in Appendix 1. Values of practical auger refusal are not considered to be reliable indicators from which to infer the bedrock surface and, as such, are only described on their respective Soil Profile and Test Data sheets. Other than at BH 5-19, where a hard smooth surface was encountered at auger refusal, it is inferred that the auger refusal depths were on nested cobbles or boulders in the glacial till layer.

Based on available geological mapping, the depth to bedrock in the area is expected to range from 10 to 15 m. The bedrock is shown to be part of the March formation, which consists of interbedded sandstone and dolomite. Information provided from the construction of the school indicates that pilling refusal occurred 15-16 m below the installation depth.

4.3 Groundwater

Groundwater levels were measured in the standpipe installations in the boreholes subsequent to the completion of their respective fieldwork programs. The groundwater level (GWL) readings are summarized in Table 3, in Appendix 2.The previous measured groundwater levels ranged from elevation 89.4 m to 91.9 m, with a mean of elevation 91.0 m. The measured groundwater levels were fairly similar throughout the period covered by the previous investigations. The measured groundwater levels of the 2019 investigation are higher and range from elevation 92.7 m to 93.4 m, with a mean of elevation 93.1 m. It is expected that these significantly higher groundwater levels could be the result of the borehole readings being recorded when thick snow cover is melting, and the infiltrating melt water is potentially perched in the existing fill material over the native soils.

No readings have been provided for standpipe installations that appeared to be blocked or could not be found and were assumed to have been removed or destroyed. The groundwater levels recorded in BH 2-15 and BH 5-15 were low, compared to the other observations, and the level of the underside of stiff clay crust, and these installations were inferred to be blocked or to have not completely stabilized by the applicable time of reading. The installations in BH 3-19 and BH 5-19 also appear to be blocked and the one in BH 6-19 sunk after installation. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could be different at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed project. It is recommended that the proposed auditorium addition be founded on deep foundations such as driven piles.

Due to the presence of a silty clay deposit throughout portions of the site, permissible grade raises restrictions have been provided. The geodetic finished floor elevation (FFE) of the proposed auditorium addition has been set at 94.60 m.

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

5.2 Site Grading and Preparation

Compacted Granular B Type II Working Platform

The proposed building will be supported on a driven pile foundation that requires the use of heavy equipment (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the piling equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 0.6 m of OPSS Granular B Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be regraded, and contamination from soil tracked in, or of soil pumping up from the pile installation locations, can be bladed off, and the surface can be topped up, if necessary, and recompacted to act as the substrate for further fill placement for the slab-on-grade.

Stripping Depth

Topsoil and any deleterious fill, such as that containing organic materials, should be generally stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures to expose inorganic subgrade media.

Much of the existing fill that has been placed on the subject property has not been consolidated during placement and, as such, should be reviewed at the time of construction to confirm its suitability to be maintained under pavements or other structures.

With respect to the granular fill working platform, within the building, it is anticipated that the granular fill working platform will be founded on the existing fill within much of the proposed auditorium addition footprint. The working platform can be founded on the existing fill following the excavation to the platform underside level and subsequent evaluation of any remnant existing fill (and topsoil, if present) under the observation of the geotechnical consultant. Evaluation methods would include a grid of shallow test pits to determine the thickness of the fill and proof-rolling (under dry conditions) the existing fill to evaluate its suitability. Soft areas would have to be subexcavated and filled with suitable material, whether consisting of suitable unspecified fill, or granular fill.

Alternatively, an inorganic in situ soil subgrade could be prepared under the entire building footprint by subexcavating where necessary below the proposed underside of working platform level and filling the subexcavations with Granular B Type II. It should be noted that should this option be retained, the use of lightweight fill may be required due to the additional stress applied from the engineered fill grade raise.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. Testing may consist of the suppliers own Quality Control testing, or samples can be submitted to the geotechnical consultant for testing.

The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building areas should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD). The materials comprising the pavement structures should be compacted to at least 100% of their SPMDD values.

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geo-composite drainage membrane connected to a perimeter drainage system.

5.3 Foundation Design

General Comments

With the proposed FFE (94.6 m), the weight of the grade raise fill uses all the available overconsolidation capacity of the silty clay so it is not feasible to support the proposed building on shallow footing foundations. As such, the building has been designed to be supported by a deep (pile) foundation. End (toe) bearing steel pipe piles, filled with concrete, are recommended for this purpose.

Pile Foundation Design

The foundation loads for the proposed high school can be transferred to bedrock using a deep foundation alternative, such as end-bearing (toe-bearing) piles, driven to refusal at or near the bedrock surface. For deep foundations, concrete-filled steel pipe piles are frequently utilized in the Ottawa area. The steel pipe piles would be driven closed-ended to refusal with a plate welded to the toe of the pile and then, after observation by the geotechnical consultant confirms the pile is not damaged, the pipe would be cut-off and filled with concrete.

Limit States Design Pile Axial Resistance

Applicable pile axial resistance at serviceability limit states (SLS) values and factored pile resistance at ultimate limit states (ULS) values are summarized in Table 1, on the following page, for several typical steel pipe pile stocks, and using moderate driving energies. Additional pile size alternatives can be provided upon request. A resistance factor of 0.4 has been incorporated into the factored at ULS values.

Note that the tabulated values are all geotechnical axial resistance values. The structural values are expected to be greater than the geotechnical values, as the applicable factors for the fully laterally restrained concrete and/or steel structural materials should be in excess of what are required by the applicable geotechnical capacities, tabulated below, and that the toes of the piles will be well constrained by soil at the refusal level.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula. The Piling Contractor will be expected to prepare and submit a pile size or sizes, along with driving criteria, to meet the design pile requirements from the Structural Engineer.

It is recommended that dynamic monitoring and capacity testing should be conducted early in the pile driving program to verify the transferred energy from the pile driving equipment and confirm the pile geotechnical capacity using the accepted driving criteria. This testing is conducted during a restrike on pile(s) that have been previously driven and given a few days to stabilize. Based on the results of the dynamic monitoring, adjustments can be made to the driving criteria to ensure that the geotechnical pile capacities will be achieved.

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

Table 1 - Summary of Pile Foundation Design Data

Pile Outside Diameter (mm)	**Pile Wall Thickness (mm)**	**Geotechnical Axial Resistance**		**Final Set (blows/ 12 mm)**	**Transferred Hammer Energy (kJ)**
		SLS (kN)	**Factored at ULS (kN)**		
245	9.0	750	900	6	29
245	11.0	850	1020	6	35
245	13.0	950	1150	5	42

The maximum recommended driving energy is 6 x 10^6 joules per square metre of steel cross sectional area. The tabulated axial resistance values are based on driving energies lower than the maximum, so that there is some flexibility in adjusting the driving criteria based on the dynamic monitoring program.

Downdrag Loads

Due to the proposed grade raises at the site, downdrag loads should be considered on the piles. Based on the available subsurface information, it is expected that the piles will be driven through approximately 10 m of stiff to firm silty clay. The 3 m of stiff silty clay crust has a cohesion of 100 kPa and the 7 m of firm silty clay an average cohesion of 35 kPa.

Assigning adhesion factors of 0.5 and 0.75, to the stiff and firm clay, respectively the silty clay can be taken to have an ultimate adhesion of between 50 and 25 kPa against the sides of the piles. As such, the maximum estimated downdrag load for each 245 mm diameter pile is anticipated to be 260 kN.

The downdrag load is effectively applied to each pile at the location of the “neutral plane,” where negative (i.e. downdrag) skin friction becomes positive shaft resistance. In the case of the end-bearing piles at this site, the neutral plane will be located within the glacial till near the bedrock surface.

The downdrag load is a structural pile capacity criterion and does not affect the geotechnical capacity of the piles. The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the downdrag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but downdrag load is to be excluded.

At the depth of the neutral plane where the downdrag load is applied, the pile structure is well confined. The 5th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the downdrag load, can be determined by applying an appropriate resistance factor on the pile material strength (steel yield and concrete 28 day compressive strength). As such, downdrag loads are not expected to be of concern with the pile design at this site.

Minimum Pile Spacing

The minimum centre-to-centre pile spacing should be 2.5 times the pile diameter. The closer the piles are spaced, however, the more the potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of the earlier-driven piles, are checked as part of the field review of the pile driving operations.

Grade Raise Stress Effects

The proposed high school auditorium addition will be founded on piles and, as such, the foundation loads will not contribute stress to the soil. However, the granular fill materials needed to achieve a FFE of 94.6 m will add a sustained load to the soil.

It is expected that conventional granular fill materials will be used and the granular working platform will be constructed over the reviewed and approved fill layer. Based on available grading plans, the estimated grade raise is approximately 0.3 to 0.8 m above the existing grade (existing grade elevation varies from 94.35 to 93.80 m within the proposed auditorium addition). Therefore, the proposed grade raise would impart a fill weight stress of the order of 10 to 20 kPa in addition to the existing fill in place.

Due to the presence of the silty clay deposit throughout the site, a permissible grade raise restriction to a **maximum geodetic elevation of 94.6 m** is recommended for the subject site where silty clay is encountered below the building footprint.

The following assumptions and recommendations were considered and should be taken into account during the design process:

- ☐ The estimated amount of total slab-on-grade settlement is not great and is within the recompression range of the soil therefore most of the estimated settlement is expected to occur elastically during the construction period and before the slab-on-grade concrete is placed.
- ☐ It is recommended to construct the granular working platform to the FFE of 94.6 m to allow for initial settlement to occur during the pilling activities and remove the thickness of granular pad required to allow for the placement of the slab-on-grade.
- ☐ There is essentially no difference in estimated settlement between the portion of the slab within the building and the settlement of the surrounding grade, so differential effects across entrance areas should not be of concern.
- ☐ A settlement tolerant floor surface finish is recommended.
- ☐ It is recommended that the slab be designed to be ‘floating” (not bonded to) with respect to the pile-supported structural elements.
- ☐ All load-bearing walls should be supported on pile foundations.

5.4 Seismic Design

The site class for seismic site response can be taken as **Class X_D** for the foundations considered as defined in Table 4.1.8.4.-B of the Ontario Building Code (OBC) 2024. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade Construction

The slab-on-grade will be built up over the granular fill working platform, once the piles have been driven and cut off and the pile caps and grade beams have been constructed. Any contaminated soil should be removed from the granular fill and the surface should be compacted to re-establish the 98% of SPMDD level of compaction.

The use of OPSS Granular B Type I, or Type II, with a maximum particle size of 50 mm, is recommended under the building slab-on-grade. The upper 150 mm of sub-slab fill should consist of OPSS Granular A crushed stone for slab-on-grade floors. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the SPMDD.

5.6 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of the car parking areas and local roadways.

Table 4 – Recommended Pavement Structure – Car only Parking Area

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 5 – Recommended Pavement Structure – Access Road, Heavy Vehicle Traffic

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed addition. The system, should consist of a 100 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 10 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer. The new perimeter foundation drain should be placed at the same elevation and connected to the existing building perimeter foundation drain.

Foundation Backfill

For proposed buildings with below-grade space, backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient space will be available for the great part of the excavations to be undertaken by open-cut methods (i.e., unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below ground water level. The subsoil at this site appeared to be mainly a Type 2 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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 use of non-woven geotextile and clear stone may be necessary to control silt and prevent clogging of submersible pumps.

Groundwater Control for Building Construction

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

6.5 Winter Construction

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

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

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

6.6 Corrosion Potential and Sulphate

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

The appropriate concrete exposure class is “N”, for soil contact based on chloride content, where freezing and thawing (F-1 or F-2 exposure class) is not an issue.

6.7 Landscaping Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed as part of the previous investigation on the recovered silty clay samples at selected locations throughout and in proximity to the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. Therefore, the silty clay soils should be considered to be within the low to medium potential for soil volume change.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit and where trees are located near buildings founded on cohesive soils.

- ☐ Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.

- ☐ Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), provided that the conditions noted below are met.

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

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

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

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

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ☐ Review preliminary and detailed grading and servicing plan(s) from a geotechnical perspective.
- ☐ Full time field review services during the driving of piles, including recording pile lengths, refusal criteria, relative pile location and plumbness, and complete pile dynamic monitoring.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Observation of the placement of the foundation insulation, if applicable.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided herein are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Conseil des Écoles Publiques de l'Est de l'Ontario, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
January 14, 2026
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Nicolas Seguin

Nicolas R. P. Seguin, P.Eng.

Joey R Villeneuve, M.A.Sc, P.Eng.

Report Distribution:

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☐ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TABLE 1: SUMMARY OF SUBSURFACE INFORMATION

TABLE 2: SUMMARY OF CONSOLIDATION TEST RESULTS

TABLE 3: SUMMARY OF GROUNDWATER LEVELS

CONSOLIDATION TEST SHEETS

ATTERBERG LIMITS RESULTS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 22, 2019

FILE NO. PG3968

HOLE NO. BH 1-19

SOIL DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand with gravel and cobbles, some clay — 2.20

Very stiff to stiff, brown **SILTY CLAY**

- grey by 3.8m depth

Augered 4.42m to 10.67m depth — 8.00

Drill chatter commencing at 8.0m depth

Inferred **GLACIAL TILL**

Auger refusal at 10.67m depth — 10.67

GLACIAL TILL: Grey silty clay with sand, gravel, cobbles, boulders — 11.28

Dynamic Cone Penetration Test commenced to 11.28m depth.

Inferred **GLACIAL TILL** — 15.22

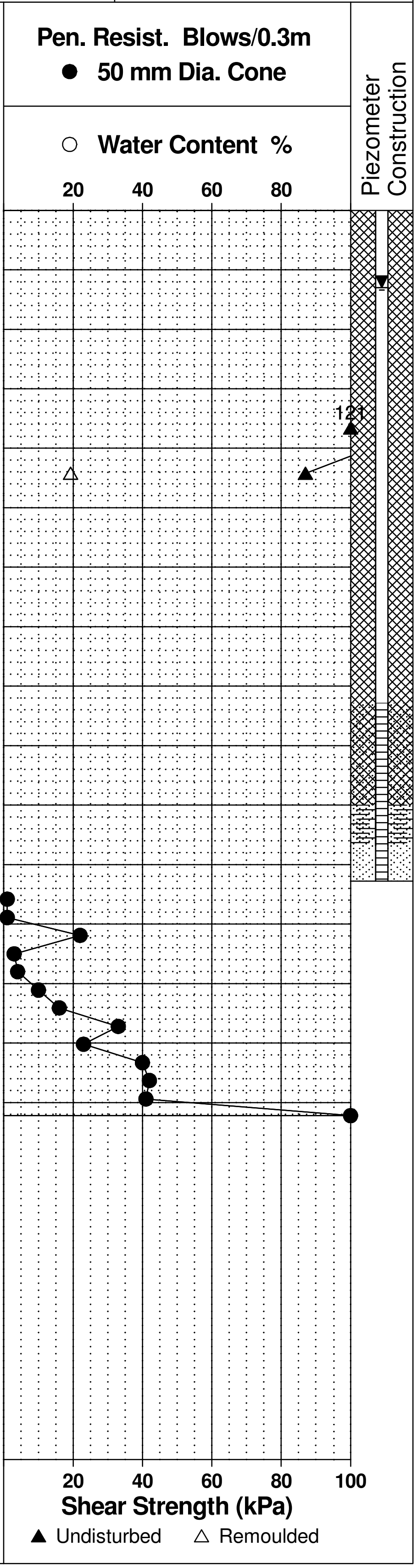
End of Borehole

Practical DCPT refusal at 15.22m depth

(GWL @ 1.30m - March 29, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	15
SS	3	46	44
SS	4	83	4
SS	5	100	2
SS	6	100	2
SS	7	100	3

DEPTH (m)	ELEV. (m)
0	94.45
1	93.45
2	92.45
3	91.45
4	90.45
5	89.45
6	88.45
7	87.45
8	86.45
9	85.45
10	84.45
11	83.45
12	82.45
13	81.45
14	80.45
15	79.45



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 22, 2019

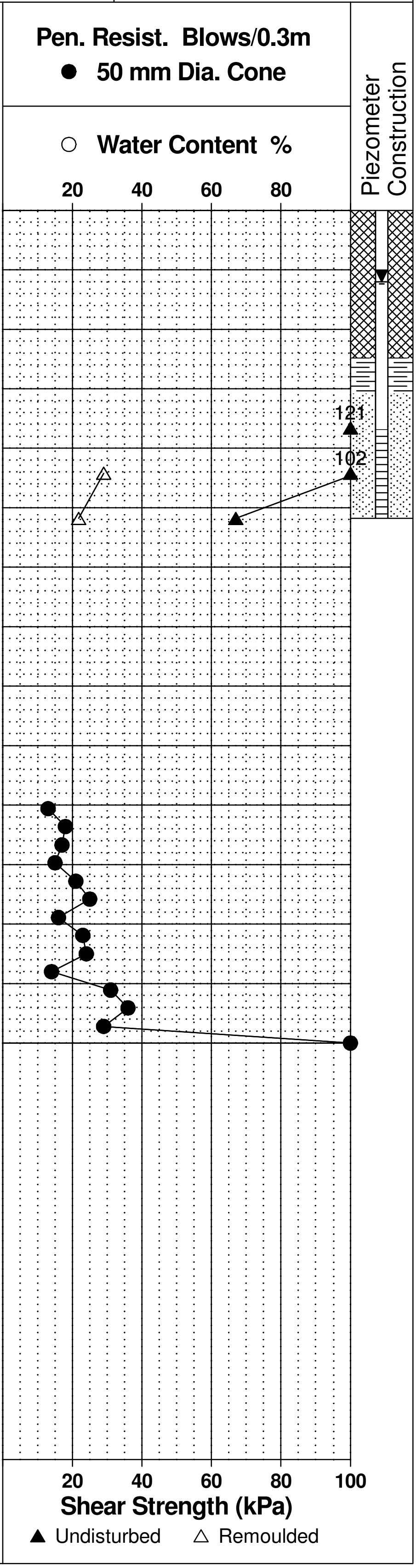
FILE NO. PG3968

HOLE NO. BH 2-19

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, some organics	1.50
FILL: Dark grey sandy silt, trace clay and gravel	1.80
Very stiff to stiff, brown **SILTY CLAY** - grey by 3.8m depth	5.18
Borehole augered from 5.18 to 9.75m depth Inferred **SILTY CLAY** Drill chatter commencing at 8.8m depth	8.80
Inferred **GLACIAL TILL** Auger refusal at 9.75m depth	9.75
Dynamic Cone Penetration Test commenced at 9.75m depth Inferred **GLACIAL TILL**	14.00
End of Borehole Practical DCPT refusal at 14.00m depth (GWL@ 1.20m - March 29, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	10
SS	3	38	9
SS	4	8	8
SS	5	100	5
SS	6	100	2
SS			

DEPTH (m)	ELEV. (m)
0	93.87
1	92.87
2	91.87
3	90.87
4	89.87
5	88.87
6	87.87
7	86.87
8	85.87
9	84.87
10	83.87
11	82.87
12	81.87
13	80.87
14	79.87



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

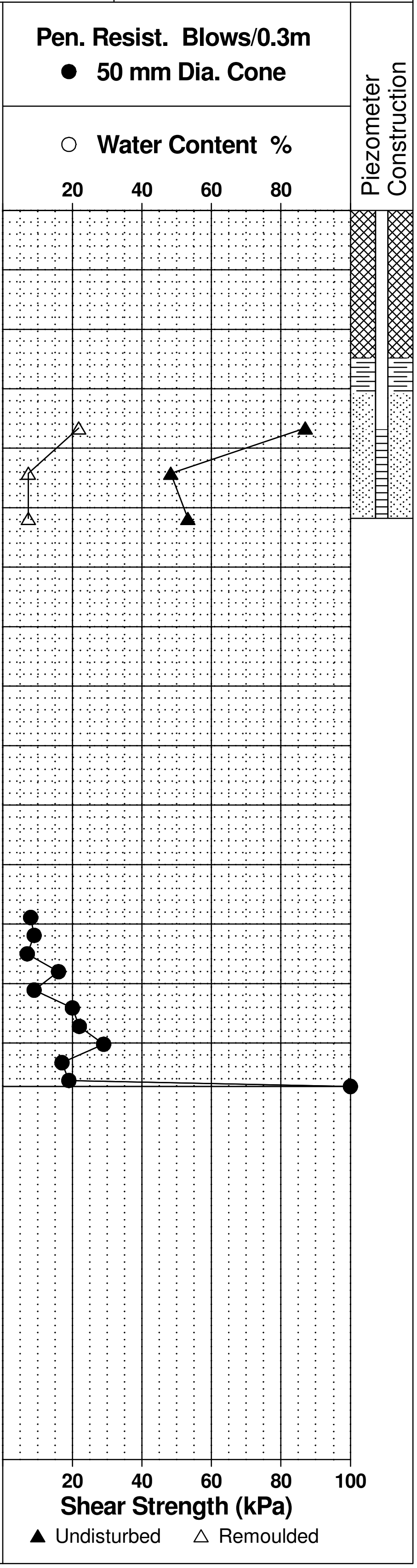
BORINGS BY CME 55 Power Auger

DATE March 22, 2019

FILE NO. PG3968

HOLE NO. BH 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.36
FILL: Brown silty sand with gravel, cobbles and organics		AU	1				
1.20		SS	2	58	9	1	92.36
Stiff, brown **CLAYEY SANDY SILT**		SS	3	75	5	2	91.36
2.30		SS	4	100	3		
Stiff, brown **SILTY CLAY**		SS	5	100	2	3	90.36
- grey by 3.8m depth		SS	6	100	2	4	89.36
5.18		SS	7	100	2	5	88.36
Dynamic Cone Penetration Test commenced at 5.18m depth. Cone pushed to 11.3m depth.						6	87.36
						7	86.36
Inferred **SILTY CLAY**						8	85.36
						9	84.36
						10	83.36
11.30						11	82.36
Inferred **GLACIAL TILL**						12	81.36
						13	80.36
14.73						14	79.36
End of Borehole Practical DCPT refusal at 14.73m depth (Piezometer blocked - March 29, 2019)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 20, 2019

FILE NO. **PG3968**

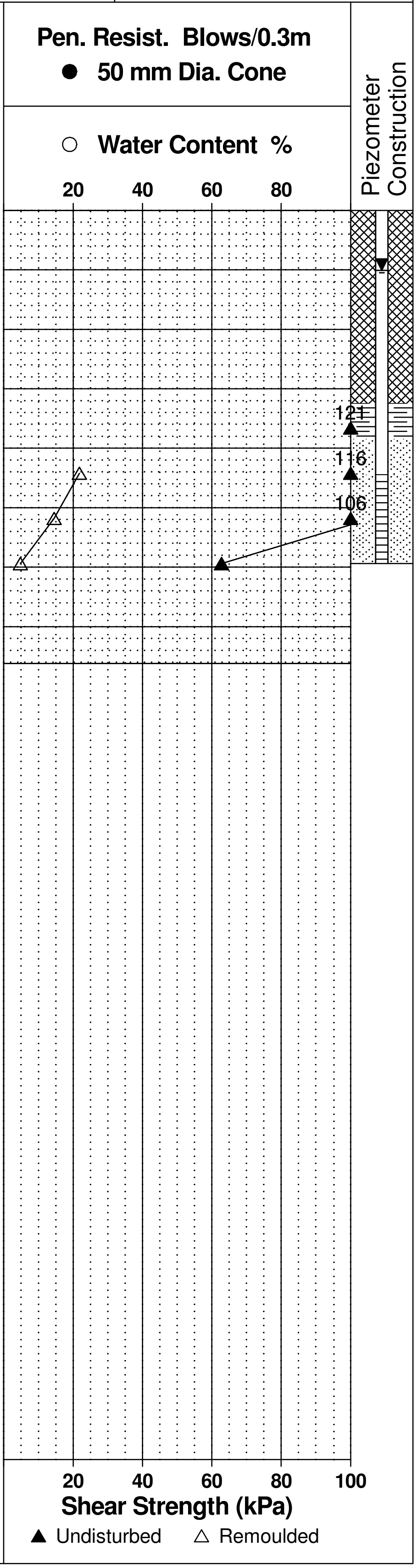
HOLE NO. **BH 4-19**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	94.45
FILL: Brown silty sand, some clay, gravel, cobbles and boulders, trace organics 1.50		AU	1				
		SS	2	67	50+	1	93.45
Brown **CLAYEY SANDY SILT** 2.95		SS	3	38	9	2	92.45
		SS	4	58	14	3	91.45
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth Augered 6m to 7.6m depth. 6.10		SS	5	100	2		
		SS	6	100	2	4	90.45
		SS	7	100	2	5	89.45
		SS	8	100	2	6	88.45
Drill chatter commencing at 6.1m depth. Inferred **GLACIAL TILL** 7.62						7	87.45

End of Borehole

Practical refusal to augering at 7.62m depth on inferred cobble or boulder.

