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Final Geotechnical Investigation

Proposed Barrhaven High School
Strandherd Road at Chapman Mills Drive
Ottawa, Ontario

Prepared For

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de l'Est de l'Ontario

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Appendices

1.0 Introduction

Paterson Group Inc. (Paterson) has been retained by the Conseil des écoles publiques de l’Est de l’Ontario (CEPEO) to conduct a final geotechnical investigation for the proposed Barrhaven High School project on Strandherd Drive at Chapman Mills Drive, in Ottawa, Ontario (refer to Figure 1 - Key Plan). The current geotechnical investigation report is a stand-alone document and includes the results of current supplementary geotechnical investigation work as well as compiled previous geotechnical information.

Paterson previously prepared a geotechnical report using compiled previous information, as detailed in our File PG3968-1, dated November 21, 2016. Paterson has undertaken a number of geotechnical investigation phases for the Harmony (Clarke Lands) Development for Minto Communities Inc. (Minto). Minto has granted permission for the results of these investigation stages to be relied on by CEPEO and their agents for purposes of school development at the subject site.

In addition to the current geotechnical investigation work, consisting of six (6) boreholes, three (3) stages of previous geotechnical investigation are also compiled in this report. Four (4) boreholes were put down on, or adjacent to, the subject site in a 2011 investigation stage. More recent 2015 and 2016 stages of investigation are represented by seven (7) and three (3) boreholes, respectively, drilled in a manner to reduce disturbance to in situ testing and allow for the recovery of relatively undisturbed samples. These investigative works are all described in this report.

The purpose of the current report is to provide final geotechnical recommendations pertaining to the proposed high school development, based on the results of the current and previous test holes and associated field and laboratory testing.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 Proposed Development

The proposed Barrhaven High School will consist of a one to three storey steel-frame slab-on-grade building. A gymnasium will be part of the school building. The school development will be serviced by municipal services and will incorporate sports fields, a race track, and parking areas, roadways and fire lanes.

The subject high school site is bordered to the north by Strandherd Road, to the west by future residential development, to the east by Chapman Mills Drive and then residential development lands, and to the south by an undeveloped rapid transit corridor and then a tributary of the Jock River.

The subject subject school parcel consists of cleared land, previously used for agricultural purposes. The property was originally relatively flat, with the ground levels in the north part of the development parcel about 0.5± m higher than those in the south. As part of site activities, there are fill deposits over part of the site, including several small piles of fill materials, as well as levelled fill materials.

3.0 Scope of Current and Previous Investigation Work

Paterson has conducted current and previous investigative work on and adjacent to the subject development site. Those investigative works that were conducted on or directly adjacent to the school parcel have been included in this compilation geotechnical investigation report for the geotechnical evaluation of the Clarke Lands school parcel.

The factual geotechnical investigation and testing work from the current and previous investigations that is included with this geotechnical report, is as follows:

❏ Six (6) boreholes have been put down specifically for the subject development in 2019 in the location of the high school building.
❏ Three (3) of the boreholes put down during the 2016 geotechnical investigation program for a larger parcel are included in this report, in Appendix 1.
❏ Seven (7) of the boreholes put down during the 2015 geotechnical investigation program for a larger parcel are included in this report, in Appendix 1.
❏ Four (4) of the boreholes put down during the 2011 geotechnical investigation program for a larger parcel are included in this report, in Appendix 1.
❏ Applicable consolidation testing conducted as part of the 2016 (3 tests), 2015 (7 tests) and 2011 (8 tests) geotechnical investigations have been included. The consolidation test results are all provided in Appendix 2.

- ❑ An updated summary of subsurface information, from the current and previous boreholes, is provided in Table 1, in Appendix 2.
- ❑ A summary of the consolidation test results is provided in Table 2, Appendix 2.
- ❑ An updated summary of the groundwater readings is provided in Table 3, in Appendix 2.

The purpose of this final geotechnical investigation report for the Barrhaven High School development has been to:

- ❑ Present the results of our current and previous investigative findings in a comprehensive stand-alone geotechnical report for the proposed Barrhaven High School development.
- ❑ Provide final geotechnical recommendations for the design and construction of the various aspects of the proposed development, including foundation design, slab-on-grade support, considering permissible grade raises, and servicing and pavement design.

4.0 Method of Investigation

4.1 Field Investigation

The current (2019) and all the previous (2011, 2015 and 2016) boreholes were put down using track-mounted CME-55 power auger drill rigs operated by a crews of two. Each stage of fieldwork was conducted under the full-time supervision of a staff member of our geotechnical division under the direction of a senior engineer.

The drilling procedure, for the current and the previous 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 previous 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. No Shelby tube samples were recovered as part of the current investigation, as there was no need to conduct further consolidation testing. 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. At present there is no ASTM standard for the DCPT.

For the current 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 BH5-19, the auger refusal was encountered at too shallow a depth to be considered accurate. At BH4-19 and BH6-19, 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 BH2-19 and BH3-19 a cone was driven to practical refusal below the depth of auger refusal, and was able to penetrate further through overburden. At BH1-19, 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 from the applicable boreholes for the high school parcel are presented 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.

4.2 Field Survey

All the 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 locations and ground elevations of the current (2019) and previous (2011, 2015 and 2016) boreholes are presented, along with the final site plan elements, on Drawing PG3968-1, Revision No. 1, in Appendix 3.

4.3 Laboratory Testing

All samples were returned to our geotechnical laboratory for further perusal and testing. The split spoon and auger samples 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 2 and are discussed under subsections 6.3 and 6.4 of this report. Atterberg Limits test results are also provided in Appendix 2.

5.0 Observations

5.1 Surface Conditions

The Barrhaven High School parcel consisted of cleared level land, recently used for agricultural purposes. The property was originally relatively flat, with the ground levels in the north part of the development parcel about 0.5±m higher than those in the south. It has since been subjected to the placement and stockpiling of unselected fill materials. Reference should be made to Table 1, in Appendix 2, for the present ground elevations and the interpreted uppermost inorganic soil original ground elevations, below the fill materials, at the borehole locations.

5.2 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 current boreholes, as well as 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 current boreholes. The topsoil or cultivated soil layer appears 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 current 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 current 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 current and 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.

The consolidation test results indicate that the silty clay is overconsolidated with overconsolidation ratios for the tested samples (with acceptable disturbance ratios) varying between 1.4 and 2.3. For purposes of comparison and analyses, it is assumed that the clay crust thickness (ignoring existing fill) is 3 m and the low pre-development groundwater level is 2.5 m.

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 BH1-19 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 3, and summarized in Table 1, in Appendix 2. 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 BH5-19, 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.

5.3 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 current measured groundwater levels 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 BHs 2-15 and 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 BHs 3-19 and 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.

6.0 Discussion and Recommendations

6.1 Geotechnical Assessment

The Barrhaven High School development will include a 1 to 3 storey steel-frame slab-on-grade building, with a gymnasium component, as well as servicing and playground, parking, access and fire lanes. Design proceeded on the development based on the previous compilation report, so this report has the benefit of being able to target recommendations to the actual development characteristics.

The subsoil conditions at this site consist of a deep stiff to firm silty clay deposit. From a geotechnical viewpoint, the development of the subject lands will require grade raises, dictated by the hydraulic grade line from the nearby Jock River 1:100 flood plain elevation. The finished floor elevation (FFE) of the proposed high school building has