(GWL @ 1.01m - March 29, 2019)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 20, 2019

FILE NO. PG3968

HOLE NO. BH 5-19

SOIL DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand with gravel and clay, some cobbles — 2.30

Brown **CLAYEY SANDY SILT** — 2.95

Very stiff to stiff, brown **SILTY CLAY**

- grey by 5.0m depth

— 5.94

Borehole augered from 5.94 to 17.68m depth.

Inferred **SILTY CLAY** — 10.70

Drill chatter commencing at 10.7m depth

Inferrred **GLACIAL TILL** — 17.68

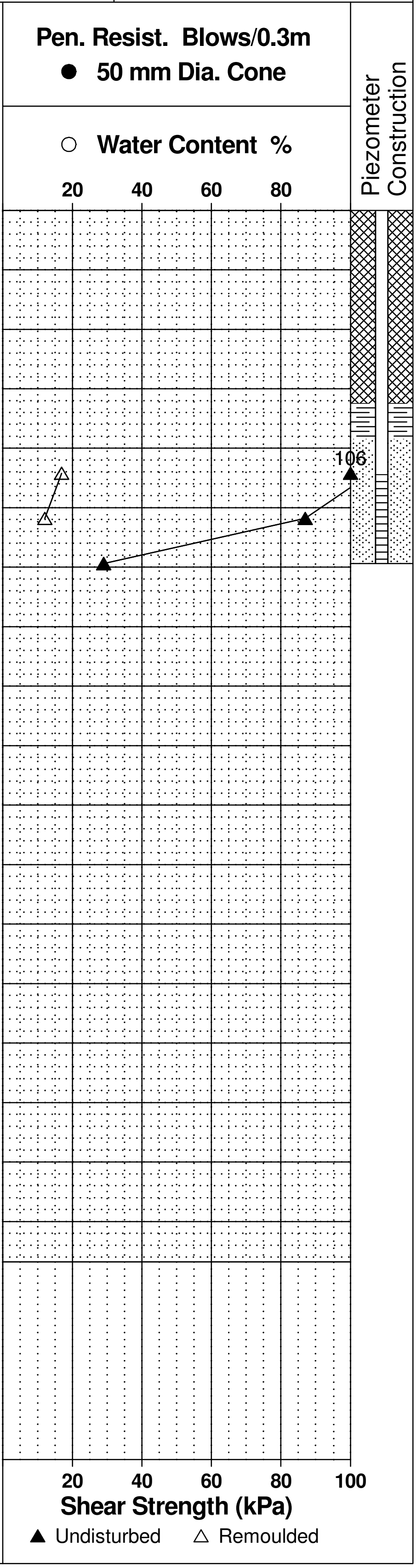
End of Borehole

Practical refusal to augering at 17.68m depth on inferred bedrock.

(Piezometer blocked - March 29, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	67	27
SS	3	50	50+
SS	4	29	11
SS	5	62	7
SS	6	100	2
SS	7	100	2
SS	8	100	2

DEPTH (m)	ELEV. (m)
0	94.39
1	93.39
2	92.39
3	91.39
4	90.39
5	89.39
6	88.39
7	87.39
8	86.39
9	85.39
10	84.39
11	83.39
12	82.39
13	81.39
14	80.39
15	79.39
16	78.39
17	77.39



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed High School - Strandherd Drive
Ottawa, Ontario

DATUM TBM - Top of manhole cover located in front of subject site, along Chapman Mills Drive. Geodetic elevation = 94.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 20, 2019

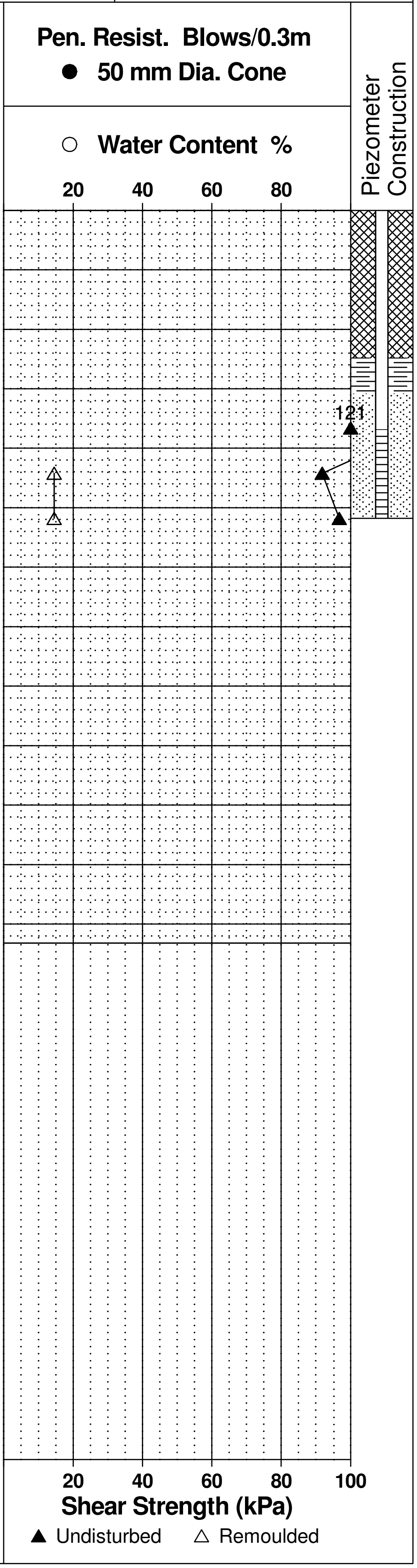
FILE NO. PG3968

HOLE NO. BH 6-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	93.95
FILL: Brown silty sand with gravel, trace clay	AU	1				
1.35	SS	2	21	14	1	92.95
Brown **CLAYEY SANDY SILT,** trace roots	SS	3	29	11		
2.30					2	91.95
Very stiff to stiff, brown **SILTY CLAY**	SS	4	79	6	3	90.95
	SS	5	100	2		
- grey by 3.8m depth	SS	6	38	2	4	89.95
5.18	SS	7	100	2	5	88.95
Borehole augered from 5.18 to 12.32m depth.					6	87.95
					7	86.95
					8	85.95
Inferred **SILTY CLAY**					9	84.95
					10	83.95
11.60					11	82.95
Drill chatter commencing at 11.6m depth Inferred **GLACIAL TILL** 12.32					12	81.95
End of Borehole						

Practical refusal to augering at 12.32m depth on inferred cobble or boulder

(Piezometer sunk below grade - March 29, 2019)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. **PG1984**

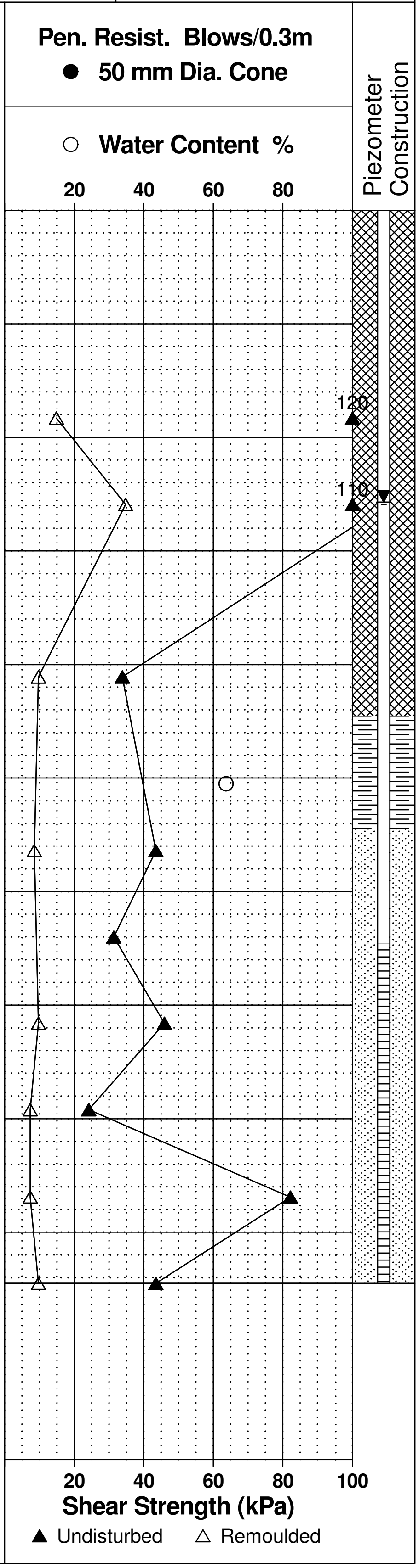
REMARKS

HOLE NO. **BH 2-15**

BORINGS BY CME 75 Power Auger

DATE March 4, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.01
FILL: Brown silty clay, some sand, trace gravel		AU	1				
0.63							
		SS	2	100	10	1	91.01
Very stiff, brown **SILTY CLAY,** some sand seams							
						2	90.01
						3	89.01
3.20							
		TW	3	98			
						4	88.01
		TW	4	92			
						5	87.01
						6	86.01
Stiff to firm, grey **SILTY CLAY**							
						7	85.01
						8	84.01
						9	83.01
9.45							
End of Borehole							
(GWL @ 2.57m-March 19, 2015)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. **PG1984**

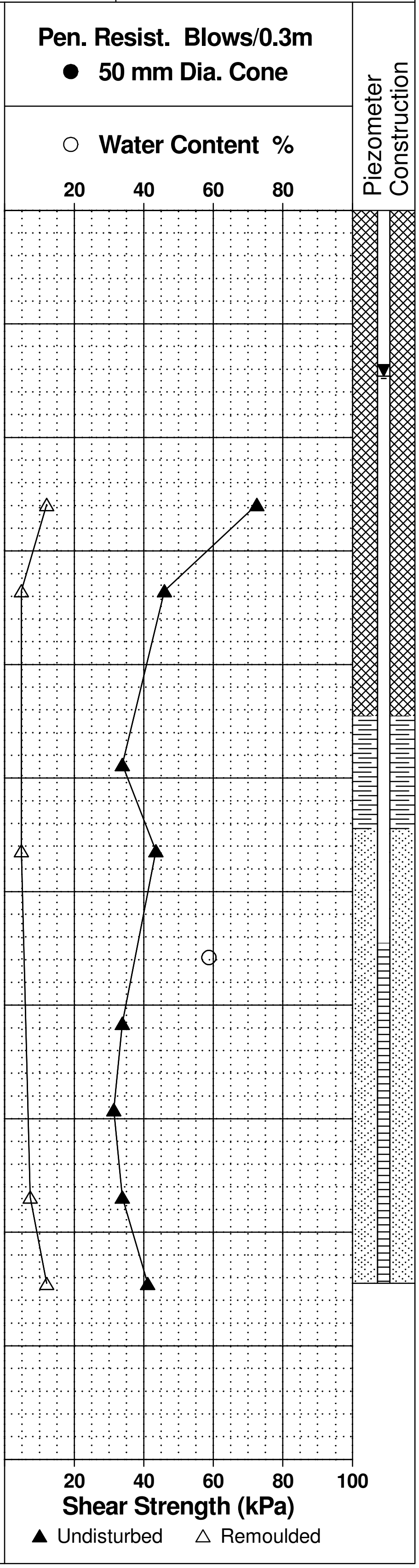
REMARKS

HOLE NO. **BH 3-15**

BORINGS BY CME 75 Power Auger

DATE March 5, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	92.35
FILL: Brown silty clay with sand, trace gravel 0.46		AU	1				
		SS	2	71	7	1	91.35
Very stiff to stiff, brown **SILTY CLAY,** some sand		SS	3	100	3	2	90.35
3.20						3	89.35
		TW	4	96		4	88.35
						5	87.35
Firm, grey **SILTY CLAY**						6	86.35
		TW	5	100			
						7	85.35
						8	84.35
9.45						9	83.35
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 12.95m depth.						10	82.35
						11	81.35



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

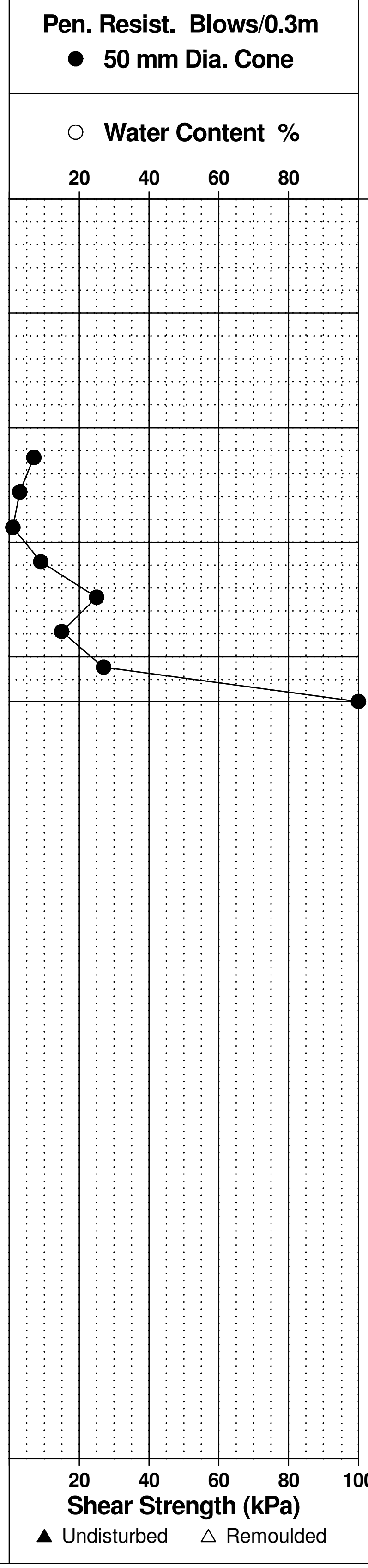
REMARKS

HOLE NO. BH 3-15

BORINGS BY CME 75 Power Auger

DATE March 5, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
						11	81.35	
						12	80.35	
Inferred firm to stiff cohesive soil over loose to compact glacial till								
						13	79.35	
						14	78.35	
						15	77.35	
15.39								
End of Borehole Practical DCPT refusal at 15.39m depth (GWL @ 1.46m-March 19, 2015)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

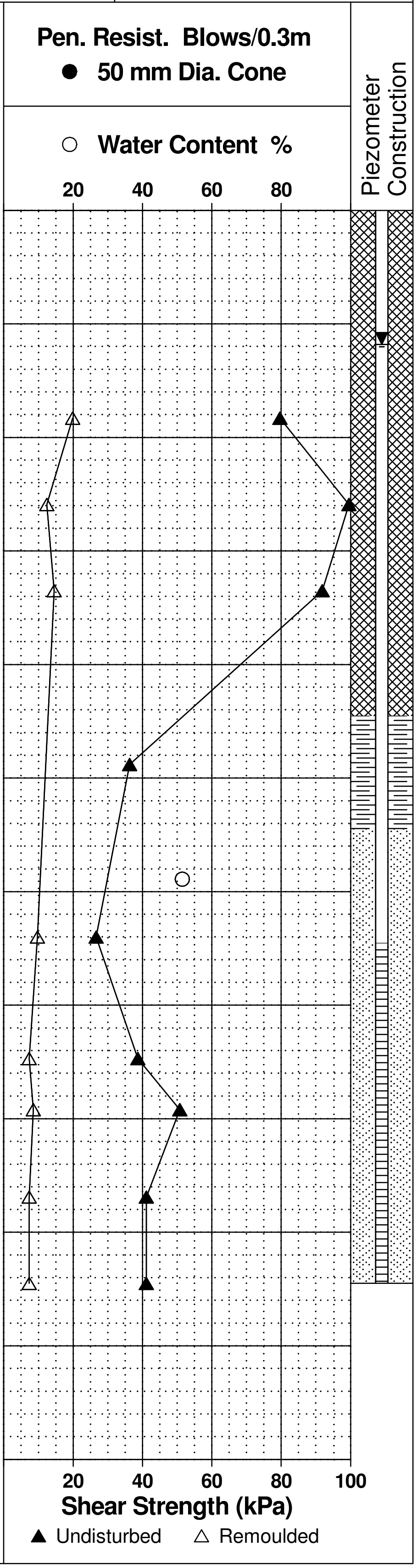
DATE March 5, 2015

FILE NO. PG1984

HOLE NO. BH 4-15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
		AU	1		
		SS	2	100	6
Very stiff to stiff, brown **SILTY CLAY**					
3.40					
		TW	3	100	
		TW	4	92	
Firm to stiff, grey **SILTY CLAY**					
9.45					
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 11.9m depth					

DEPTH (m)	ELEV. (m)
0	91.84
1	90.84
2	89.84
3	88.84
4	87.84
5	86.84
6	85.84
7	84.84
8	83.84
9	82.84
10	81.84
11	80.84



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

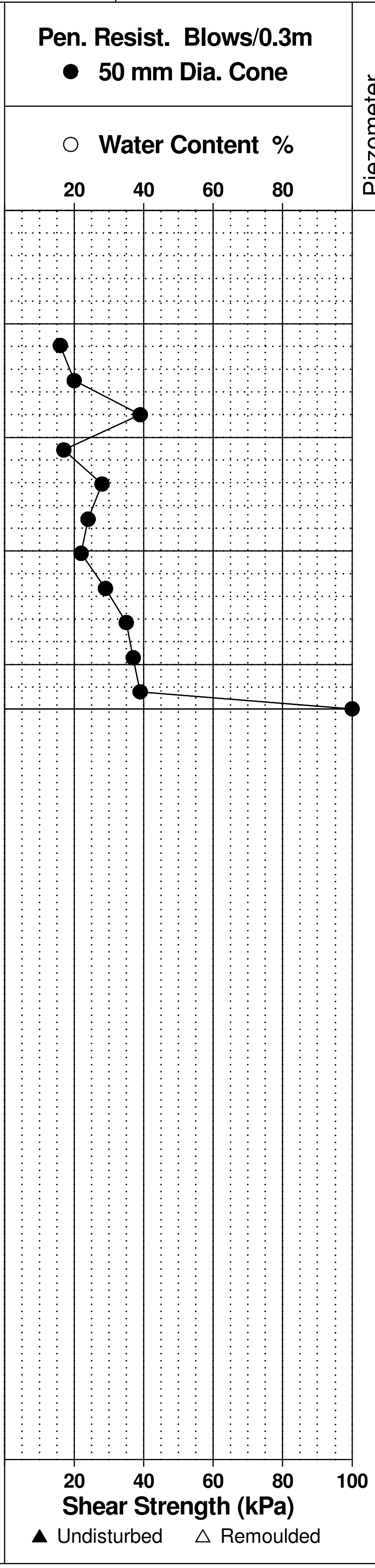
REMARKS

HOLE NO. BH 4-15

BORINGS BY CME 75 Power Auger

DATE March 5, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						11	80.84	
Inferred firm to stiff cohesive soil over compact to dense glacial till						12	79.84	
						13	78.84	
						14	77.84	
15.39						15	76.84	
End of Borehole Practical DCPT refusal at 15.39m depth (GWL @ 1.18m-March 19, 2015)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

REMARKS

HOLE NO. BH 5-15

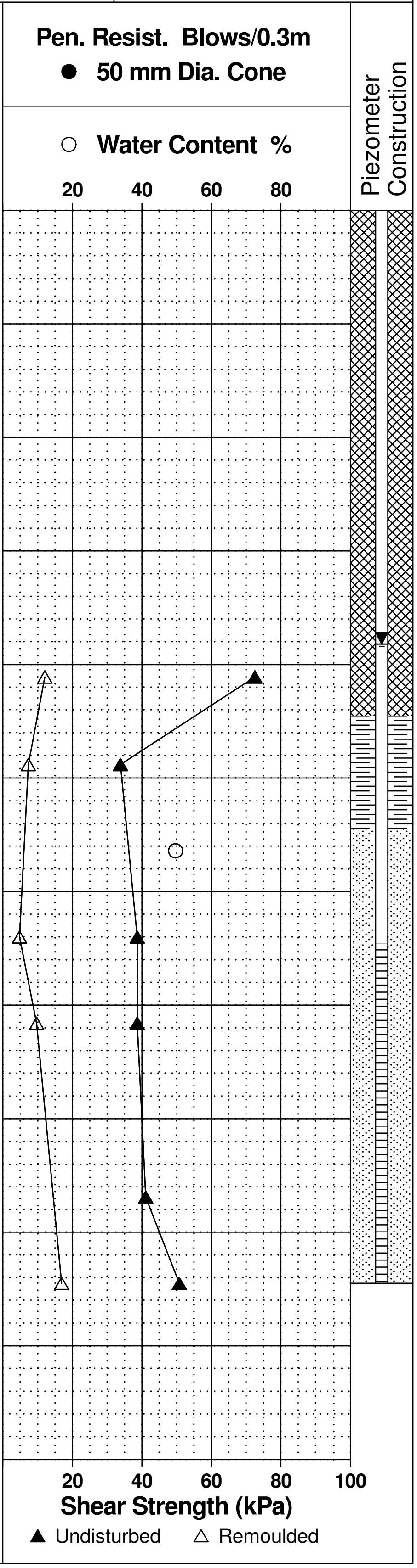
BORINGS BY CME 75 Power Auger

DATE March 6, 2015

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty clay, some sand and cobbles	1.42
Very stiff to stiff, brown **SILTY CLAY,** some sand	4.50
Firm to stiff, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 11.0m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	9
SS	3	100	12
SS	4	100	8
SS	5	100	5
TW	6	97	
TW	7	98	

DEPTH (m)	ELEV. (m)
0	93.44
1	92.44
2	91.44
3	90.44
4	89.44
5	88.44
6	87.44
7	86.44
8	85.44
9	84.44
10	83.44
11	82.44



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

REMARKS

HOLE NO. BH 5-15

BORINGS BY CME 75 Power Auger

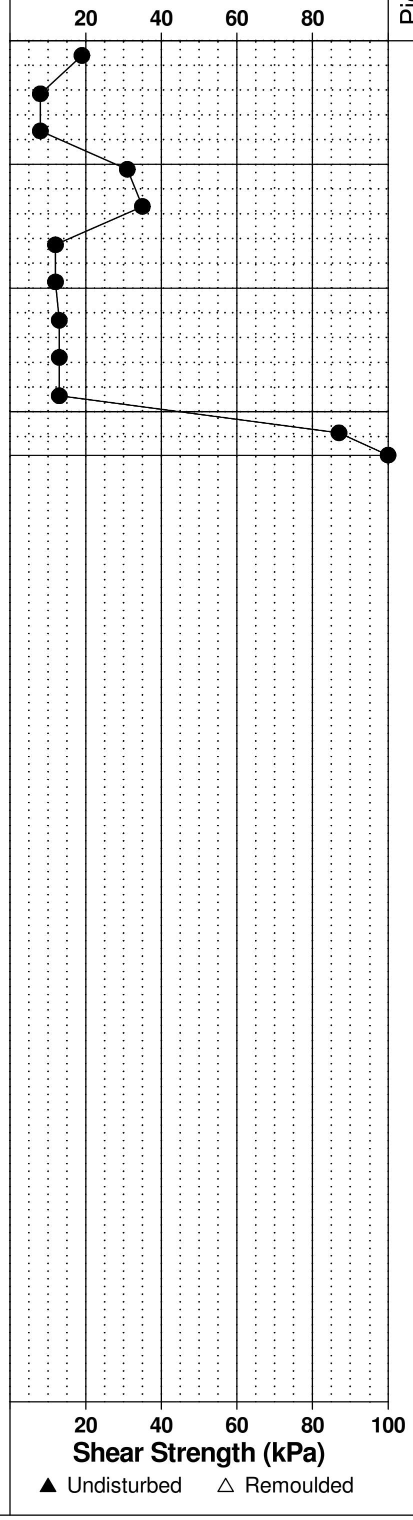
DATE March 6, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
						11	82.44
						12	81.44
Inferred firm to stiff cohesive soil over compact to dense glacial till							
						13	80.44
						14	79.44
14.35							
End of Borehole Practical DCPT refusal at 14.35m depth (GWL @ 3.82m-March 19, 2015)							

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

○ Water Content %

Piezometer Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

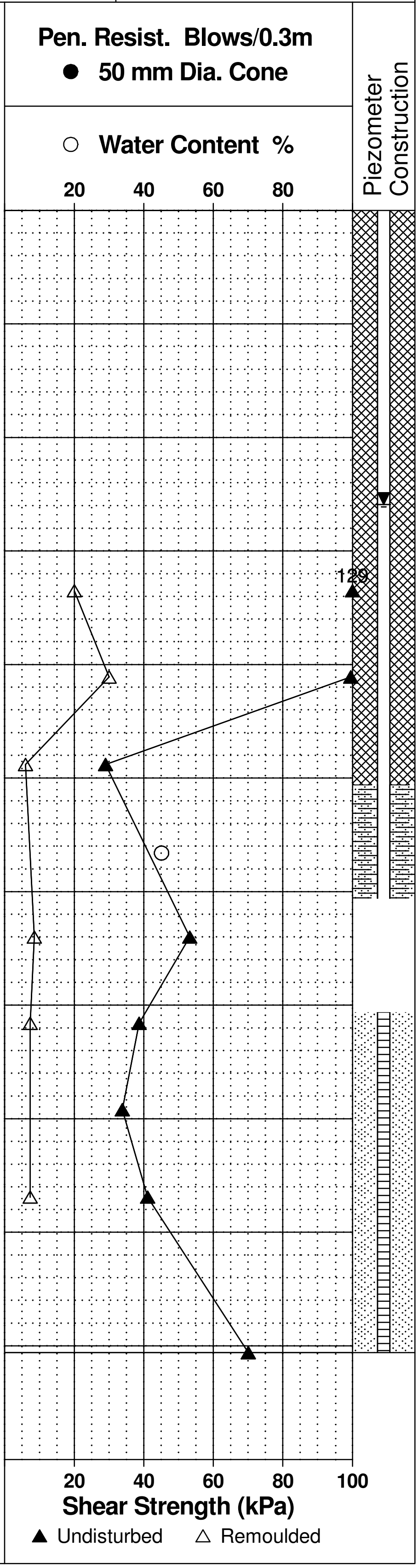
REMARKS

HOLE NO. BH 6-15

BORINGS BY CME 75 Power Auger

DATE March 6, 2015

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	94.30
FILL: Brown silty sand with gravel, trace cobbles	AU	1				
	SS	2	60	50+	1	93.30
1.45						
FILL: Brown silty clay, some sand and gravel	SS	3	71	12		
1.98					2	92.30
Very stiff to stiff, brown **SILTY CLAY,** some sand	SS	4	75	7		
					3	91.30
					4	90.30
4.60						
					5	89.30
Firm, grey **SILTY CLAY**	TW	5	95			
					6	88.30
					7	87.30
					8	86.30
					9	85.30
	TW	6	100			
					10	84.30
10.06						
End of Borehole						
(GWL @ 2.59m-March 19, 2015)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. **PG1984**

REMARKS

HOLE NO. **BH 7-15**

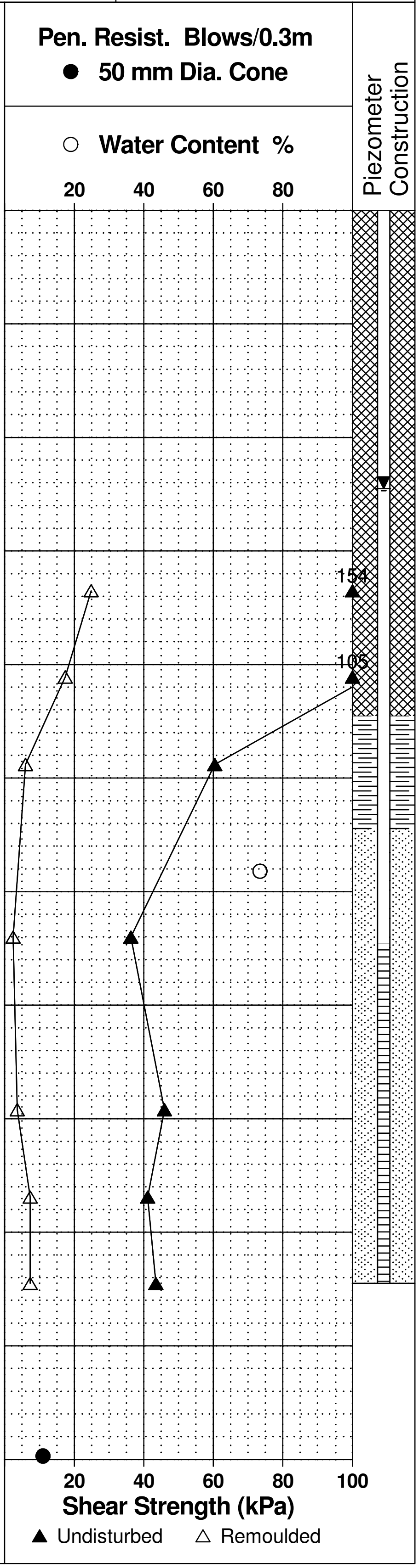
BORINGS BY CME 75 Power Auger

DATE March 9, 2015

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, trace cobbles	1.98
Very stiff to stiff, brown **SILTY CLAY,** some sand	5.00
Firm, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 10.8m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	79
SS	3	75	12
SS	4	100	6
TW	5	96	
TW	6	60	

DEPTH (m)	ELEV. (m)
0	94.05
1	93.05
2	92.05
3	91.05
4	90.05
5	89.05
6	88.05
7	87.05
8	86.05
9	85.05
10	84.05
11	83.05



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

REMARKS

HOLE NO. BH 7-15

BORINGS BY CME 75 Power Auger

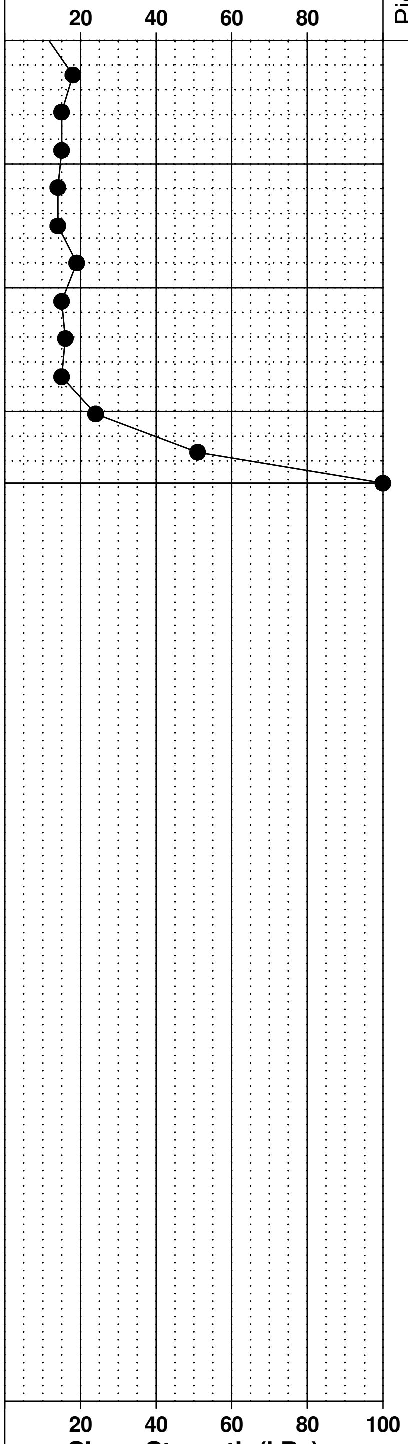
DATE March 9, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						11	83.05
						12	82.05
Inferred firm to stiff cohesive soil over compact to dense glacial till							
						13	81.05
						14	80.05
14.58							
End of Borehole Practical DCPT refusal at 14.58m depth (GWL @ 2.45m-March 19, 2015)							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. **PG1984**

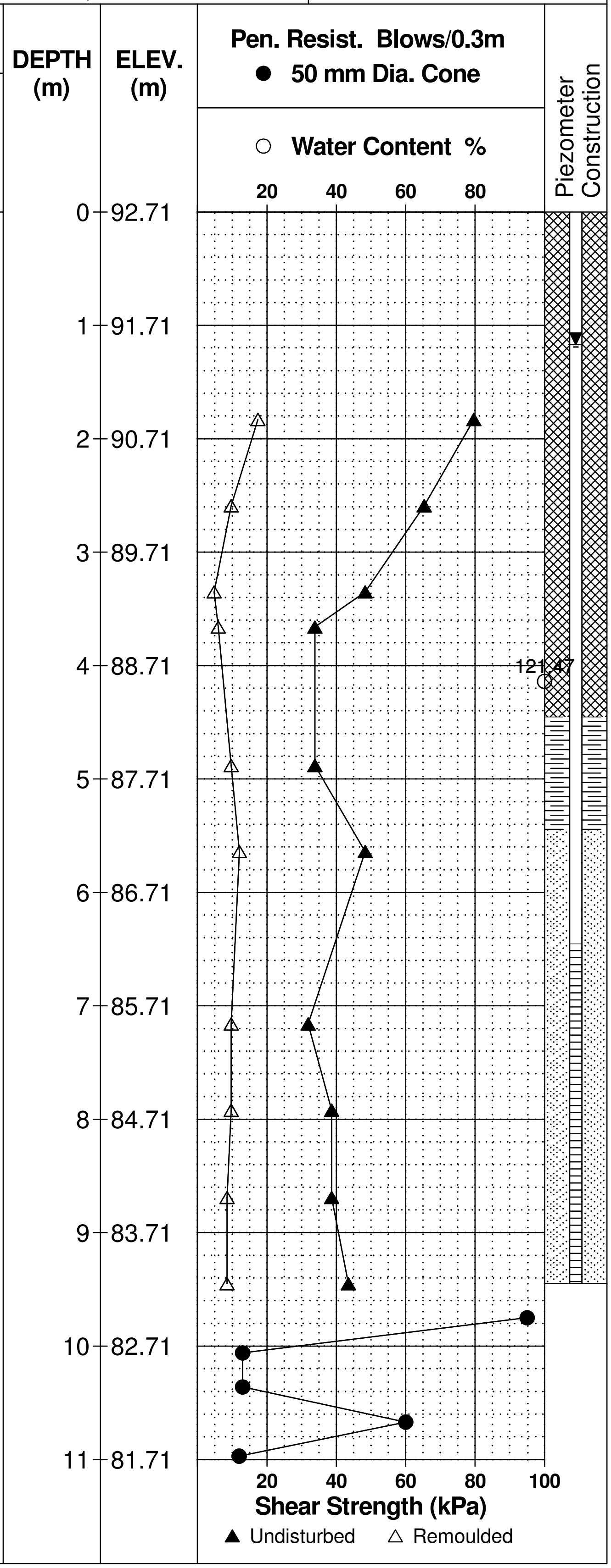
REMARKS

HOLE NO. **BH 8-15**

BORINGS BY CME 75 Power Auger

DATE March 9, 2015

SOIL DESCRIPTION	SAMPLE			
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Very stiff to stiff, brown **SILTY CLAY,** some sand	AU	1		
	SS	2	100	7
3.00				
Firm, grey **SILTY CLAY**	TW	3	93	
	TW	4	98	
9.45				
Dynamic Cone Penetration Test commenced at 9.45m depth.				



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

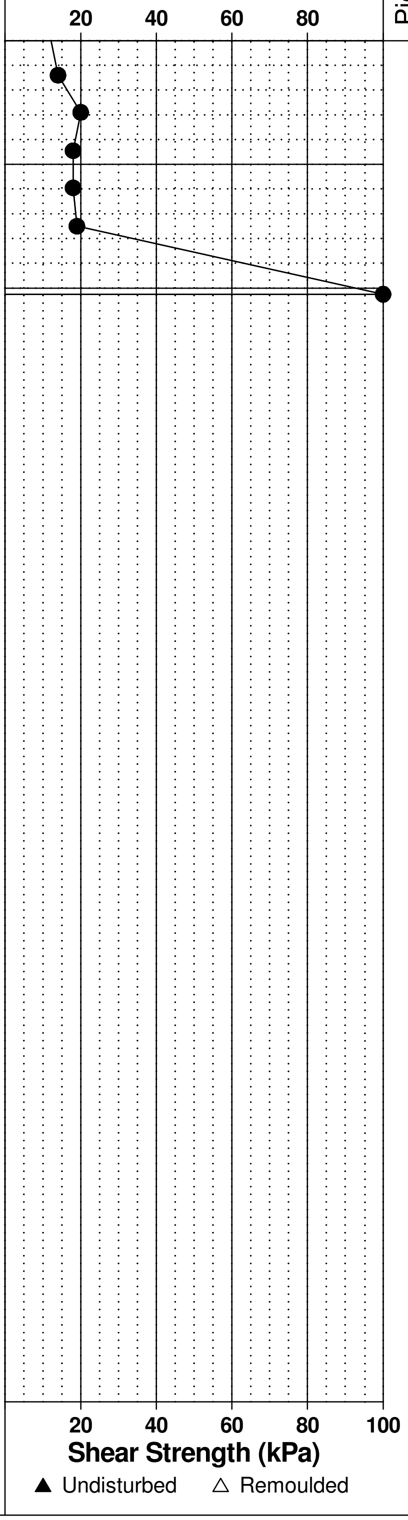
REMARKS

HOLE NO. BH 8-15

BORINGS BY CME 75 Power Auger

DATE March 9, 2015

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						11	81.71		
Inferred firm to stiff cohesive soil over compact to dense glacial till						12	80.71		
13.05						13	79.71		
End of Borehole Practical DCPT refusal at 13.05m depth (GWL @ 1.17m-March 19, 2015)									



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

REMARKS

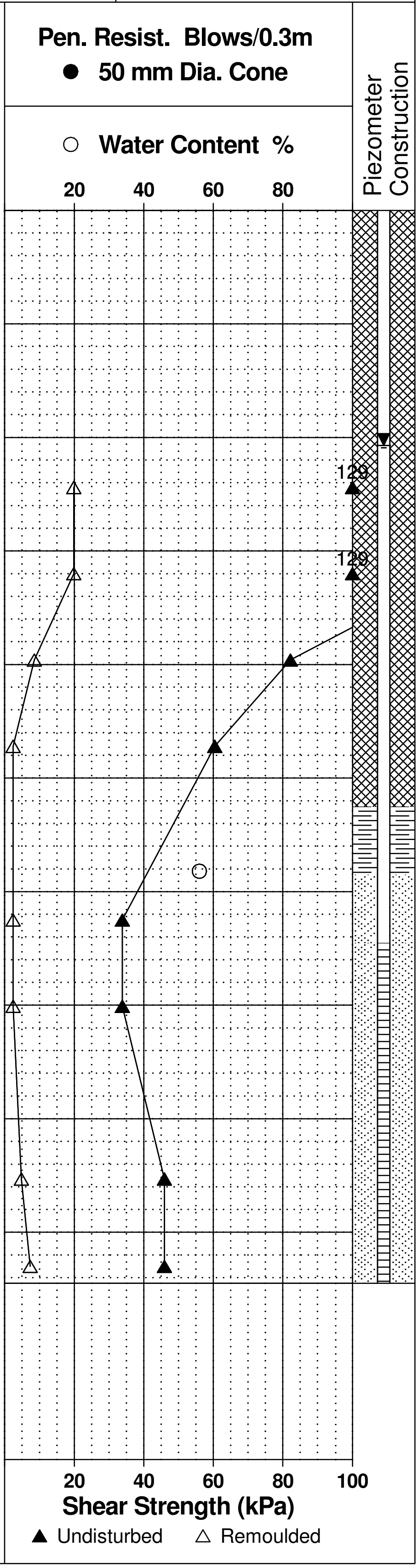
HOLE NO. BH13-16

BORINGS BY CME 55 Power Auger

DATE April 21, 2016

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand, some gravel, cobbles and boulders, trace clay (to 0.60)	AU	1		
FILL: Brown silty clay, some sand, gravel and cobbles (to 1.83)	SS	2	0	9
	SS	3	25	4
Very stiff, brown **SILTY CLAY,** trace sand (to 3.80)	G	4		
Stiff to firm, grey **SILTY CLAY** (to 9.45)	G	5		
	TW	6	100	
	G	7		
	TW	8	100	
	G	9		
End of Borehole				
(GWL @ 2.07m-May 12, 2016)				

DEPTH (m)	ELEV. (m)
0	93.99
1	92.99
2	91.99
3	90.99
4	89.99
5	88.99
6	87.99
7	86.99
8	85.99
9	84.99



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

REMARKS

HOLE NO. BH14-16

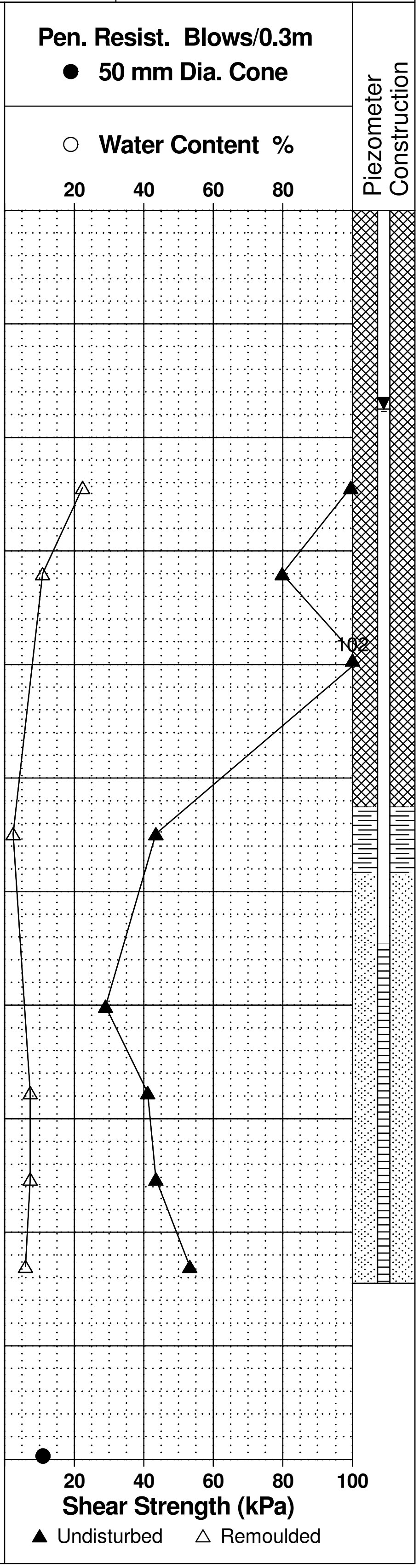
BORINGS BY CME 55 Power Auger

DATE April 21, 2016

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Dark brown sandy silt with gravel, cobbles and organic matter	1.68
Very stiff, brown **SILTY CLAY,** trace sand	4.30
Stiff to firm, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 10.7m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	7
SS	3	62	5
G	4		
G	5		
TW	6	71	
TW	7	79	
G	8		

DEPTH (m)	ELEV. (m)
0	93.49
1	92.49
2	91.49
3	90.49
4	89.49
5	88.49
6	87.49
7	86.49
8	85.49
9	84.49
10	83.49
11	82.49



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

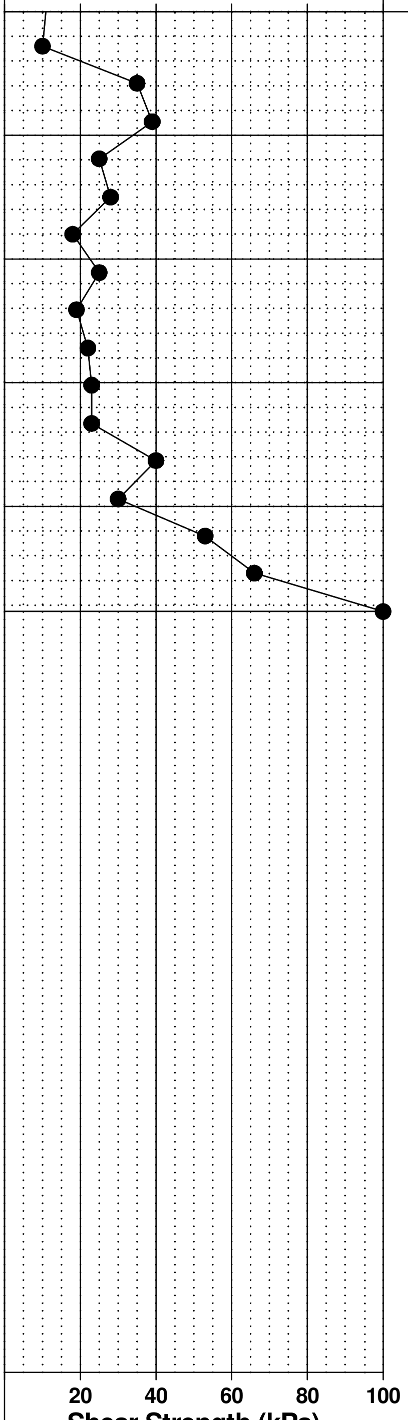
REMARKS

HOLE NO. BH14-16

BORINGS BY CME 55 Power Auger

DATE April 21, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						11	82.49		
						12	81.49		
Inferred stiff cohesive soil over compact to dense glacial till						13	80.49		
						14	79.49		
						15	78.49		
15.85									
End of Borehole Practical DCPT refusal at 15.85m depth (GWL @ 1.75m-May 12, 2016)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

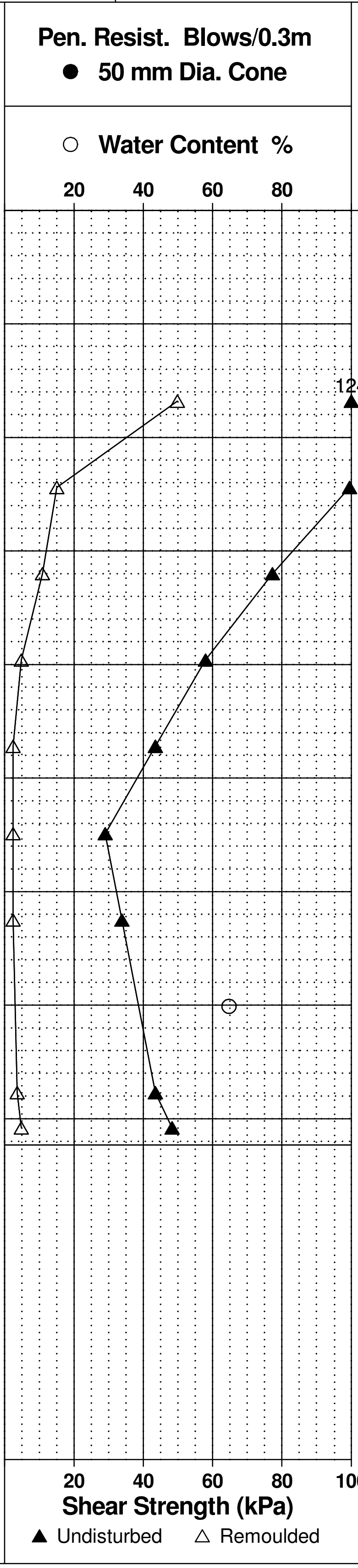
REMARKS

HOLE NO. BH14A-16

BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.49
FILL: Brown silty clay, some gravel and cobbles		AU	1				
		SS	2	25	6	1	92.49
1.68							
Very stiff to stiff, brown **SILTY CLAY,** some sand seams						2	91.49
						3	90.49
3.80							
Stiff to firm, grey **SILTY CLAY**						4	89.49
						5	88.49
						6	87.49
		TW	3	100		7	86.49
						8	85.49
8.23							
End of Borehole							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. **PG1984**

REMARKS

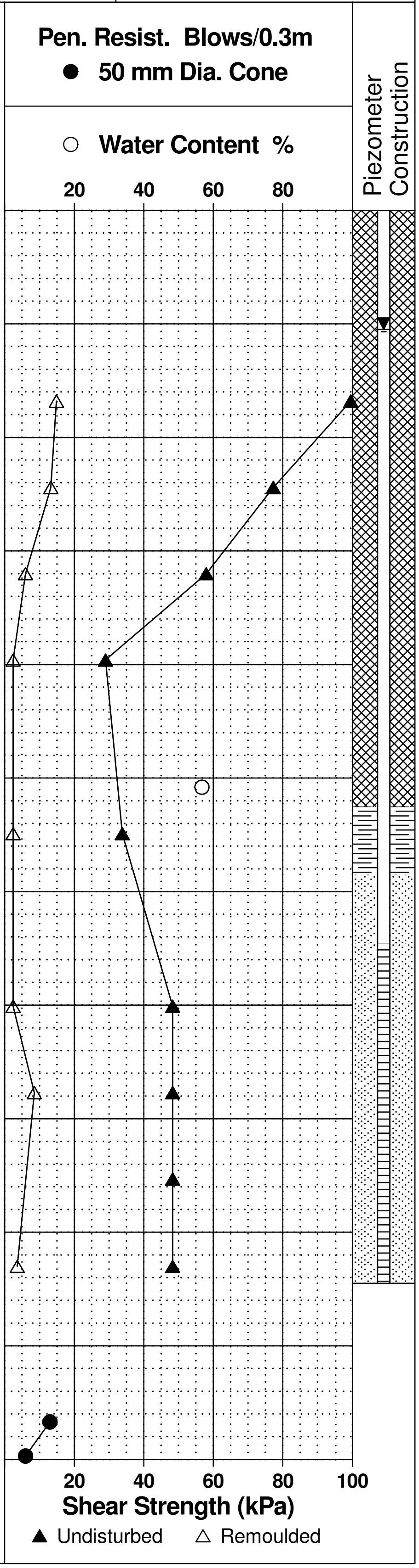
HOLE NO. **BH23-16**

BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** some sand		SS	2	75	5
	3.40	G	3		
Stiff to firm, grey **SILTY CLAY**		TW	4	100	
		TW	5	100	
	9.45	G	6		
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 10.4m depth.					

DEPTH (m)	ELEV. (m)
0	92.29
1	91.29
2	90.29
3	89.29
4	88.29
5	87.29
6	86.29
7	85.29
8	84.29
9	83.29
10	82.29
11	81.29



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Clarke Lands
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by J.L. Richards and Associates Ltd.

FILE NO. PG1984

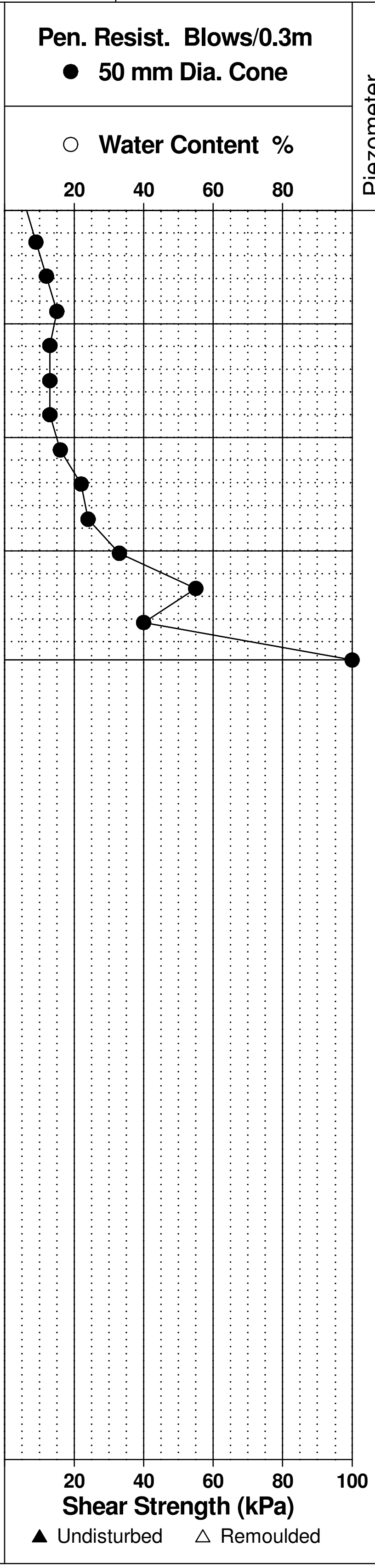
REMARKS

HOLE NO. BH23-16

BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
						11	81.29	
						12	80.29	
Inferred firm to stiff cohesive soil over compact to dense glacial till						13	79.29	
						14	78.29	
14.96								
End of Borehole Practical DCPT refusal at 14.96m depth (GWL @ 1.05m-May 12, 2016)								



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

FILE NO. **PG1984**

REMARKS

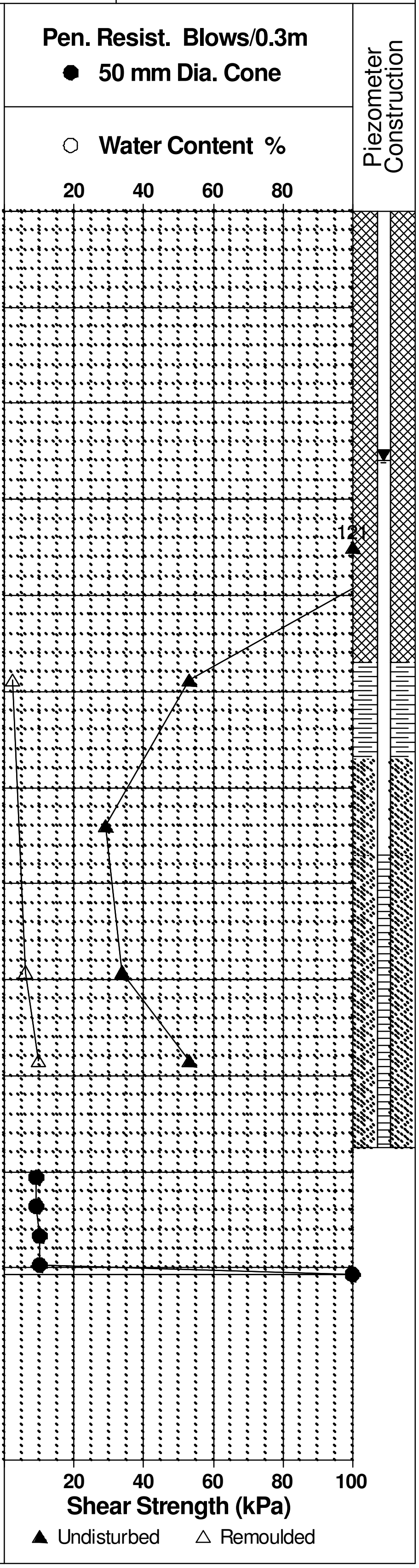
HOLE NO. **BH 7**

BORINGS BY CME 75 Power Auger

DATE 4 February 2011

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel, trace clay	SS	1	100	70
	SS	2	100	11
1.45				
Very stiff to stiff, brown **SILTY CLAY**	SS	3		7
	SS	4		4
	SS	5		W
4.80				
Firm to stiff, grey **SILTY CLAY**	TW	6		
	TW	7		
	TW	8		
9.75				
Dynamic Cone Penetration Test commenced @ 9.75m depth				
11.07				
End of Borehole Practical DCPT refusal @ 11.07m depth (GWL @ 2.60m-Feb. 10/11)				

DEPTH (m)	ELEV. (m)
0	93.96
1	92.96
2	91.96
3	90.96
4	89.96
5	88.96
6	87.96
7	86.96
8	85.96
9	84.96
10	83.96
11	82.96



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

REMARKS

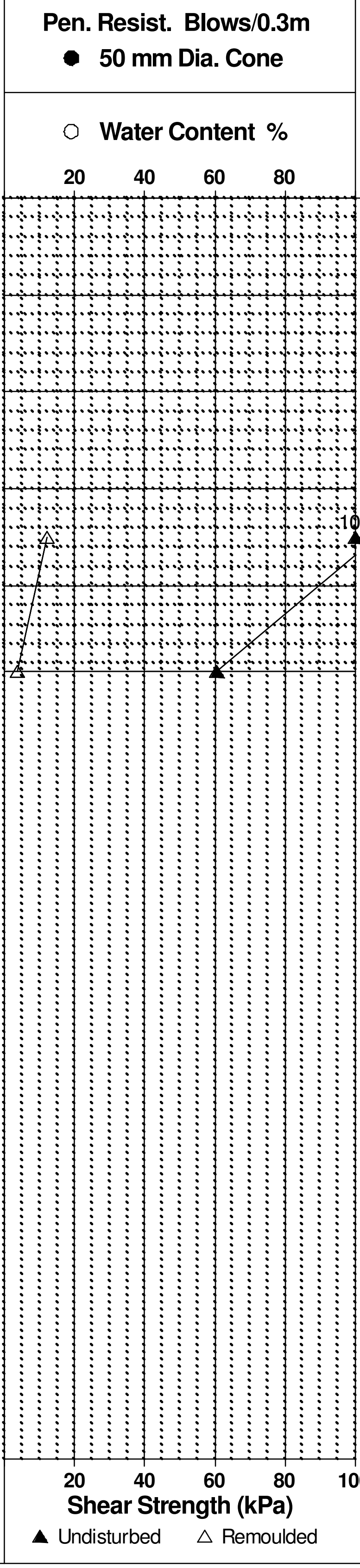
BORINGS BY CME 75 Power Auger

DATE 4 February 2011

FILE NO. PG1984

HOLE NO. BH 7B

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
FILL: Brown silty sand with gravel, trace clay						1		
1.45								
Very stiff to stiff, brown **SILTY CLAY**						2		
						3		
		TW	6			4		
4.88								
End of Borehole								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

FILE NO. **PG1984**

REMARKS

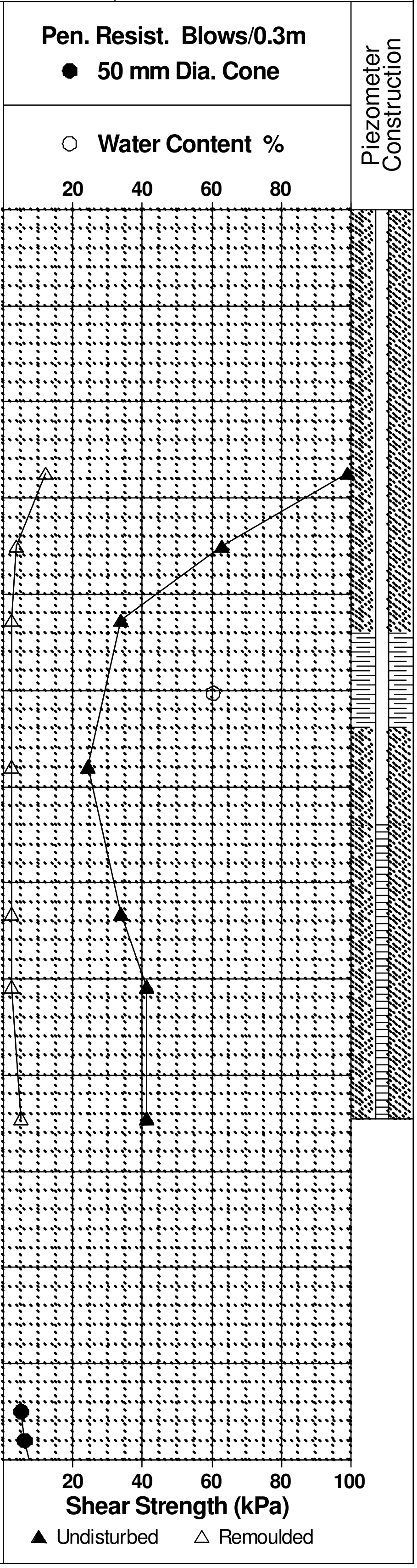
HOLE NO. **BH 8**

BORINGS BY CME 75 Power Auger

DATE 4 February 2011

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Loose to compact, brown **SILTY SAND**	0.60	SS	1	92	30
Very stiff to stiff, brown **SILTY CLAY**	3.60	SS	2	92	7
		SS	3		3
Firm, grey **SILTY CLAY**	9.45	TW	4		
		TW	5		
		TW	6		
Dynamic Cone Penetration Test commenced @ 9.45m depth. Cone pushed to 12.2m depth					

DEPTH (m)	ELEV. (m)
0	92.29
1	91.29
2	90.29
3	89.29
4	88.29
5	87.29
6	86.29
7	85.29
8	84.29
9	83.29
10	82.29
11	81.29
12	80.29
13	79.29



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

FILE NO. PG1984

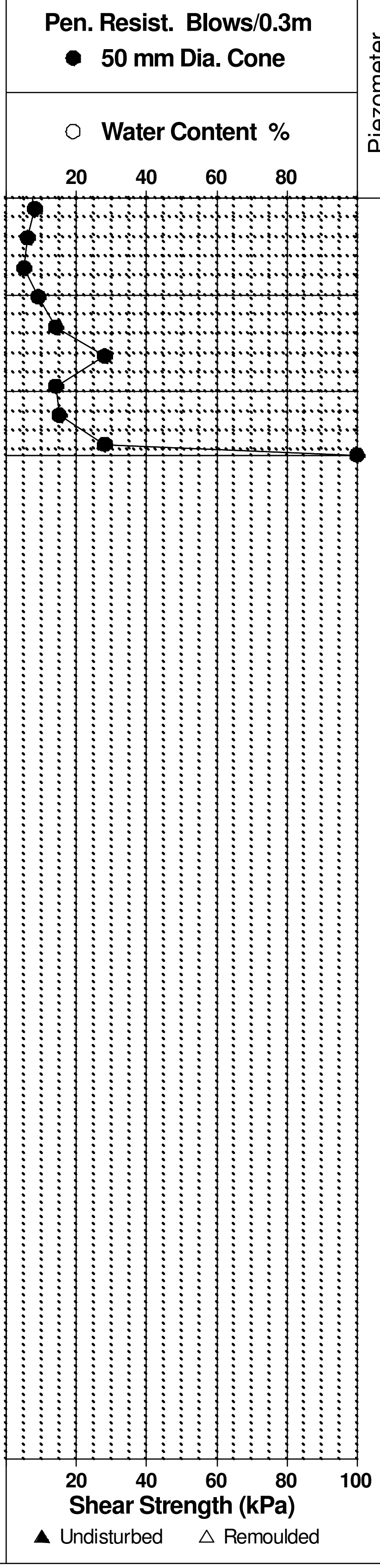
REMARKS

HOLE NO. BH 8

BORINGS BY CME 75 Power Auger

DATE 4 February 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						13	79.29
						14	78.29
						15	77.29
15.65							
End of Borehole Practical DCPT refusal @ 15.65m depth (Standpipe blocked - Feb. 10/11)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

REMARKS

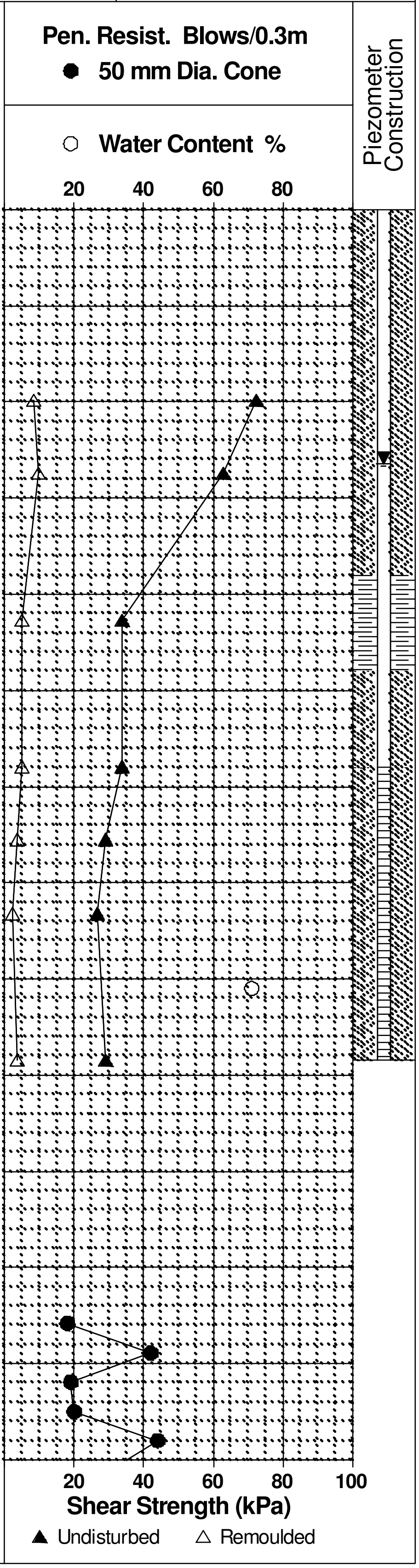
BORINGS BY CME 75 Power Auger

DATE 3 February 2011

FILE NO. PG1984

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	92.06
Compact to loose, brown **SILTY SAND,** trace clay	SS	1		16		
	SS	2		6	1	91.06
1.45						
Very stiff to stiff, brown **SILTY CLAY**					2	90.06
2.90					3	89.06
Frim, grey **SILTY CLAY**	TW	3				
					4	88.06
	TW	4			5	87.06
					6	86.06
					7	85.06
	TW	5			8	84.06
8.84						
Dynamic Cone Penetration Test commenced @ 8.84m depth. Cone pushed to 11.3m depth.					9	83.06
					10	82.06
					11	81.06
					12	80.06
					13	79.06



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

REMARKS

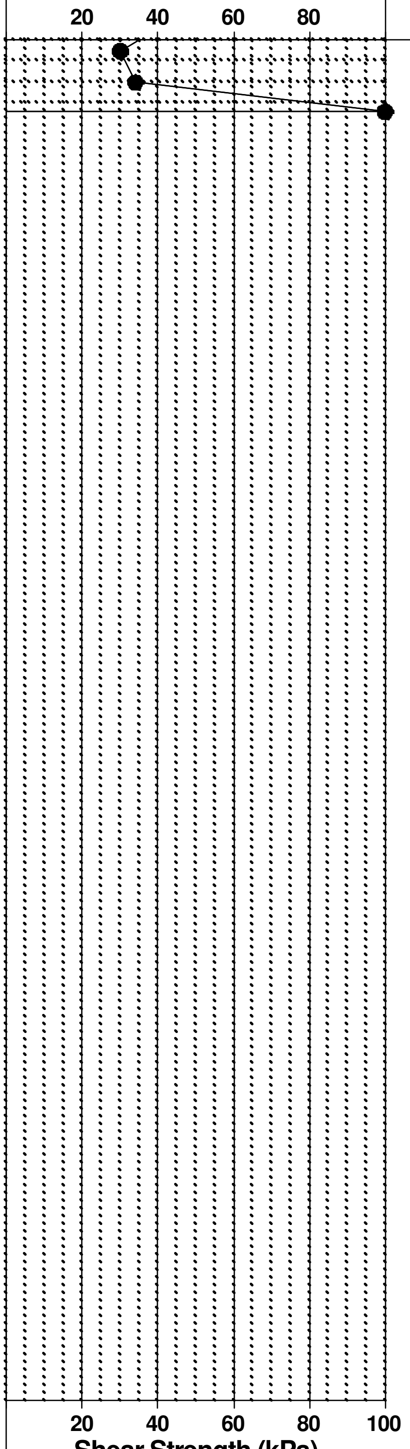
BORINGS BY CME 75 Power Auger

DATE 3 February 2011

FILE NO. PG1984

HOLE NO. BH 9

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									
						13	79.06		
13.69									
End of Borehole Practical DCPT refusal @ 13.69m depth. (GWL @ 2.64m-Feb. 10/11)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

REMARKS

BORINGS BY CME 75 Power Auger

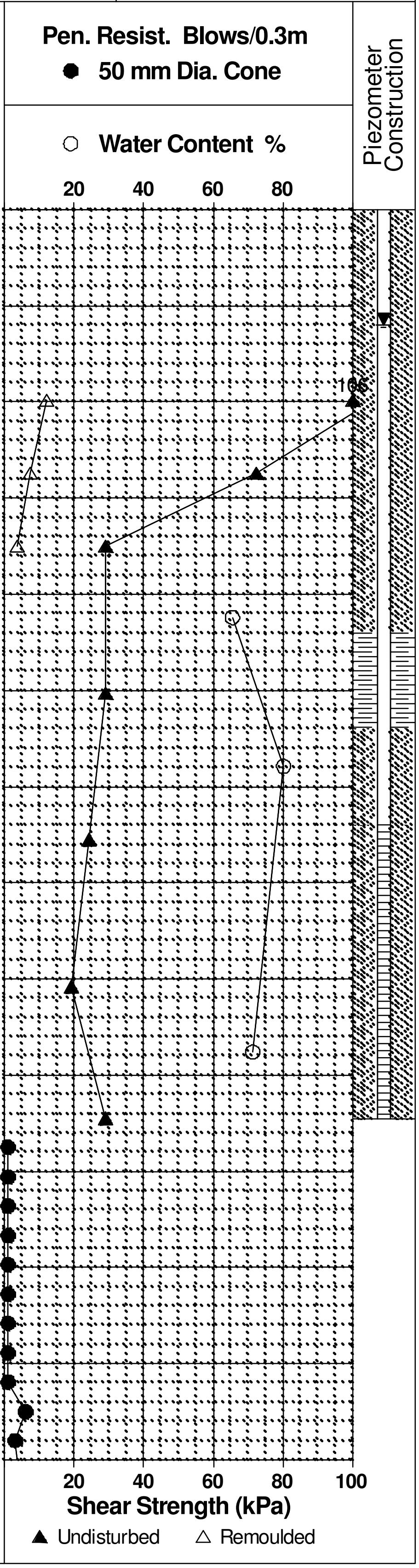
DATE 28 January 2011

FILE NO. PG1984

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Compact, brown **SILTY SAND**		SS	1	92	16
0.60					
Very stiff to stiff, brown **SILTY CLAY**		SS	2		2
2.90					
Firm to soft, grey **SILTY CLAY**		TW	3		
		TW	4		
		TW	5		
9.45					
Dynamic Cone Penetration Test commenced @ 9.45m depth.					

DEPTH (m)	ELEV. (m)
0	92.09
1	91.09
2	90.09
3	89.09
4	88.09
5	87.09
6	86.09
7	85.09
8	84.09
9	83.09
10	82.09
11	81.09
12	80.09
13	79.09



patersongroup **Consulting Engineers**

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Clarke Lands
Ottawa, Ontario

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

FILE NO. PG1984

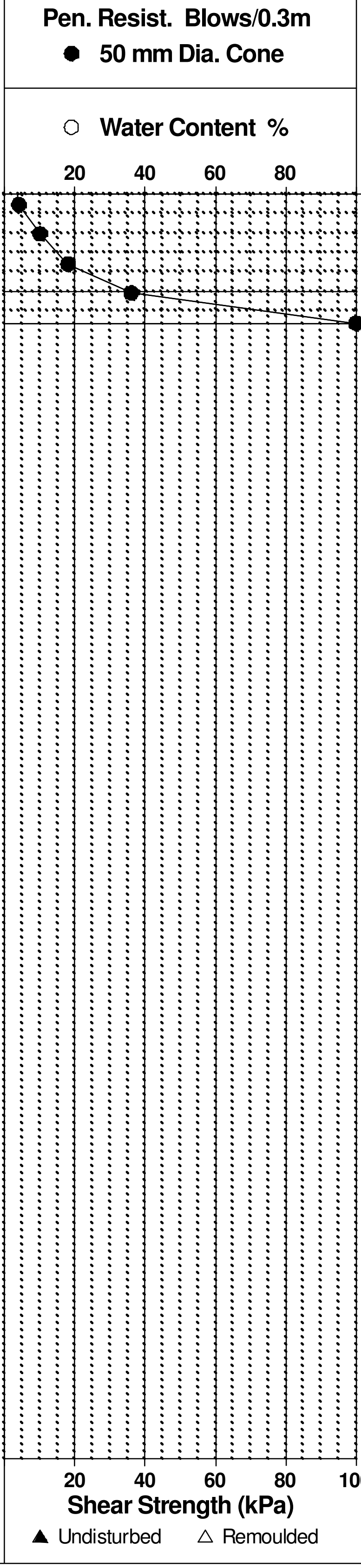
REMARKS

HOLE NO. BH10

BORINGS BY CME 75 Power Auger

DATE 28 January 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						13	79.09
						14	78.09
14.33							
End of Borehole Practical DCPT refusal @ 14.33m depth (GWL @ 1.20m-Feb. 10/11)							



Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the 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)

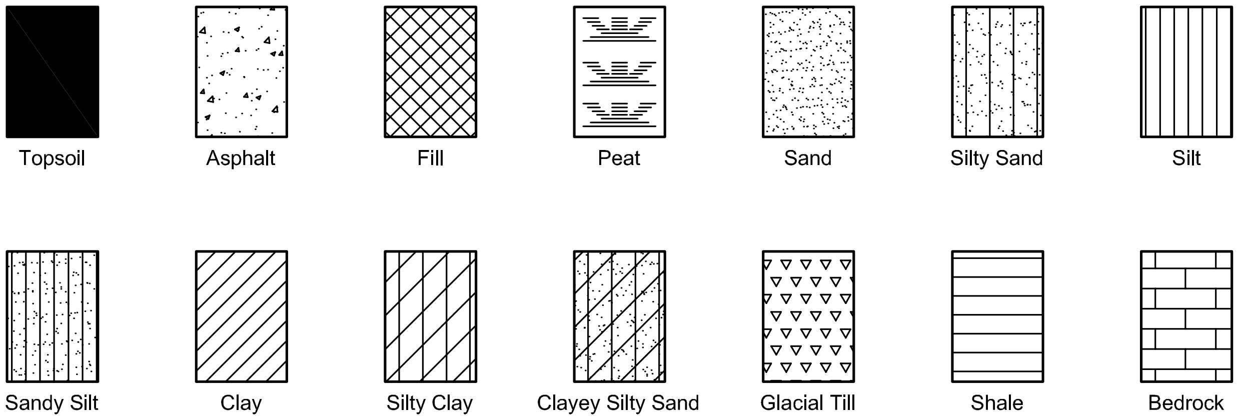
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

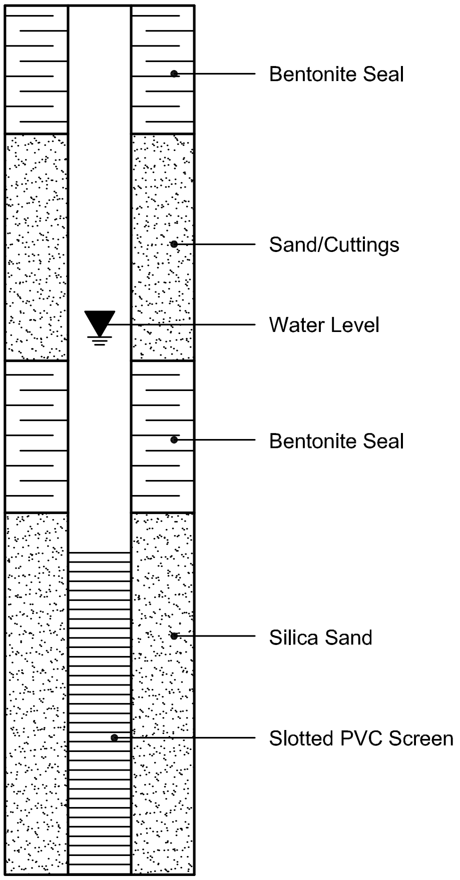
SYMBOLS AND TERMS (continued)

STRATA PLOT



MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

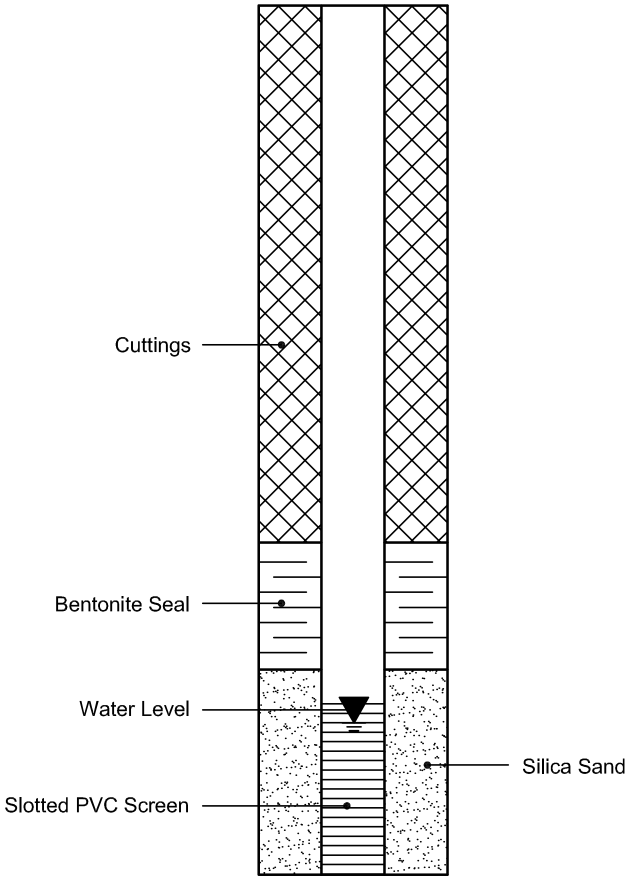


Table 1: Summary of Subsurface Information
CEPEO Barrhaven High School
Strandherd Road at Chapman Mills Drive, Ottawa (Nepean), Ontario

Test Hole Number	Ground Elevation (m)	Inorganic In Situ Soil Surface		Underside of Stiff Clay Crust		Inferred Bedrock Surface Level	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
Current 2019 Investigation Program:							
BH 1-19	94.45	2.20	92.25	5.20	89.25	15.22	79.23
BH 2-19	93.87	1.80	92.07	5.40	88.47	14.00	79.87
BH 3-19	93.36	1.20	92.16	4.40	88.96	14.73	78.63
BH 4-19	94.45	1.50	92.95	5.20	89.25	>7.6	<86.8
BH 5-19	94.39	2.30	92.09	5.50	88.89	17.68	76.71
BH 6-19	93.95	1.35	92.60	N/A	N/A	>12.3	<81.6
Clarke Lands - 2015 Investigation Program:							
BH 2-15	92.01	0.63	91.38	3.60	88.41	>9.5	<82.5
BH 3-15	92.35	0.46	91.89	3.20	89.15	15.39	76.96
BH 4-15	91.84	0.00	91.84	4.20	87.64	15.39	76.45
BH 5-15	93.44	1.42	92.02	4.50	88.94	14.35	79.09
BH 6-15	94.30	1.98	92.32	4.50	89.80	>10.1	<84.2
BH 7-15	94.05	1.98	92.07	5.20	88.85	14.58	79.47
BH 8-15	92.71	0.00	92.71	3.30	89.41	13.05	79.66
Clarke Lands - 2016 Investigation Program:							
BH 13-16	93.99	1.53	92.46	5.00	88.99	>9.5	<84.5
BH 14-16	93.49	1.38	92.11	4.50	88.99	15.85	77.64
BH 23-16	92.29	0.00	92.29	3.40	88.89	14.96	77.33
Clarke Lands - 2011 Investigation Program:							
BH 7-11	93.96	1.45	92.51	5.00	88.96	11.07	82.89
BH 8-11	92.29	0.00	92.29	3.80	88.49	15.65	76.64
BH 9-11	92.06	0.30	91.76	3.40	88.66	13.69	78.37
BH 10-11	92.09	0.30	91.79	3.10	88.99	14.33	77.76

Note:
1. Uppermost inorganic in situ soil surface is the interpreted highest inorganic native soil subgrade level.
2. Inferred bedrock surface is the depth of practical refusal of the dynamic cone penetration test (DCPT) in all applicable boreholes, except auger refusal in BH 5-19. Practical auger refusal was obtained in BHs 1-19, 2-19, 4-19 and 6-19, but is not inferred to be on bedrock.

Table 2: Summary of Consolidation Test Results
CEPEO Barrhaven High School
Strandherd Road at Chapman Mills Drive, Ottawa (Nepean), Ontario

Sample No.	Ground Elev. (m)	Depth (m)	Elevation (m)	p'_c (kPa)	p'_o (kPa)	O.C. (kPa)	Ccr	Cc	W.C. (%)	Sample Quality
Clarke Lands - 2015 Testing Program:										
BH2-15-TW4	92.01	5.05	86.96	115	56	59	0.019	1.415	64	A
BH3-15-TW5	92.35	6.58	85.77	98	68	30	0.019	1.002	59	A
BH4-15-TW4	91.84	5.89	85.95	110	67	43	0.017	0.973	52	A
BH5-15-TW6	93.44	5.64	87.80	114	56	58	0.015	1.105	50	A
BH6-15-TW5	94.30	5.66	88.64	101	53	48	0.020	0.890	45	A
BH7-15-TW5	94.05	5.82	88.23	96	53	43	0.038	1.321	73	A - P
BH8-15-TW3	92.71	4.14	88.57	107	55	52	0.026	1.878	121	A
Clarke Lands - 2016 Testing Program:										
BH13-16-TW6	93.99	5.82	88.17	67	56	11	0.024	0.738	56	D
BH14-16-TW3	93.49	7.01	86.48	87	66	21	0.035	1.101	65	P
BH23-16-TW4	92.29	5.08	87.21	137	60	77	0.023	1.366	57	A
Clarke Lands - 2011 Testing Program:										
BH7-11-TW7	93.96	7.31	86.65	100	65	35	0.016	1.199	61	P
BH8-11-TW4	92.29	5.03	87.26	107	67	40	0.014	1.076	60	A
BH8-11-TW5	92.29	6.60	85.69	92	70	22	0.028	2.856	79	P
BH9-11-TW4	92.06	5.03	87.03	100	60	40	0.014	0.656	45	A
BH9-11-TW5	92.06	8.10	83.96	112	79	33	0.023	1.717	71	P
BH10-11-TW3	92.09	4.24	87.85	90	55	35	0.025	0.853	66	A - P
BH10-11-TW4	92.09	5.79	86.30	88	65	23	0.026	1.898	80	A - P
BH10-11-TW5	92.09	8.76	83.33	78	83	-5	0.020	1.835	71	D

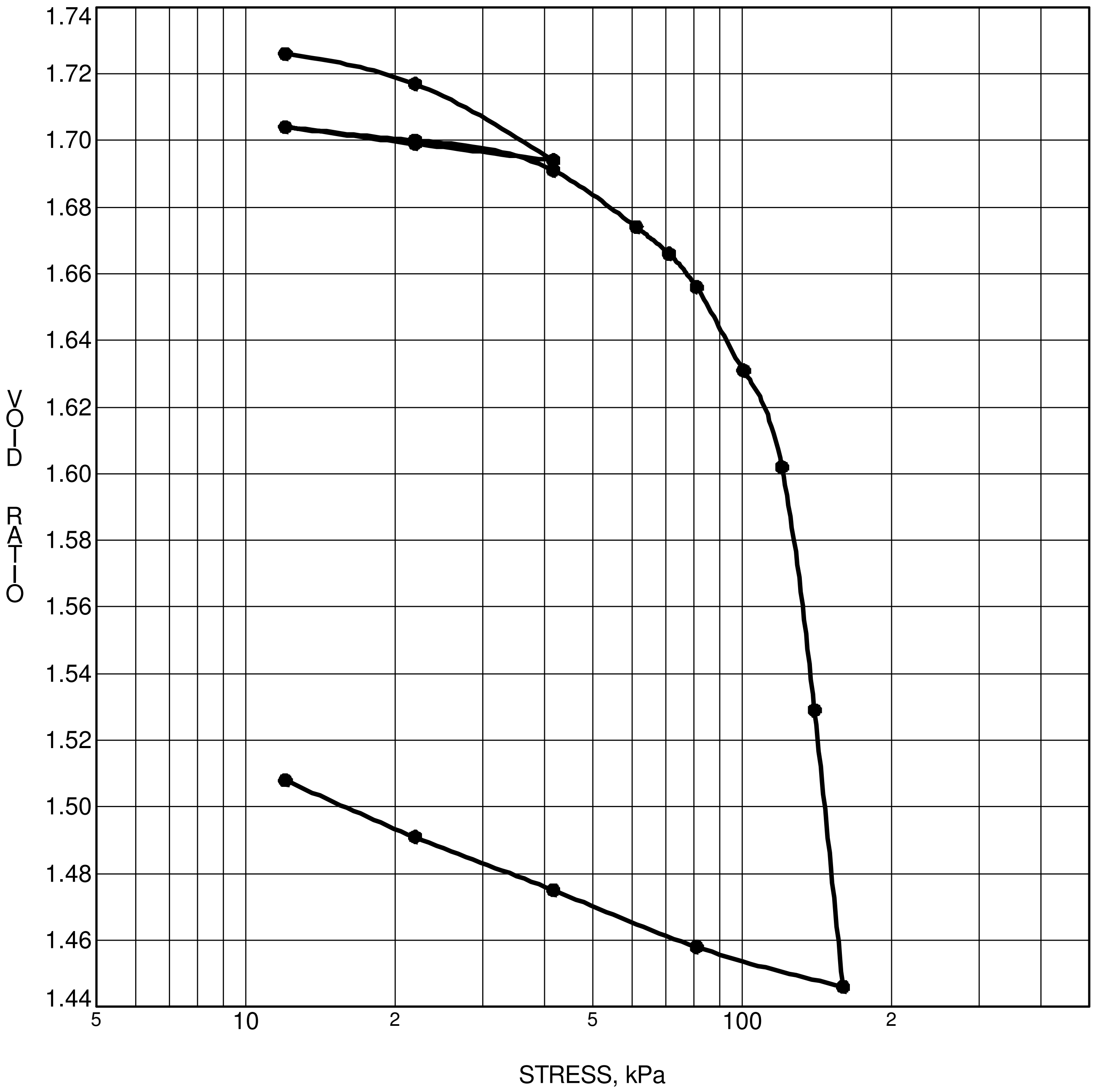
Notes:

1. Effective overburden pressure, p'_o, is based on an average crust thickness of 3.0 m, an estimated long-term low groundwater depth of 2.5 m and a mean unit weight for the grey clay of 16.0 kN/m^3. The original ground elevation is used for p'_o estimates.
2. The last column presents the quality assessment of the test sample: A = Acceptable P = Poor (Likely Disturbed) D = Disturbed

Table 3: Summary of Groundwater Levels
CEPEO Barrhaven High School
Strandherd Road at Chapman Mills Drive, Ottawa (Nepean), Ontario

Borehole Number	Ground Elevation (m)	Measured Groundwater Level (m)		Recording Date
		Depth	Elevation	
Current 2019 Investigation Program:				
BH 1-19	94.45	1.30	93.15	March 29, 2019
BH 2-19	93.87	1.20	92.67	March 29, 2019
BH 3-19	93.36	Standpipe Blocked	N/A	March 29, 2019
BH 4-19	94.45	1.01	93.44	March 29, 2019
BH 5-19	94.39	Standpipe Blocked	N/A	March 29, 2019
BH 6-19	93.95	Standpipe Sunk	N/A	March 29, 2019
Clarke Lands - 2015 Investigation Program:				
BH 2-15	92.01	2.57	89.44	March 19, 2015
BH 3-15	92.35	1.46	90.89	March 19, 2015
BH 4-15	91.84	1.18	90.66	March 19, 2015
BH 5-15	93.44	3.82	89.62	March 19, 2015
BH 6-15	94.30	2.59	91.71	March 19, 2015
BH 7-15	94.05	2.45	91.60	March 19, 2015
BH 8-15	92.71	1.17	91.54	March 19, 2015
Clarke Lands - 2016 Investigation Program:				
BH 13-16	93.99	2.07	91.92	May 12, 2016
BH 14-16	93.49	1.75	91.74	May 12, 2016
BH 23-16	92.29	1.05	91.24	May 12, 2016
Clarke Lands - 2011 Investigation Program:				
BH 7-11	93.96	2.60	91.36	February 10, 2011
BH 9-11	92.06	2.64	89.42	February 10, 2011
BH 10-11	92.09	1.20	90.89	February 10, 2011

Notes:
1. The groundwater levels were recorded in the standpipe tubing on the date noted for the applicable investigation stage.
2. The reading in BH 5-15 was low, compared to the depth of the base of the stiff clay crust, and is assumed to have not stabilized by the time of reading.

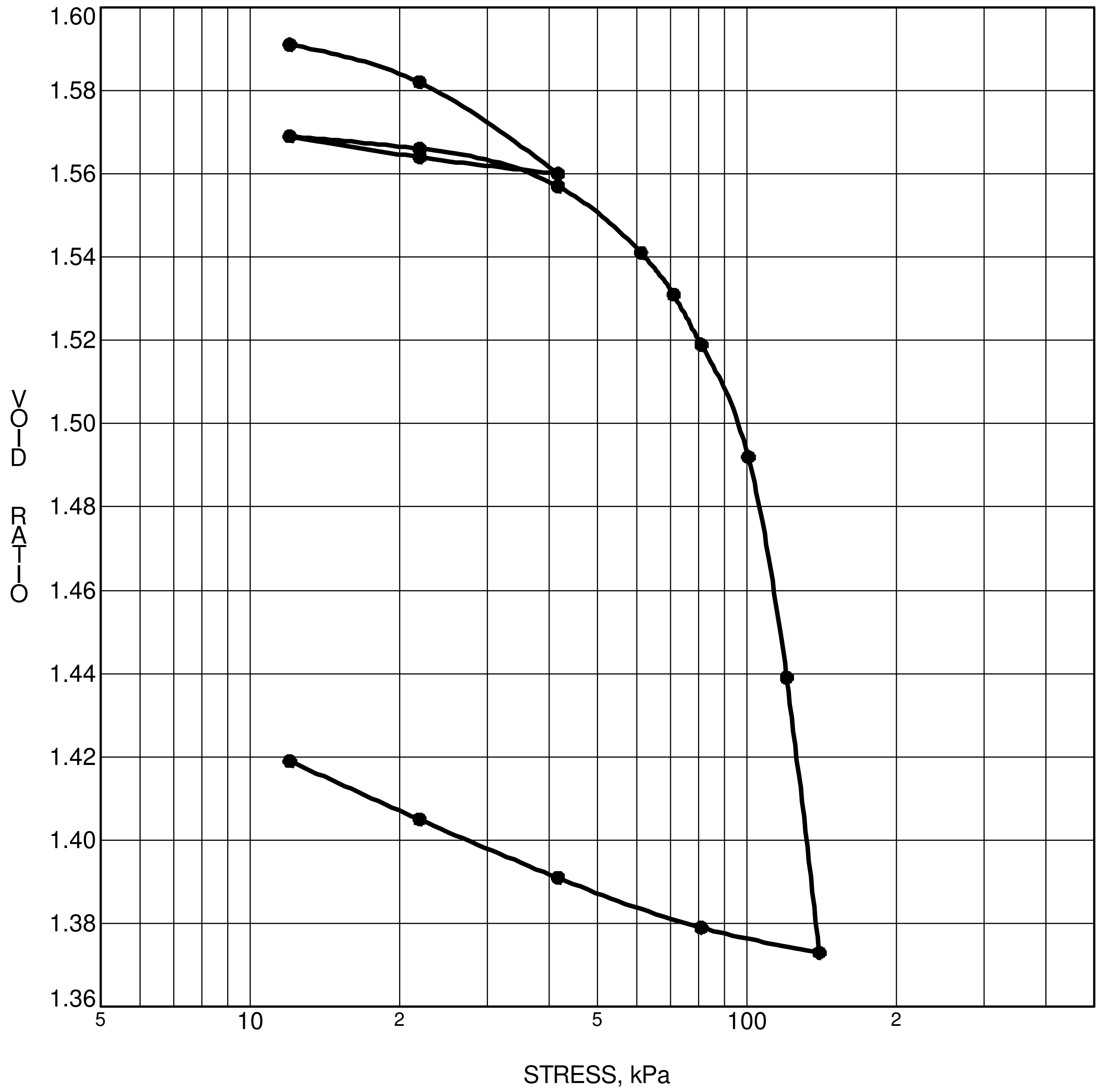


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2-15**	p'$_o$	**56** kPa	Ccr	**0.190**
Sample No.	**TW 4**	p'$_c$	**115** kPa	Cc	**1.415**
Sample Depth	**5.05** m	OC Ratio	**2.1**	Wo	**63.7** %
Sample Elev.	**86.96** m	Void Ratio	**1.751**	Unit Wt.	**16.1** kN/m^3

CLIENT **Minto Communities Inc.**
FILE NO. **PG1984**
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**
DATE **12/03/2015**

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

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3-15**	p'_o	**68** kPa	Ccr	**0.019**
Sample No.	**TW 5**	p'_c	**98** kPa	Cc	**1.002**
Sample Depth	**6.58** m	OC Ratio	**1.4**	Wo	**58.7** %
Sample Elev.	**85.77** m	Void Ratio	**1.615**	Unit Wt.	**16.4** kN/m^3

CLIENT **Minto Communities Inc.**

PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**

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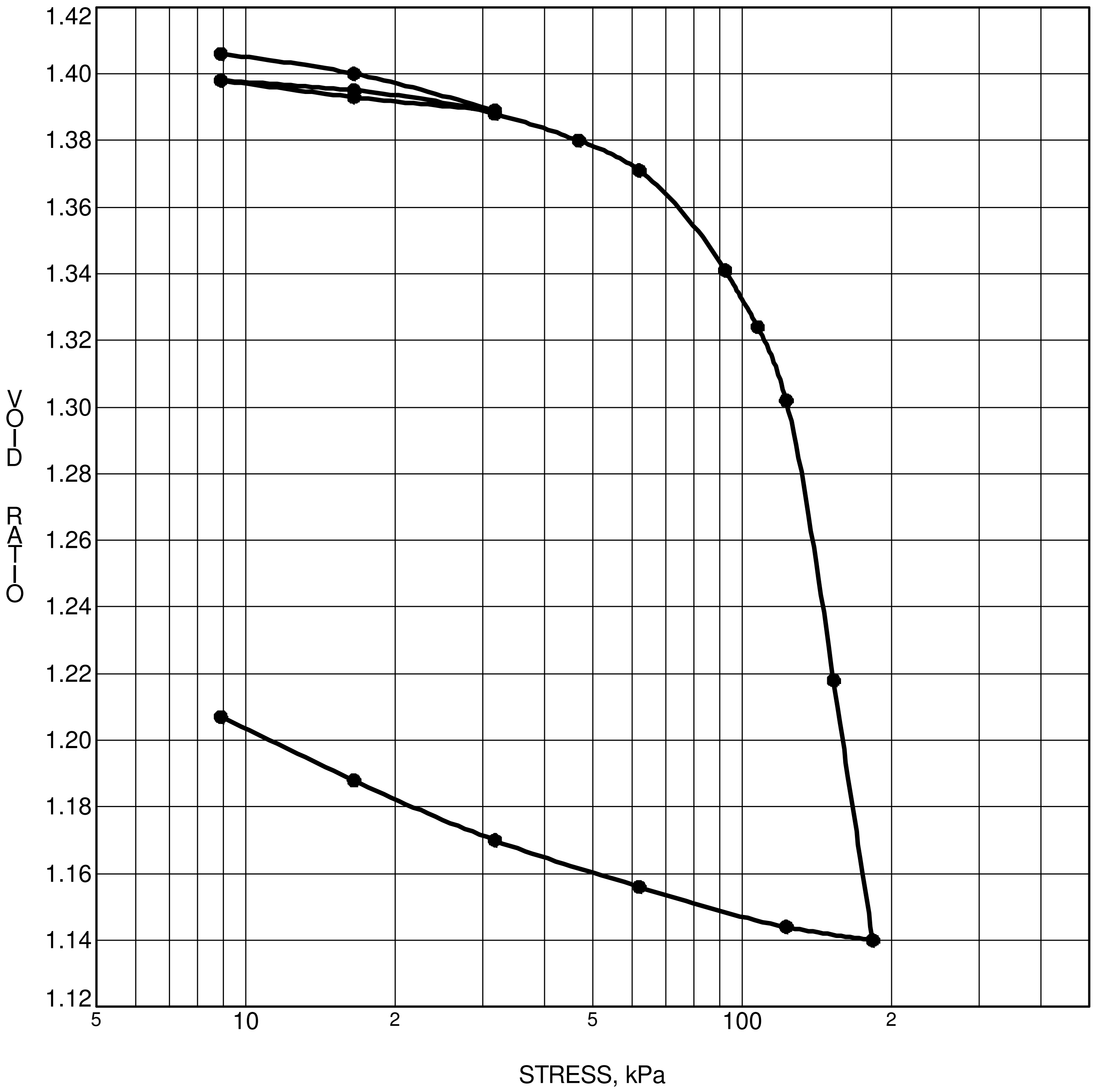
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 4-15**	p'o	**67** kPa	Ccr	**0.017**
Sample No.	**TW 4**	p'c	**110** kPa	Cc	**0.973**
Sample Depth	**5.89** m	OC Ratio	**1.6**	Wo	**51.5** %
Sample Elev.	**85.95** m	Void Ratio	**1.416**	Unit Wt.	**17.0** kN/m ^{3}

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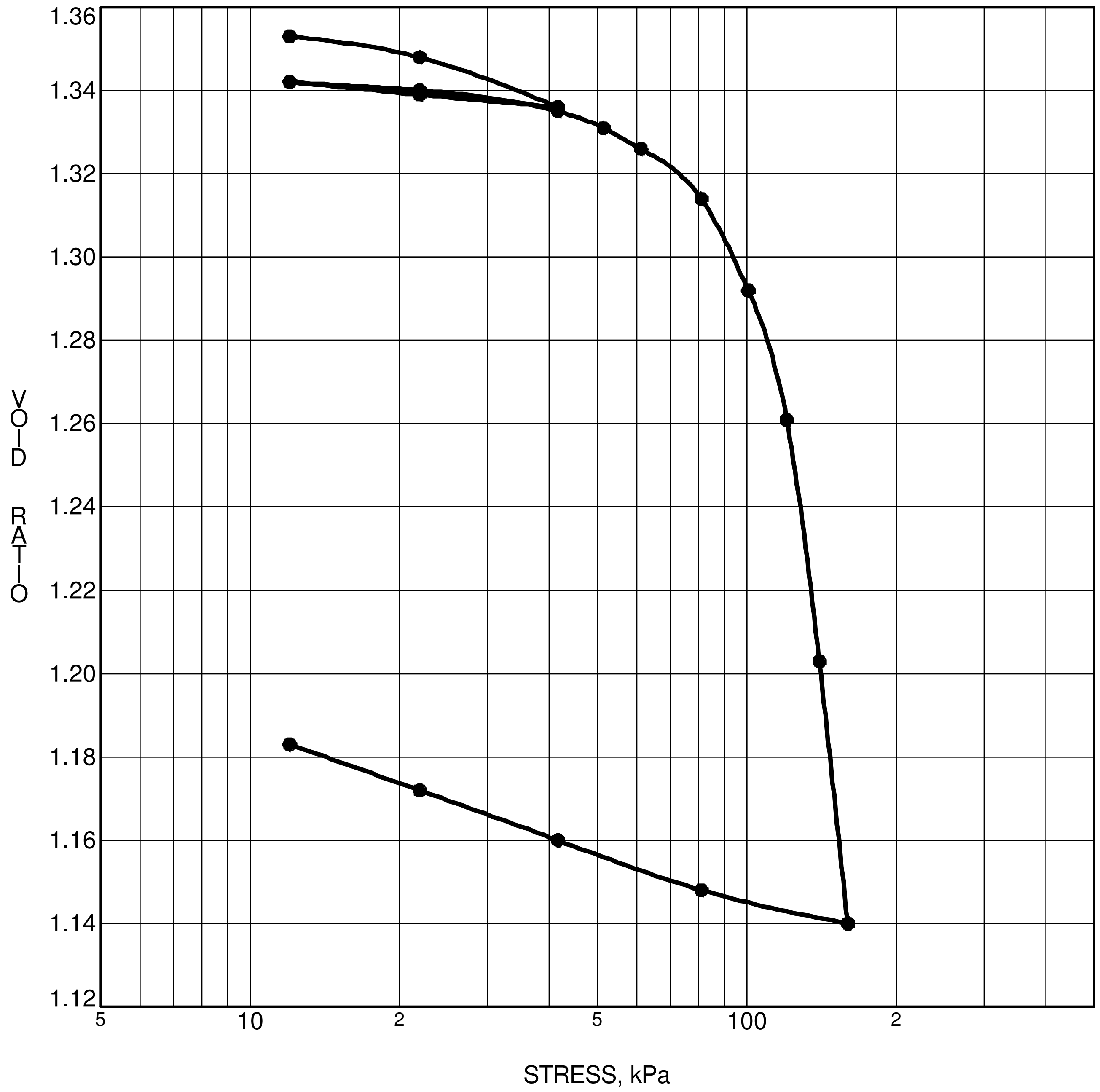
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST

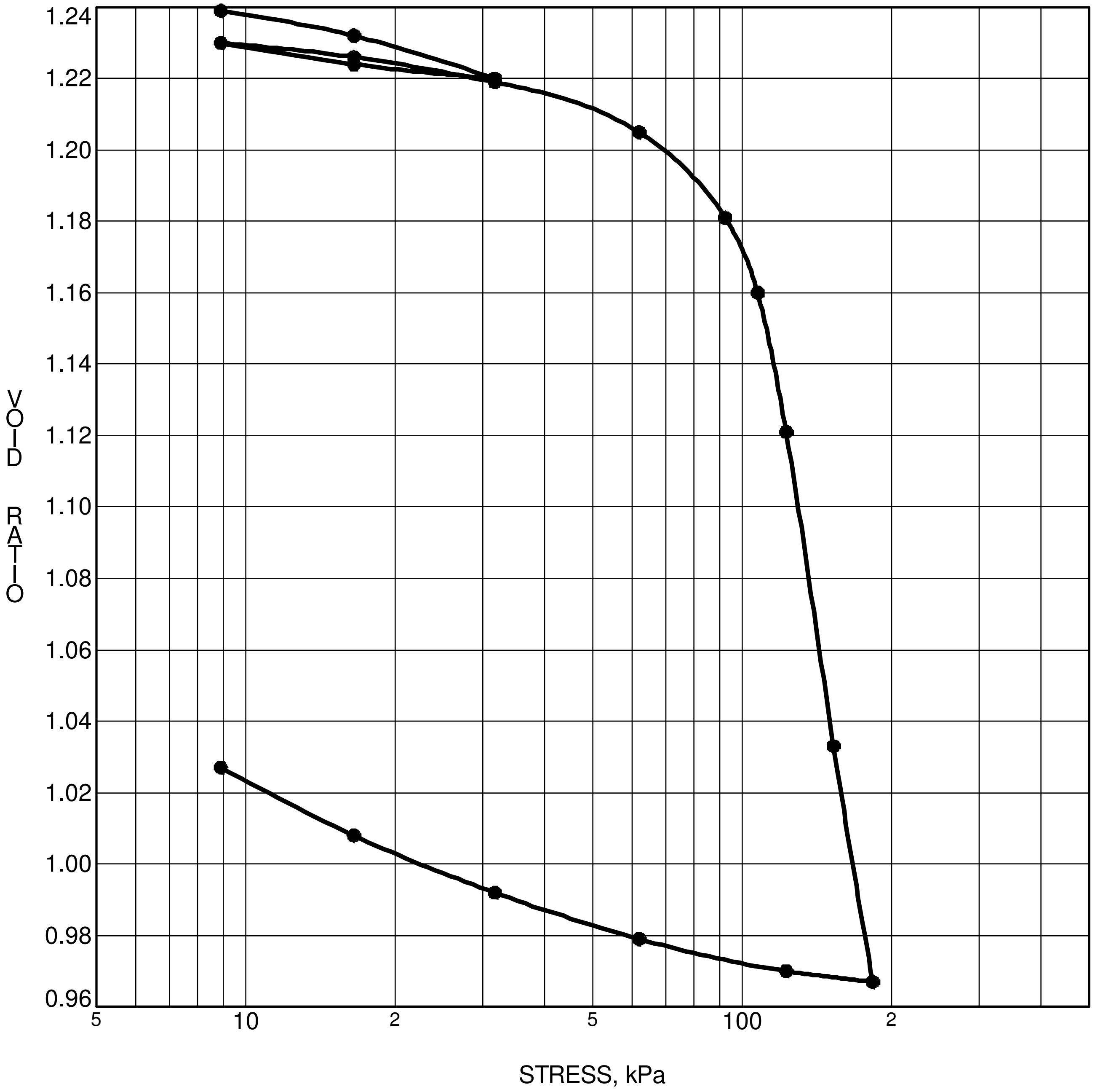


CONSOLIDATION TEST DATA SUMMARY							
Borehole No.	**BH 5-15**	p'$_o$	**56**	kPa	Ccr	**0.015**	
Sample No.	**TW 6**	p'$_c$	**114**	kPa	Cc	**1.105**	
Sample Depth	**5.64** m	OC Ratio	**2.0**		Wo	**49.7**	%
Sample Elev.	**87.80** m	Void Ratio	**1.366**		Unit Wt.	**17.0**	kN/m^3

CLIENT **Minto Communities Inc.**
FILE NO. **PG1984**
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**
DATE **19/03/2015**

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

CONSOLIDATION TEST



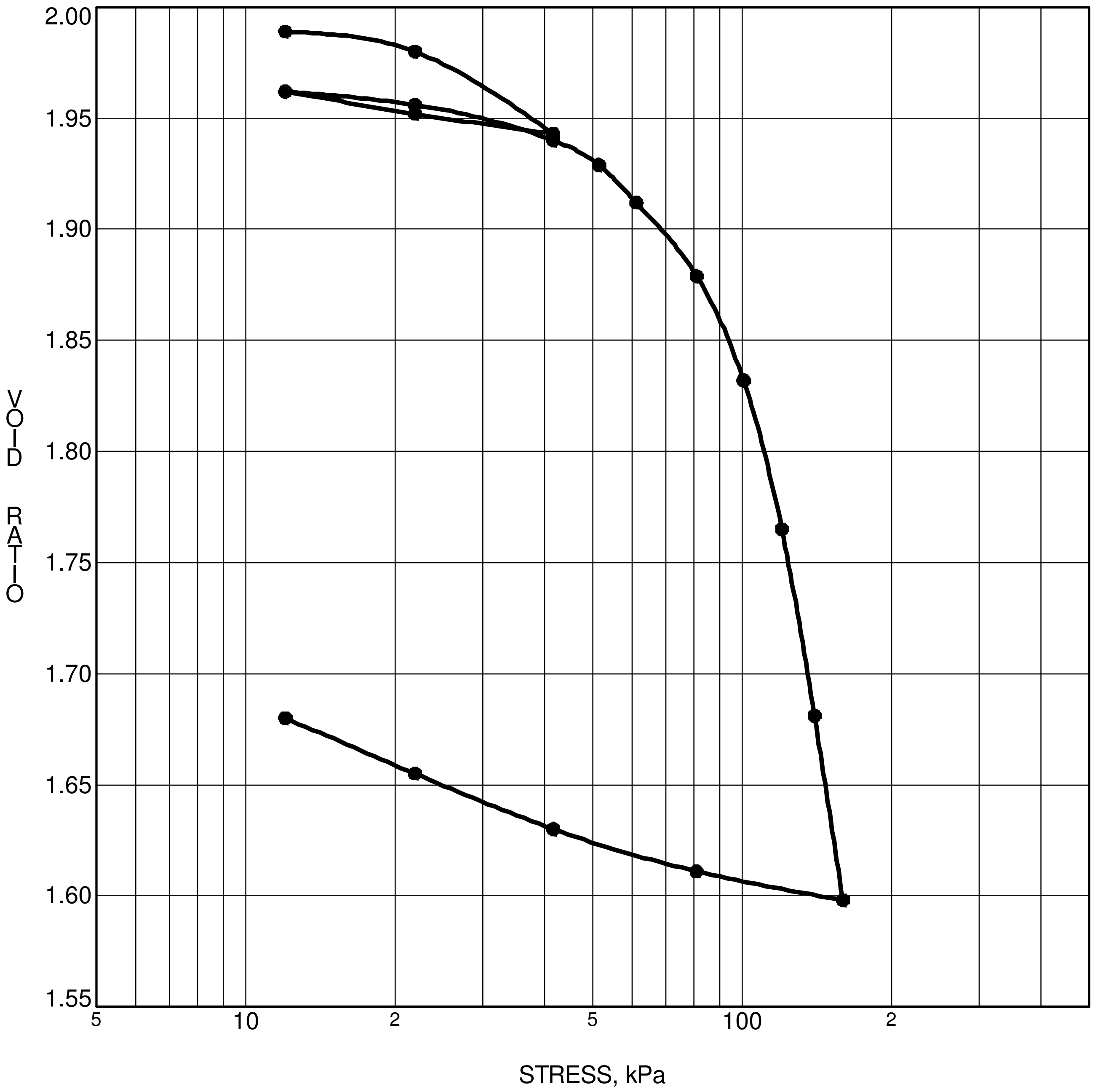
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 6-15**	p'_o	**53** kPa	Ccr	**0.020**
Sample No.	**TW 5**	p'_c	**101** kPa	Cc	**0.890**
Sample Depth	**5.66** m	OC Ratio	**1.9**	Wo	**45.1** %
Sample Elev.	**88.64** m	Void Ratio	**1.24**	Unit Wt.	**17.4** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1984**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**	DATE	**19/03/2015**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 7-15**	p'_o	**53** kPa	Ccr	**0.038**
Sample No.	**TW 5**	p'_c	**96** kPa	Cc	**1.321**
Sample Depth	**5.82** m	OC Ratio	**1.8**	Wo	**73.4** %
Sample Elev.	**88.23** m	Void Ratio	**2.02**	Unit Wt.	**15.5** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG1984**

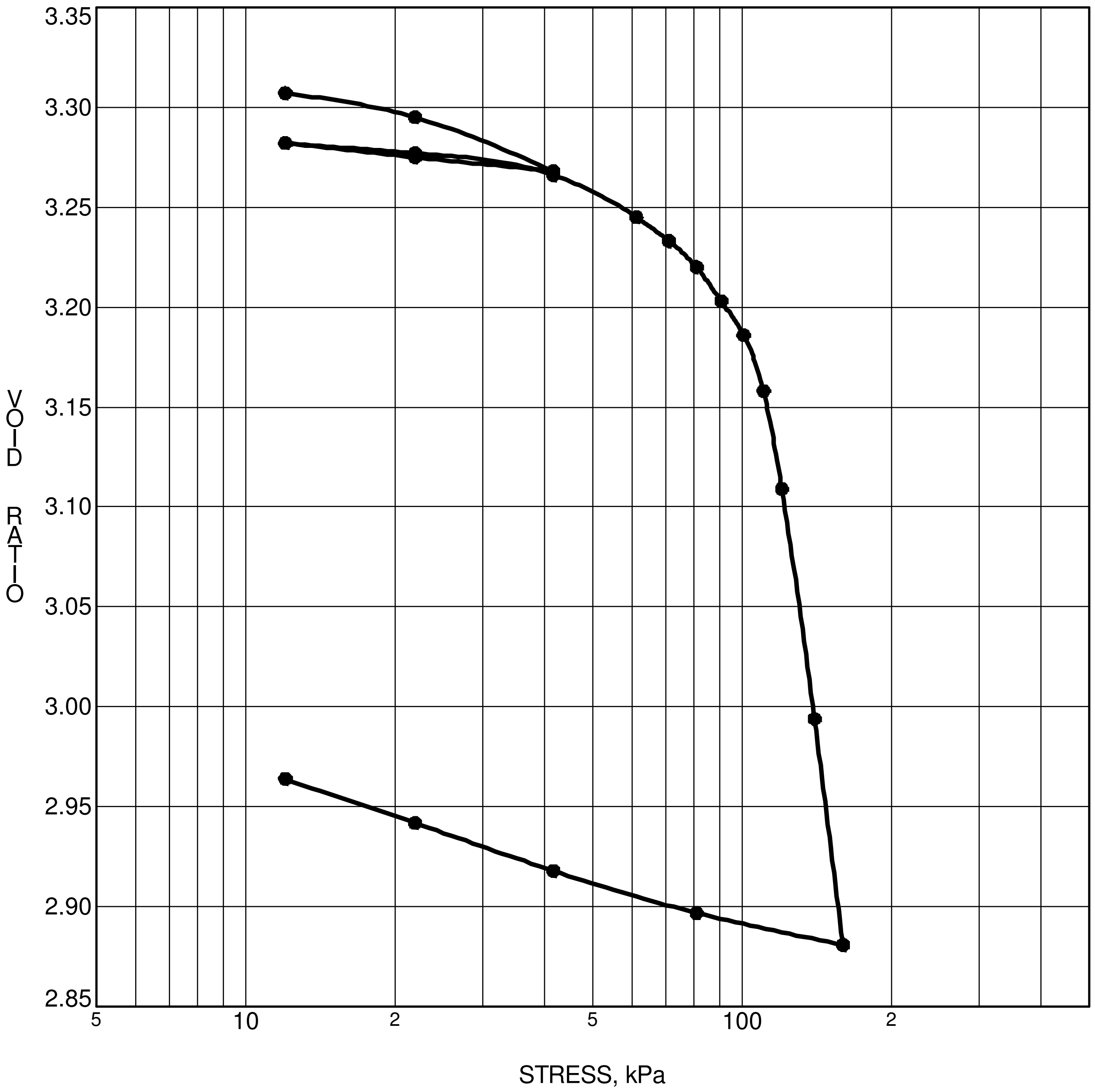
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**

DATE **06/04/2015**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 8-15**	p'o	**55** kPa	Ccr	**0.026**
Sample No.	**TW 3**	p'c	**107** kPa	Cc	**1.878**
Sample Depth	**4.14** m	OC Ratio	**1.9**	Wo	**121.5**%
Sample Elev.	**88.57** m	Void Ratio	**3.34**	Unit Wt.	**16.7** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG1984**

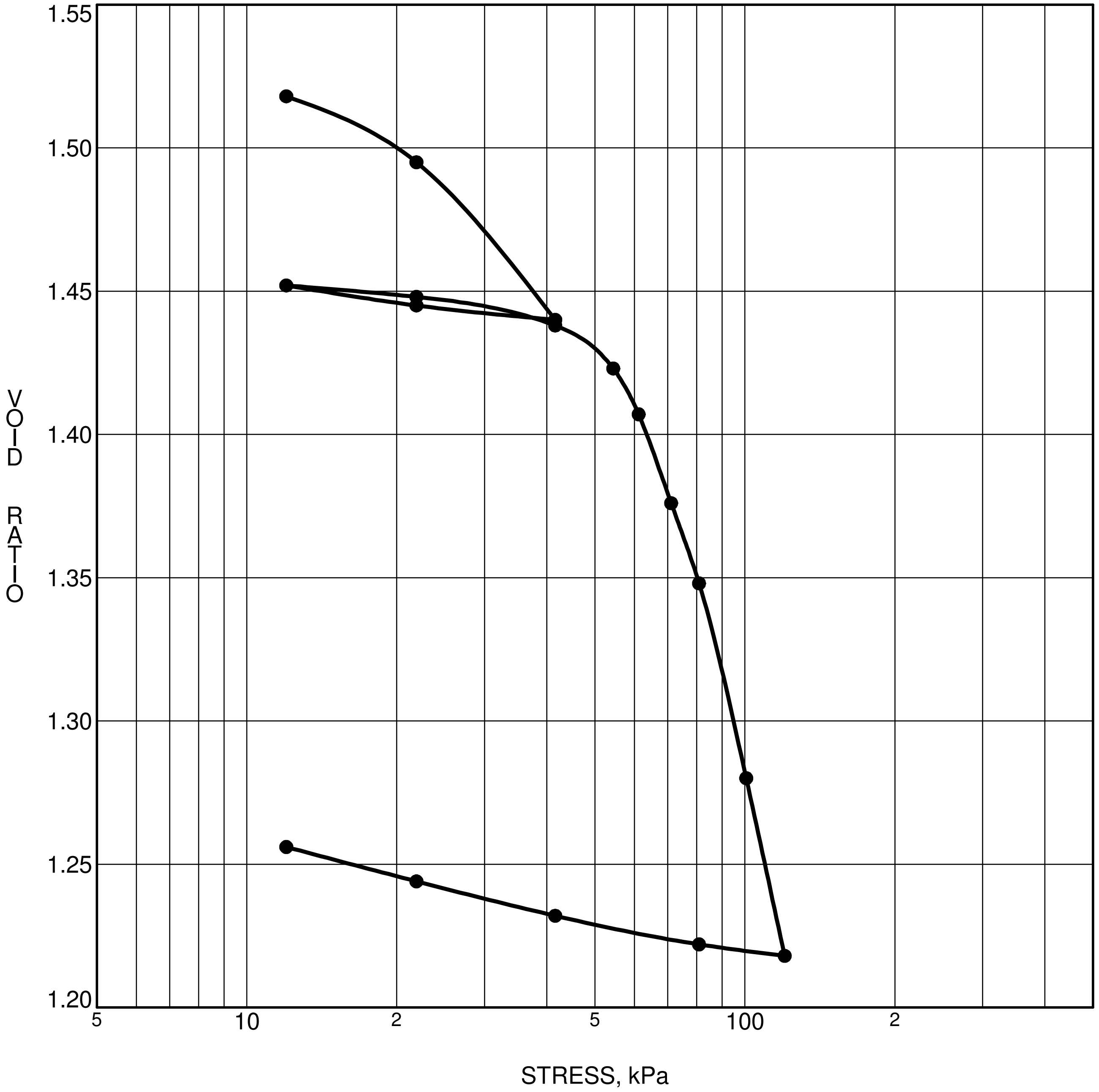
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**

DATE **26/03/2015**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



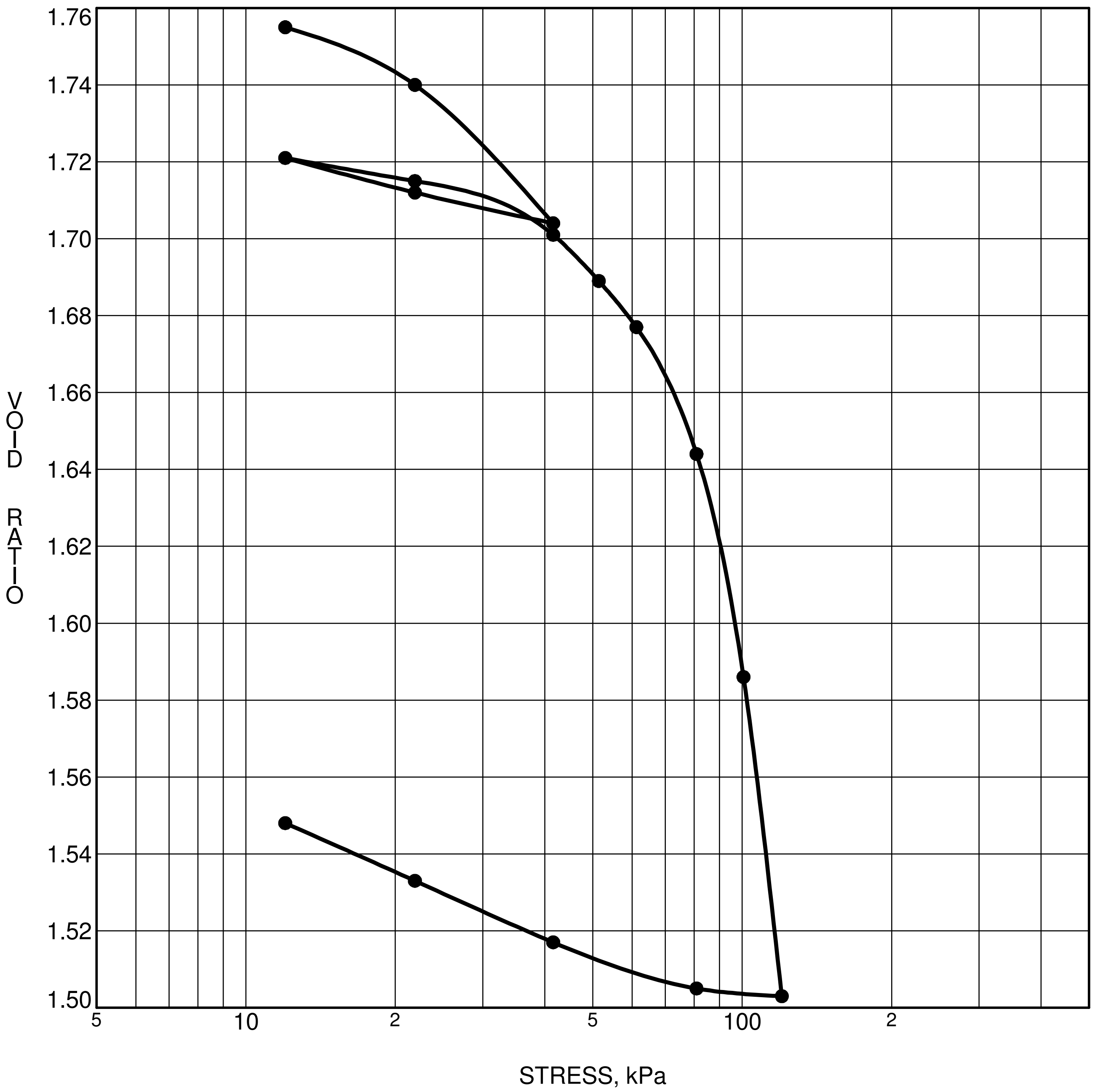
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH13-16**	p'o	**56** kPa	Ccr	**0.024**
Sample No.	**TW 6**	p'c	**67** kPa	Cc	**0.738**
Sample Depth	**5.82** m	OC Ratio	**1.2**	Wo	**56.0** %
Sample Elev.	**88.17** m	Void Ratio	**1.54**	Unit Wt.	**16.6** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1984**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - Clarke Lands**	DATE	**29/04/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



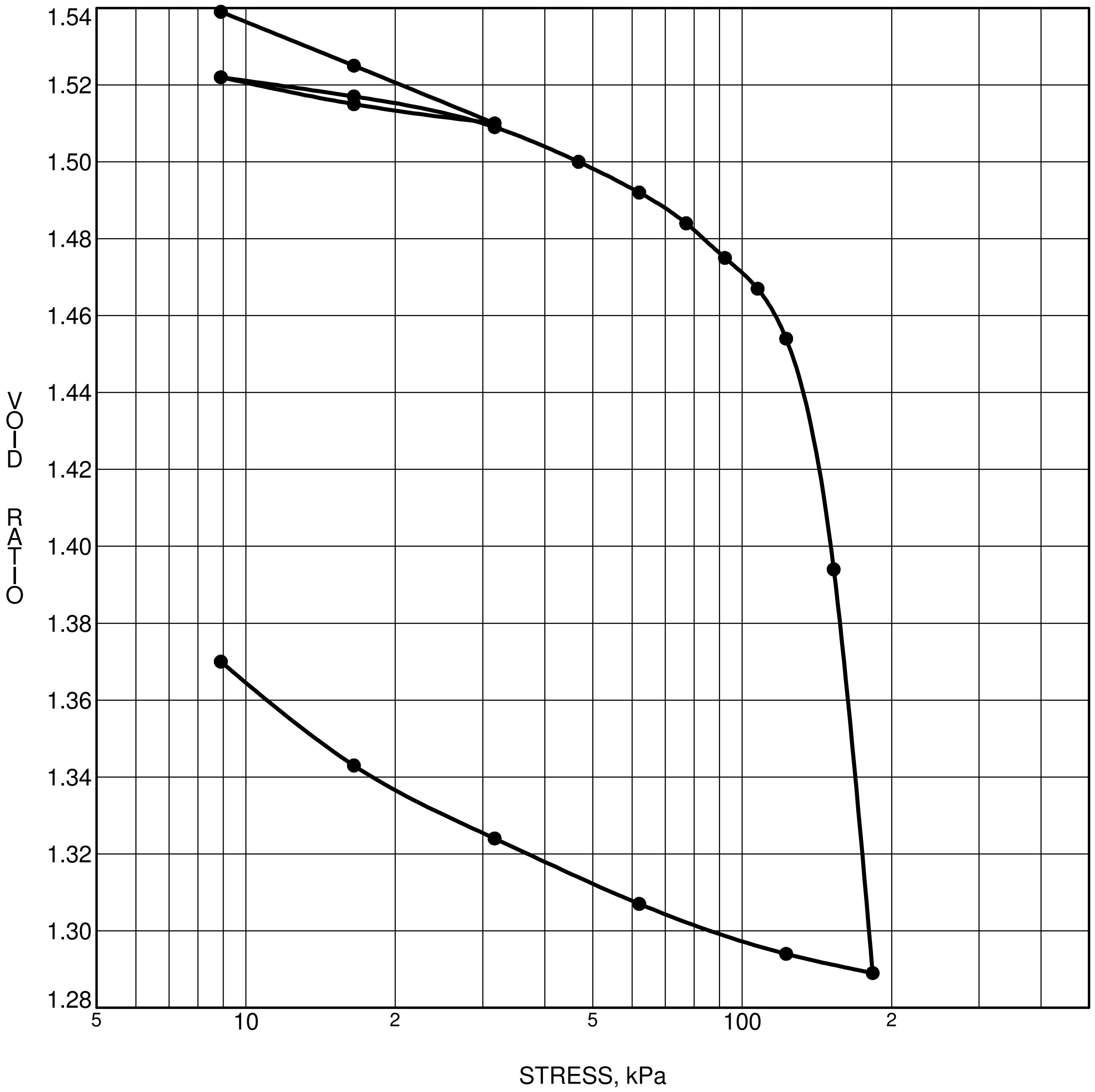
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH14A-16**	p'_o	**66** kPa	Ccr	**0.035**
Sample No.	**TW 3**	p'_c	**87** kPa	Cc	**1.101**
Sample Depth	**7.01** m	OC Ratio	**1.3**	Wo	**64.7** %
Sample Elev.	**86.48** m	Void Ratio	**1.78**	Unit Wt.	**16.0** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1984**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - Clarke Lands**	DATE	**13/05/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



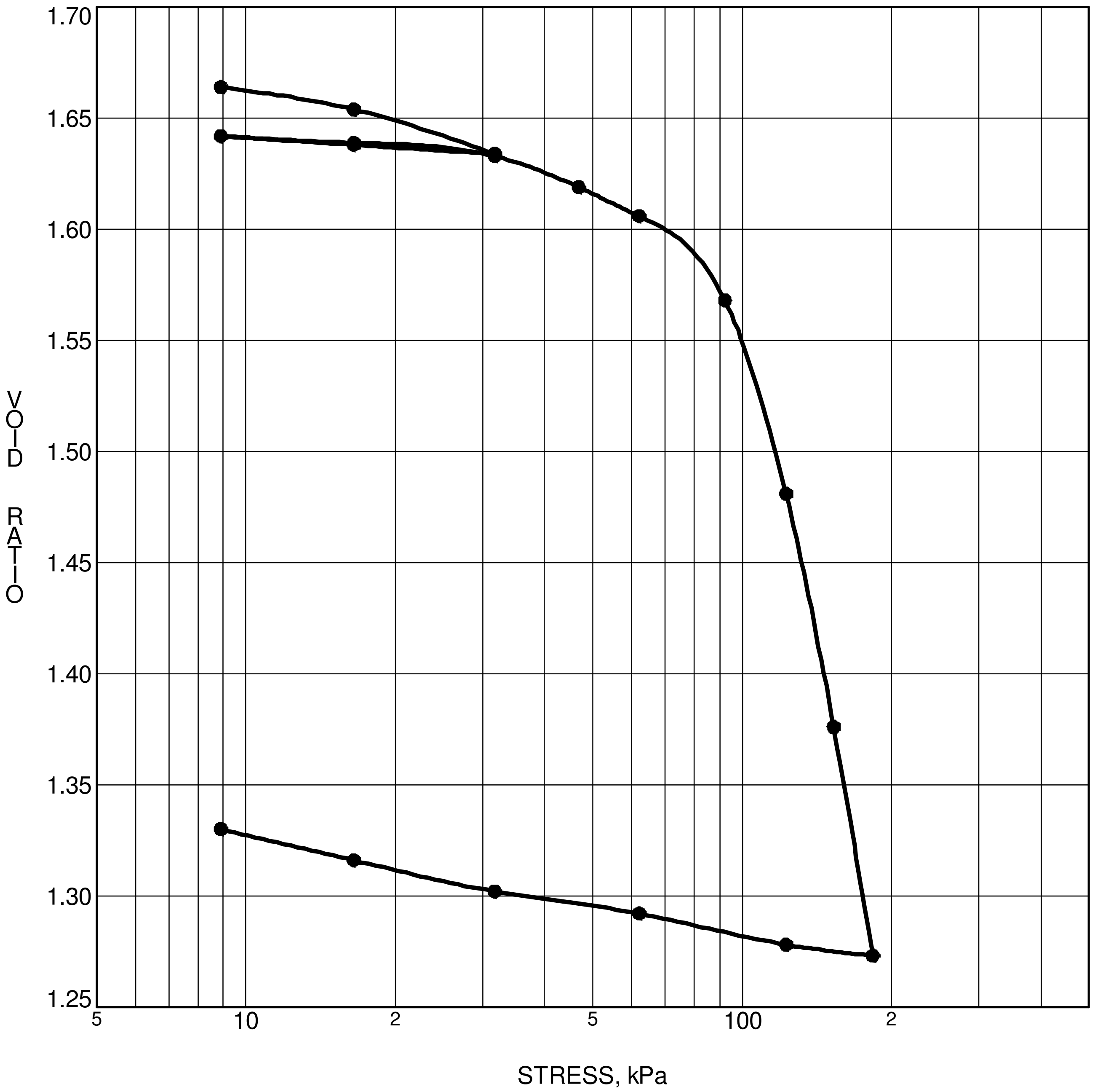
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH23-16**	p'_o	**60** kPa	Ccr	**0.023**
Sample No.	**TW 4**	p'_c	**137** kPa	Cc	**1.366**
Sample Depth	**5.08** m	OC Ratio	**2.3**	Wo	**56.7** %
Sample Elev.	**87.21** m	Void Ratio	**1.56**	Unit Wt.	**16.5** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1984**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - Clarke Lands**	DATE	**09/05/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 7**	p'_o	**65** kPa	Ccr	**0.016**
Sample No.	**TW 7**	p'_c	**100** kPa	Cc	**1.199**
Sample Depth	**7.31** m	OC Ratio	**1.5**	Wo	**61.2** %
Sample Elev.	**86.65** m	Void Ratio	**1.684**	Unit Wt.	**16.2** kN/m^3

CLIENT **Minto Communities Inc.**

PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

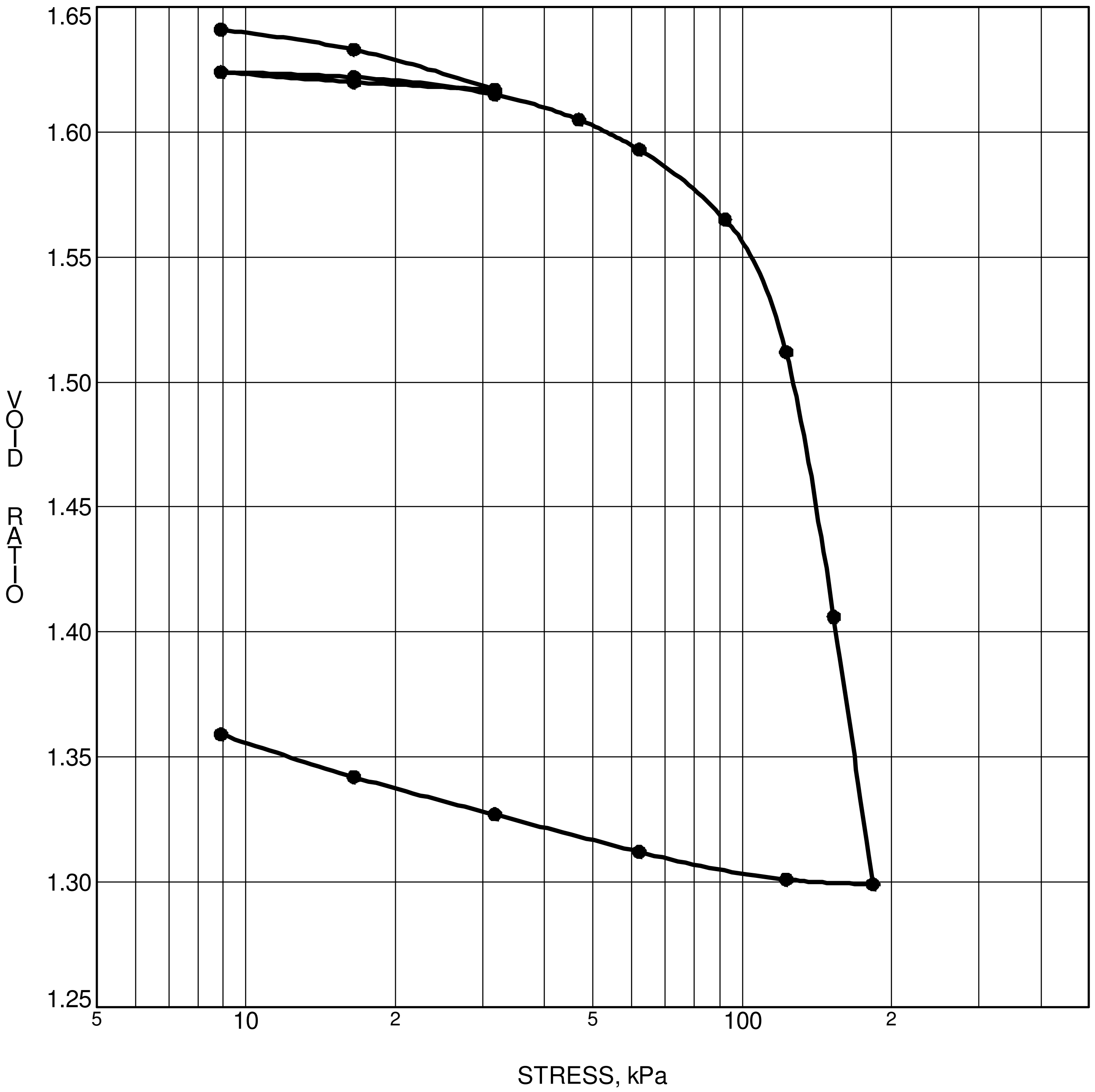
FILE NO. **PG1984**

DATE **02/25/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 8**	p'$_o$	**67** kPa	Ccr	**0.014**
Sample No.	**TW 4**	p'$_c$	**107** kPa	Cc	**1.076**
Sample Depth	**5.03** m	OC Ratio	**1.6**	Wo	**60.4** %
Sample Elev.	**87.26** m	Void Ratio	**1.66**	Unit Wt.	**16.3** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG1984**

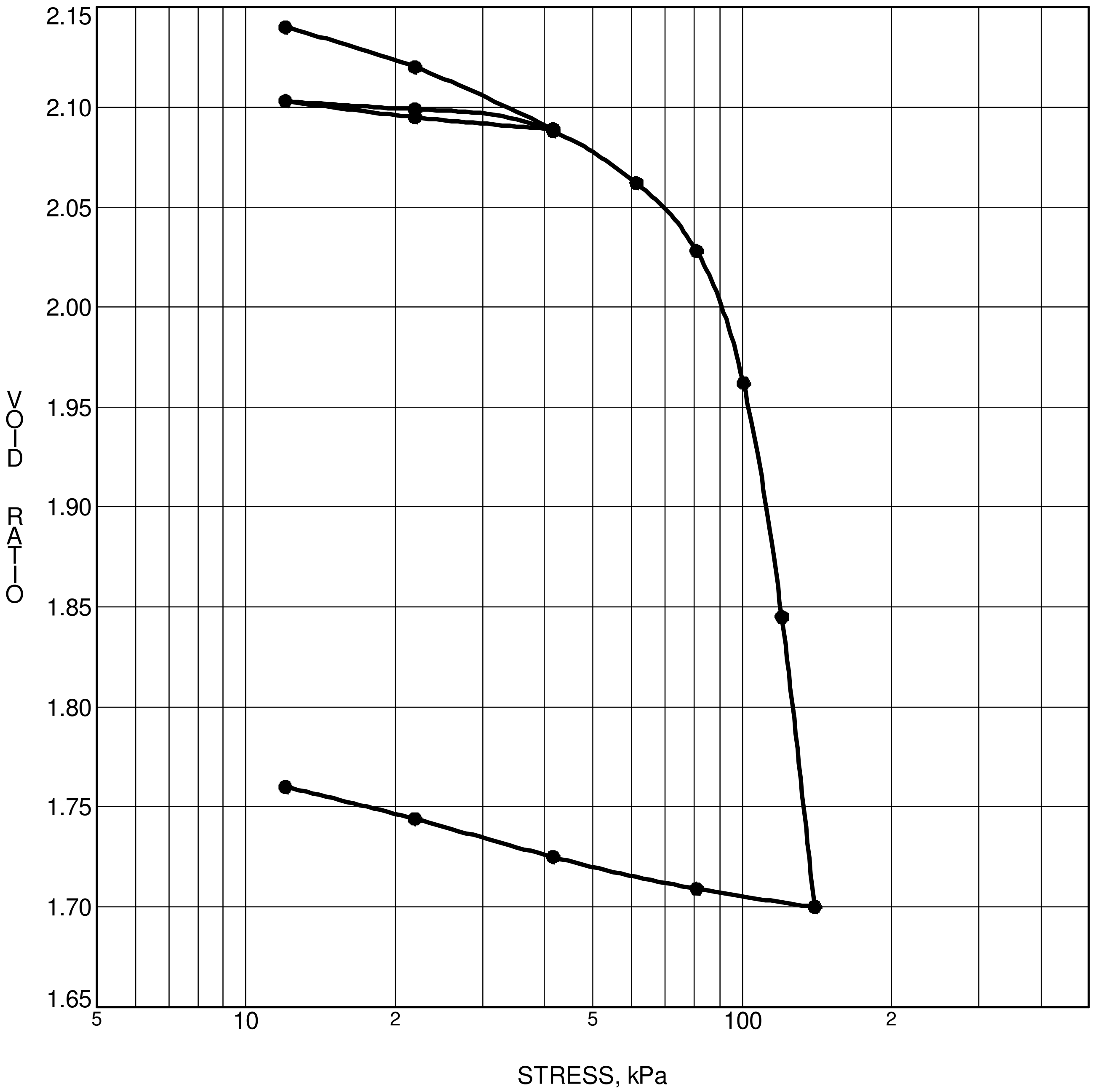
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

DATE **02/14/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST

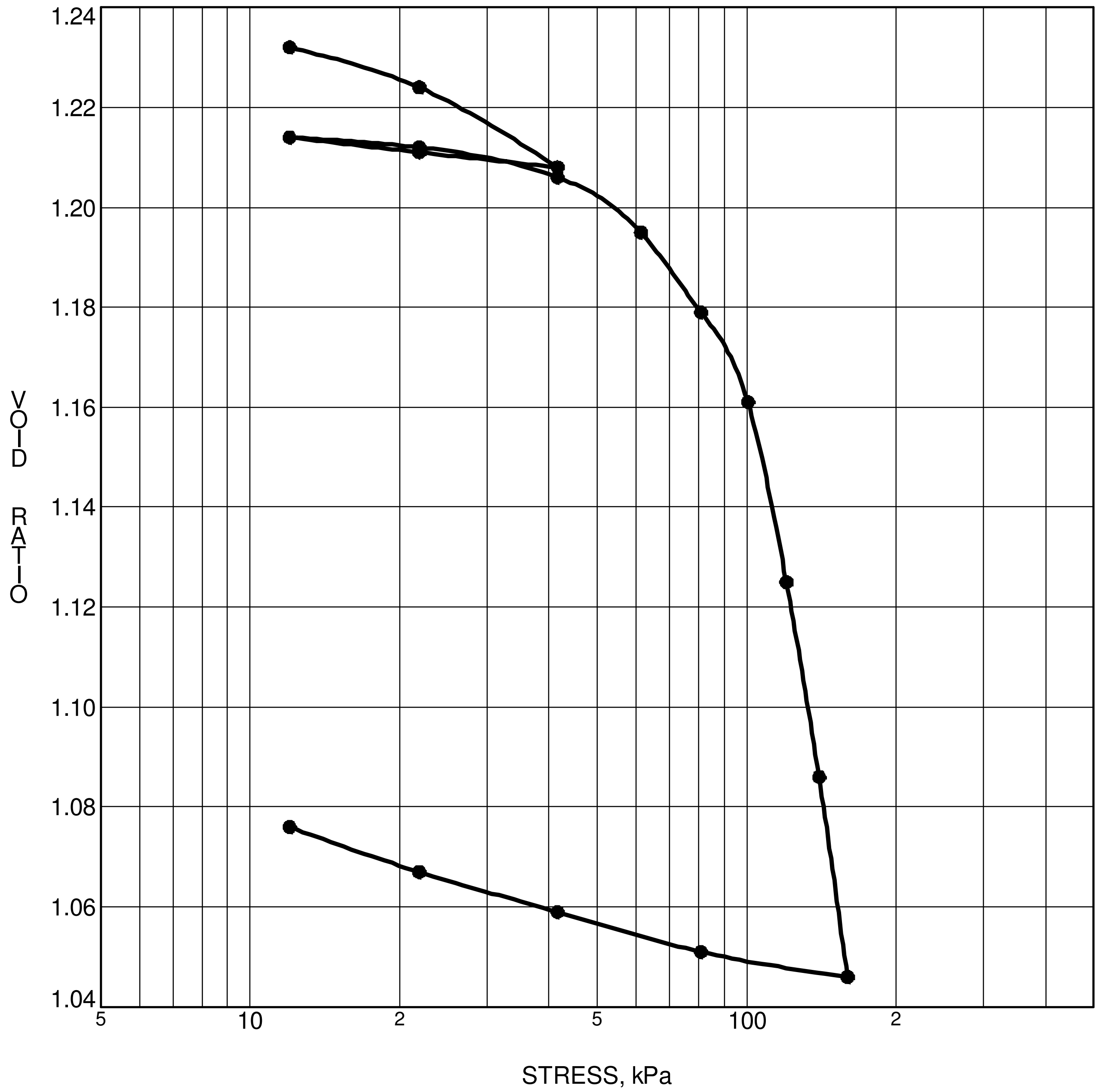


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 8**	p'$_o$	**70** kPa	Ccr	**0.028**
Sample No.	**TW 5**	p'$_c$	**92** kPa	Cc	**1.856**
Sample Depth	**6.60** m	OC Ratio	**1.3**	Wo	**78.6** %
Sample Elev.	**85.69** m	Void Ratio	**2.162**	Unit Wt.	**15.2** kN/m^3

CLIENT **Minto Communities Inc.**
FILE NO. **PG1984**
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**
DATE **03/08/2011**

patersongroup **Consulting Engineers**
28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST

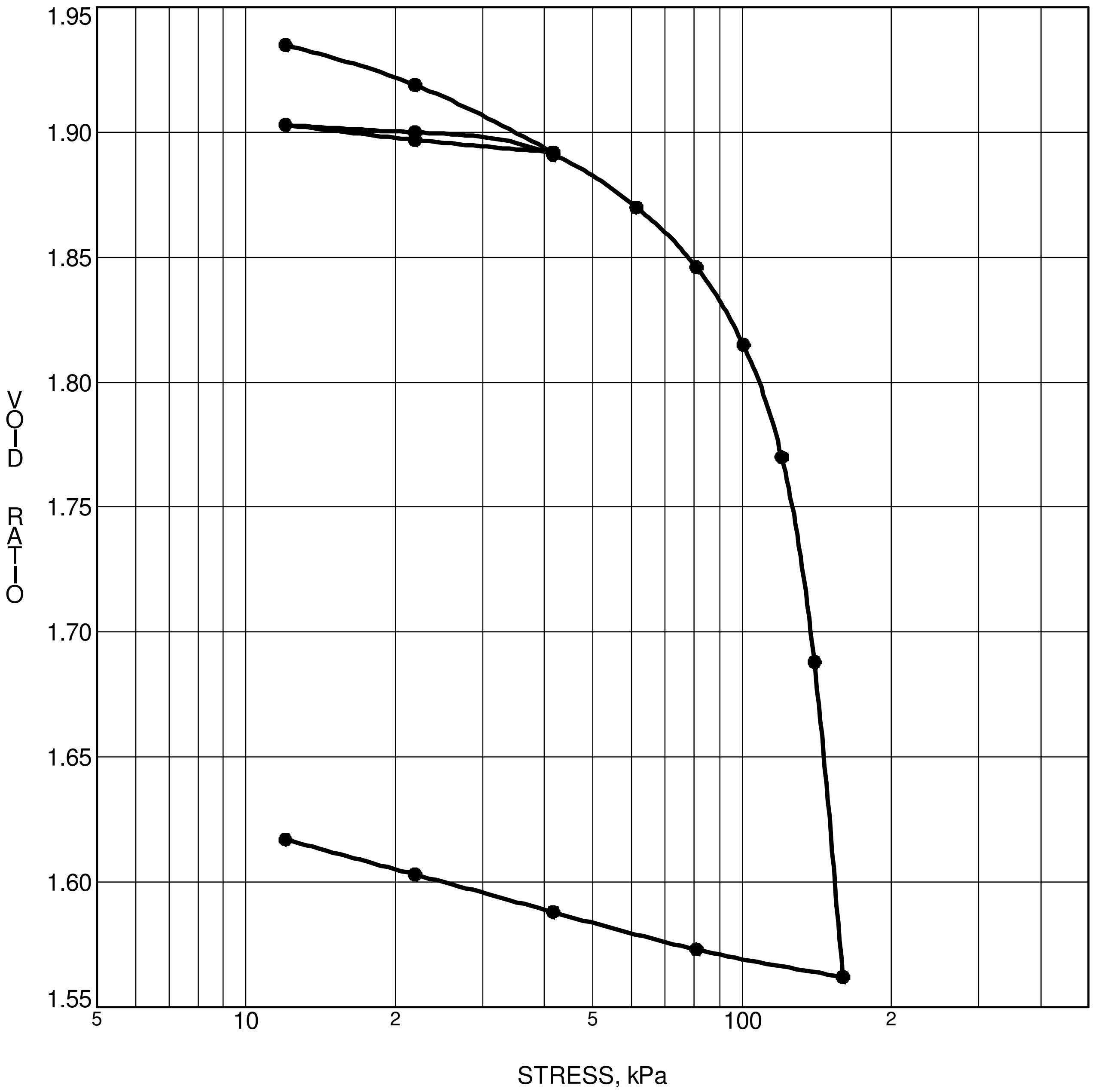


CONSOLIDATION TEST DATA SUMMARY								
Borehole No.	BH 9		p'_o	60	kPa	Ccr	0.014	
Sample No.	TW 4		p'_c	100	kPa	Cc	0.656	
Sample Depth	5.03	m	OC Ratio	1.7		Wo	45.4	%
Sample Elev.	87.03	m	Void Ratio	1.249		Unit Wt.	17.5	kN/m^3

CLIENT **Minto Communities Inc.**
FILE NO. **PG1984**
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**
DATE **02/25/2011**

patersongroup **Consulting Engineers**
28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



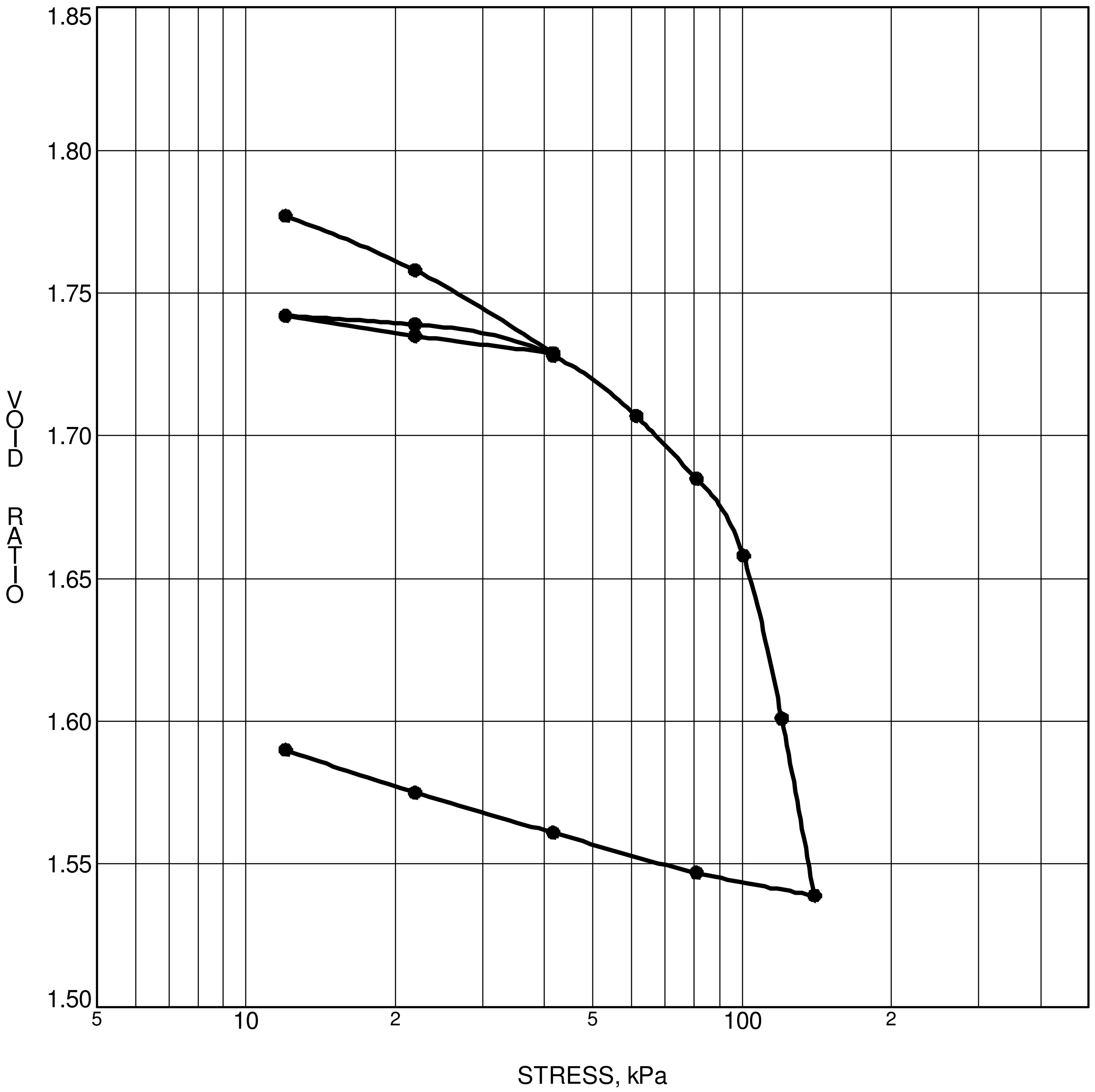
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 9**	p'_o	**79** kPa	Ccr	**0.023**
Sample No.	**TW 5**	p'_c	**112** kPa	Cc	**1.717**
Sample Depth	**8.10** m	OC Ratio	**1.4**	Wo	**71.1** %
Sample Elev.	**83.96** m	Void Ratio	**1.955**	Unit Wt.	**15.6** kN/m ^{3}

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1984**
PROJECT	**Geotechnical Investigation - Proposed Residential Development - Clarke Lands**	DATE	**02/22/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH10**	p'$_o$	**55** kPa	Ccr	**0.025**
Sample No.	**TW 3**	p'$_c$	**90** kPa	Cc	**0.823**
Sample Depth	**4.24** m	OC Ratio	**1.6**	Wo	**65.5** %
Sample Elev.	**87.85** m	Void Ratio	**1.8**	Unit Wt.	**15.9** kN/m ^{3}

CLIENT **Minto Communities Inc.**

PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

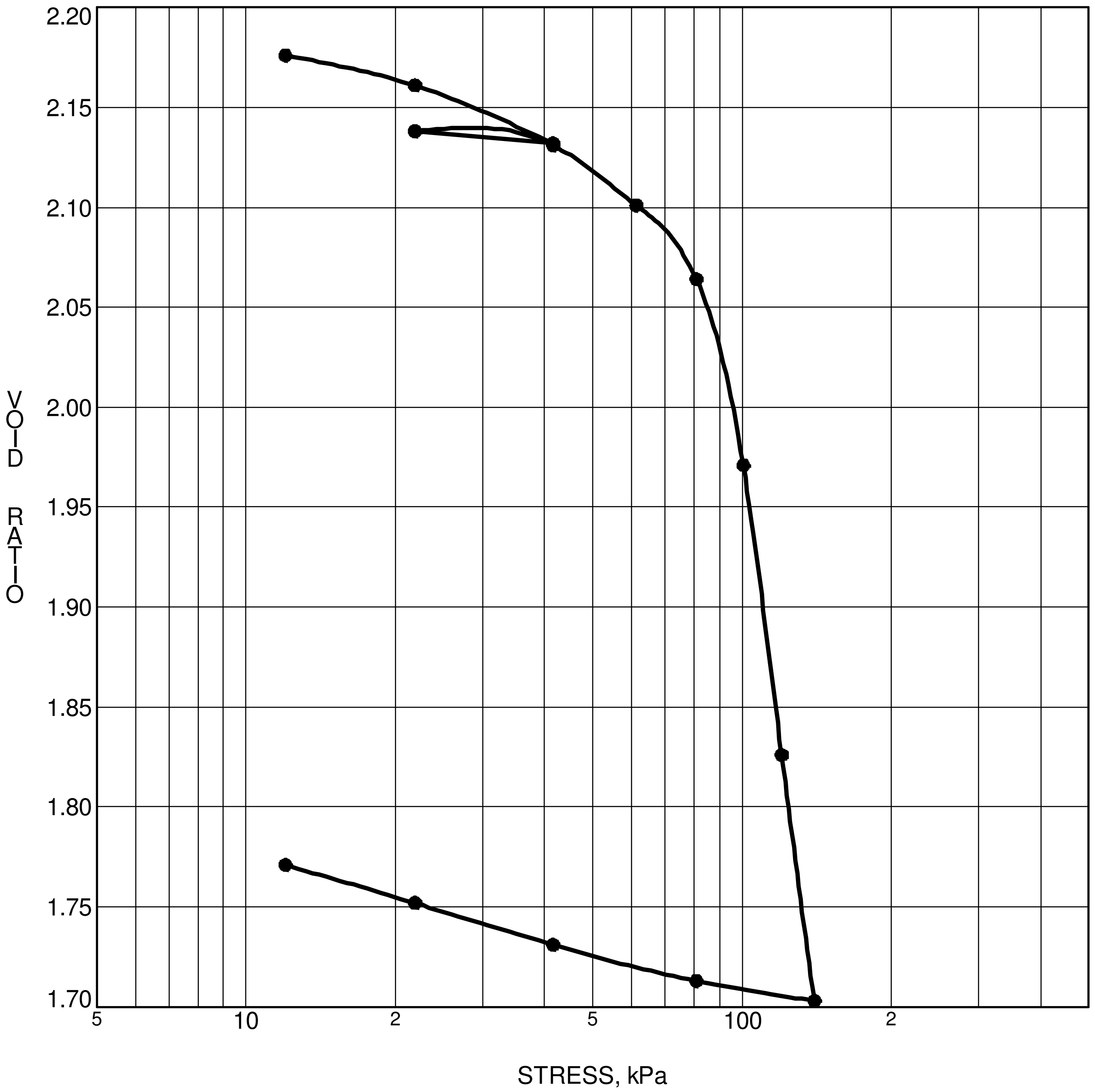
FILE NO. **PG1984**

DATE **02/04/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY

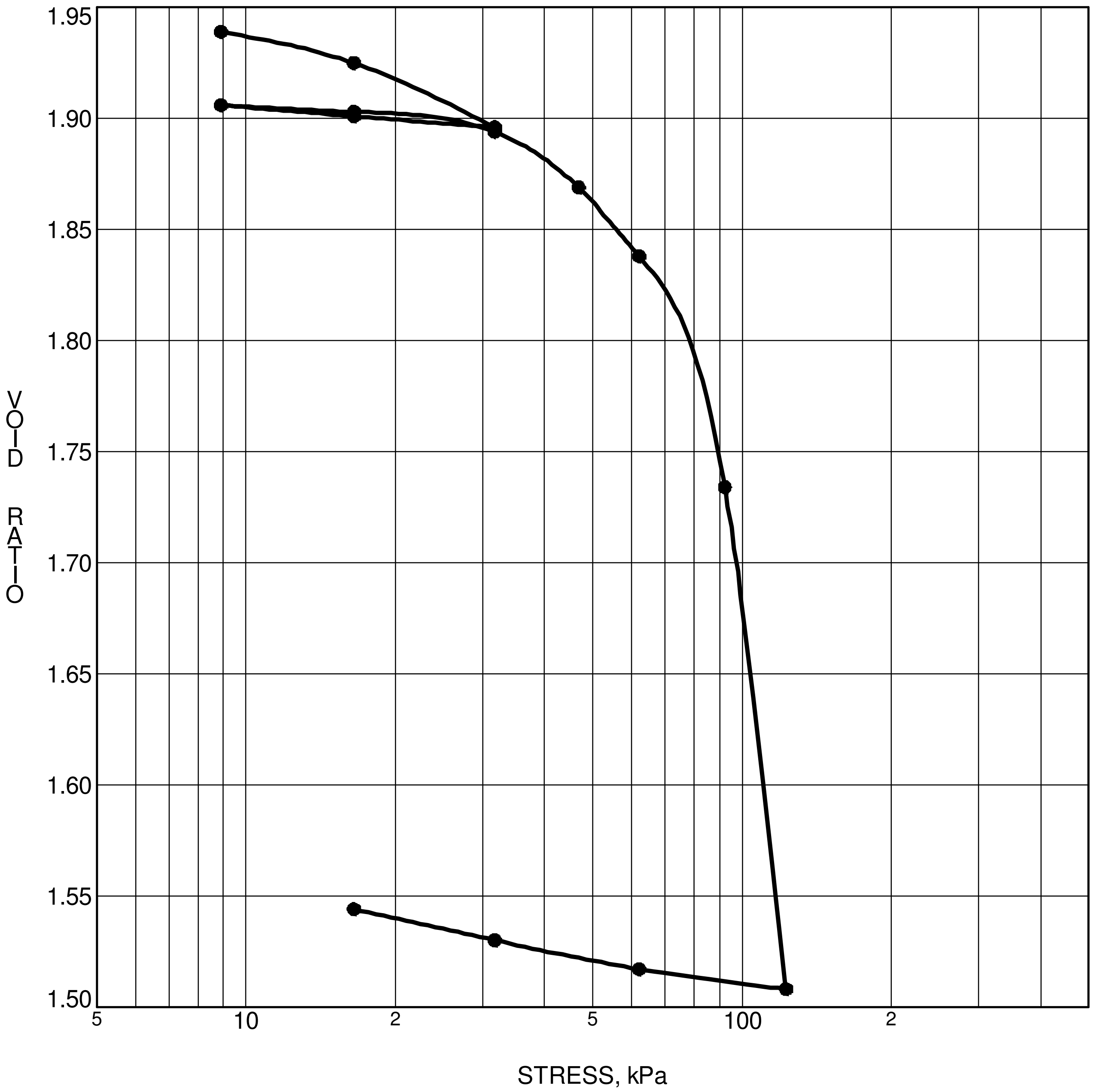
Borehole No.	**BH10**	p'$_o$	**65** kPa	Ccr	**0.026**
Sample No.	**TW 4**	p'$_c$	**88** kPa	Cc	**1.898**
Sample Depth	**5.79** m	OC Ratio	**1.4**	Wo	**80.2** %
Sample Elev.	**86.30** m	Void Ratio	**2.206**	Unit Wt.	**15.2** kN/m ^{3}

CLIENT **Minto Communities Inc.**
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

FILE NO. **PG1984**
DATE **02/22/2011**

patersongroup **Consulting Engineers**
28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST

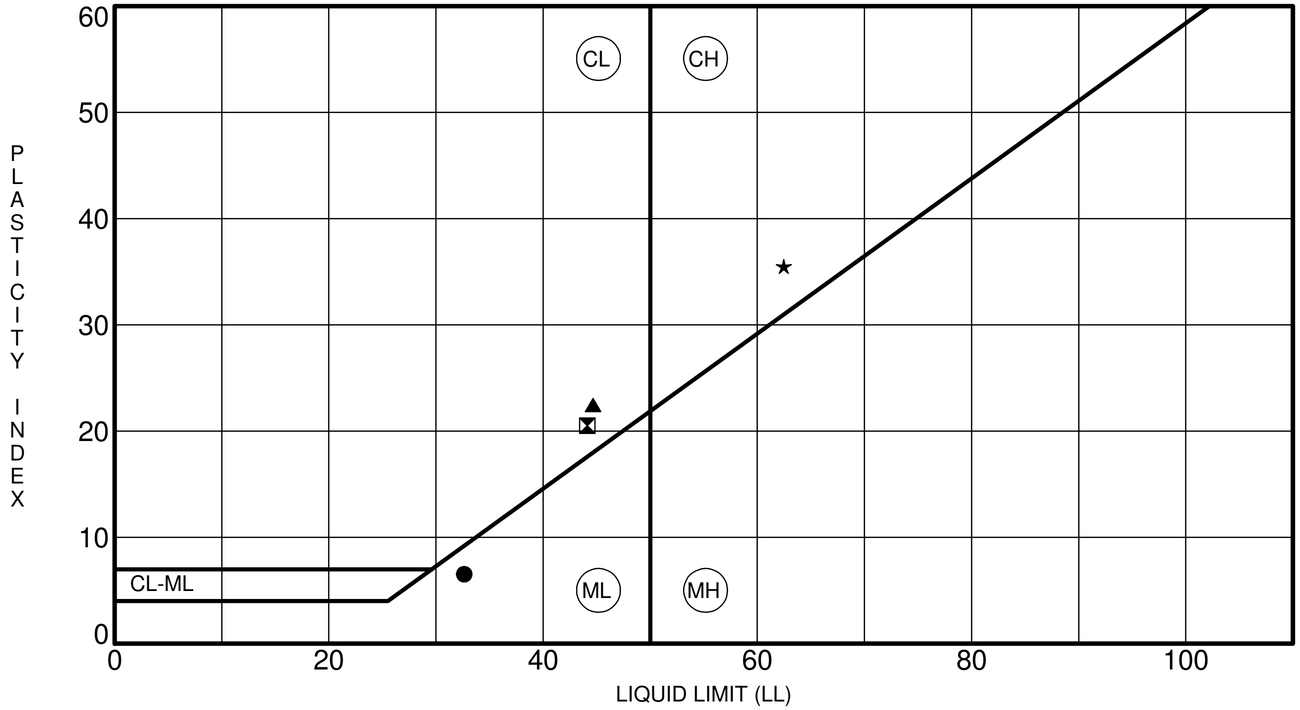


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH10**	p'$_o$	**83** kPa	Ccr	**0.020**
Sample No.	**TW 5**	p'$_c$	**78** kPa	Cc	**1.835**
Sample Depth	**8.76** m	OC Ratio	**0.9**	Wo	**71.4** %
Sample Elev.	**83.33** m	Void Ratio	**1.962**	Unit Wt.	**15.6** kN/m^3

CLIENT **Minto Communities Inc.**
FILE NO. **PG1984**
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**
DATE **02/04/2011**

patersongroup **Consulting Engineers**
28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 4-15**	**TW 3**	**33**	**26**	**7**		**ML - Silt of low compressability**
⧗	**BH 6-15**	**TW 5**	**44**	**24**	**21**		**CL - Inorganic clays of low plasticity**
▲	**BH 8-15**	**TW 3**	**45**	**22**	**22**		**CL - Inorganic clays of low plasticity**
★	**BH10-11**	**TW 4**	**62**	**27**	**36**		**CH-Inorganic clay of high plasticity**

CLIENT **Minto Communities Inc.**
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke Lands Stage 2**

FILE NO. **PG1984**
DATE **28 Jan 11**

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

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 26301

Report Date: 11-Apr-2019

Order Date: 5-Apr-2019

Project Description: PG3968

	Client ID:	BH1-19 SS4 7.'-9.5'	BH6-19 SS3 5'-7'	-	-
	Sample Date:	03/22/2019 10:00	03/20/2019 10:00	-	-
	Sample ID:	1914639-01	1914639-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	75.0	84.6	-	-
General Inorganics					
pH	0.05 pH Units	7.66	7.62	-	-
Resistivity	0.10 Ohm.m	32.4	38.2	-	-
Anions					
Chloride	5 ug/g dry	49	11	-	-
Sulphate	5 ug/g dry	53	50	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG3968-2 - TEST HOLE LOCATION PLAN

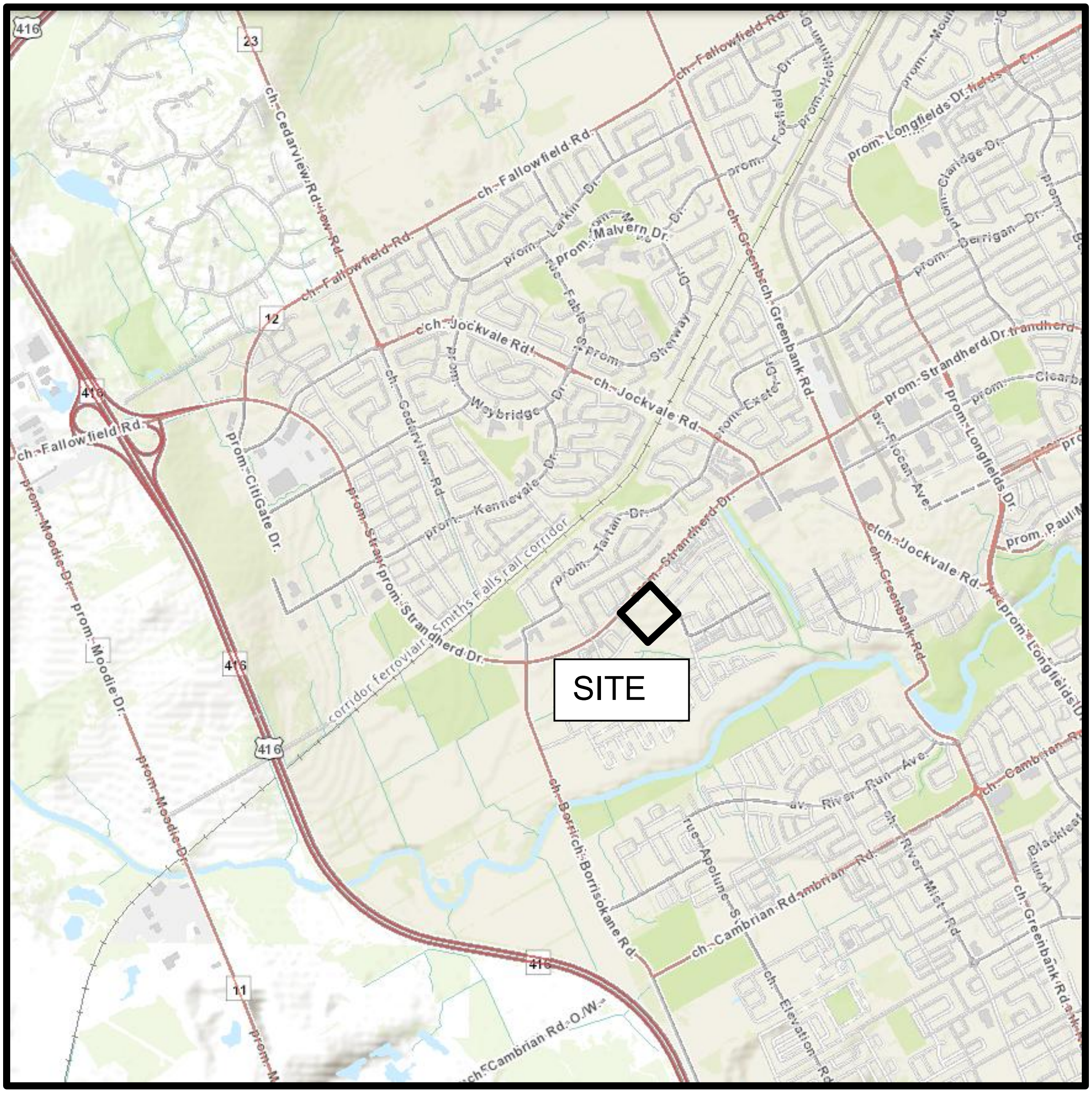
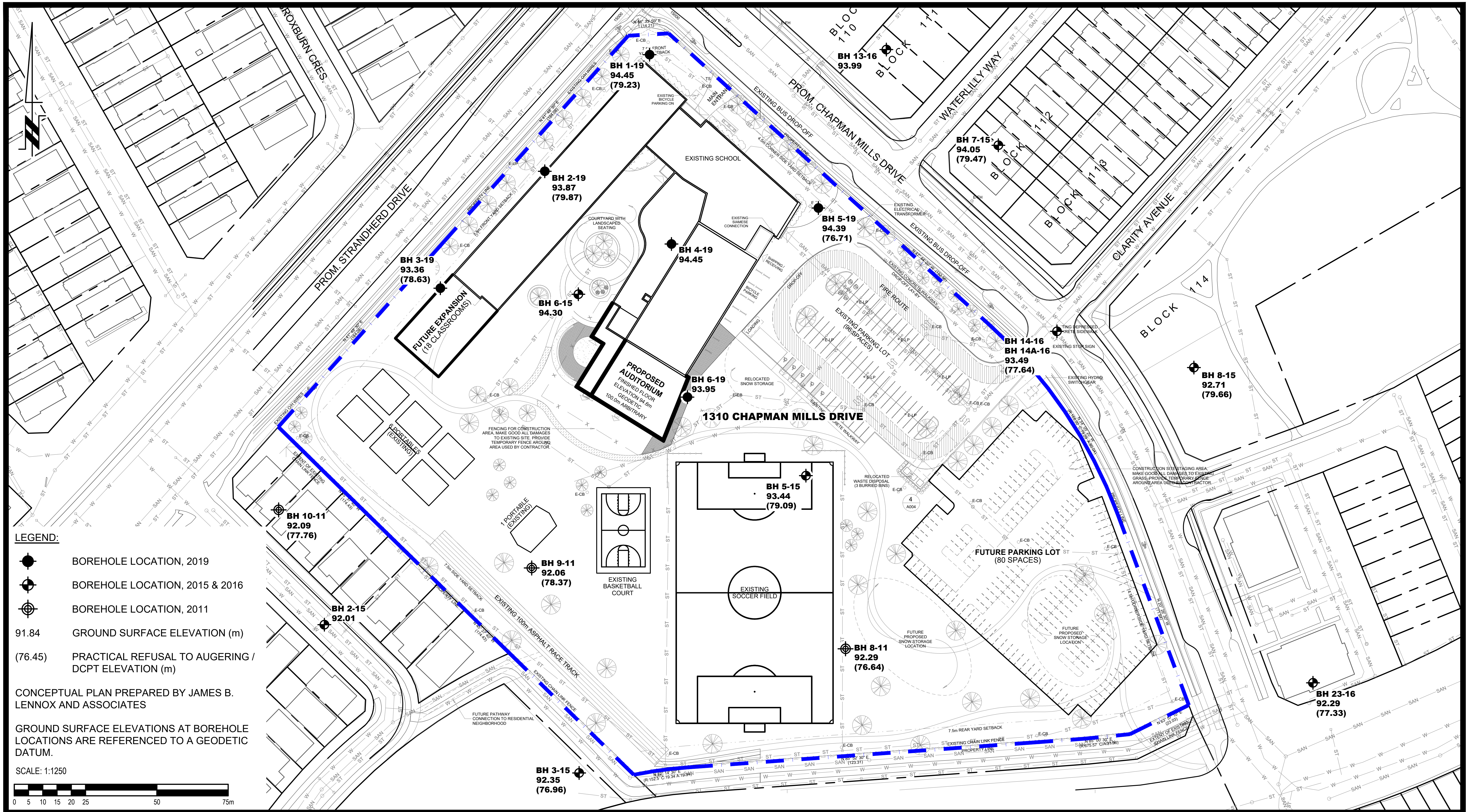


FIGURE 1

KEY PLAN



LEGEND:

●	BOREHOLE LOCATION, 2019
⊕	BOREHOLE LOCATION, 2015 & 2016
⌖	BOREHOLE LOCATION, 2011
91.84	GROUND SURFACE ELEVATION (m)
(76.45)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY JAMES B. LENNOX AND ASSOCIATES

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

SCALE: 1:1250

0 5 10 15 20 25 50 75m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CEPEO
GEOTECHNICAL DESKTOP REVIEW
PROPOSED BARRHAVEN HIGH SCHOOL AUDITORIUM
1310 CHAPMAN MILLS DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250	Date: 01/2026
Drawn by: YA	Report No.: PG3968-3
Checked by: NS	Dwg. No.: PG3968-2
Approved by: JV	Revision No.:

p:\autocad drawings\geotechnical\pg39xx\pg3968\pg3968-2-test hole location plan.dwg

11x17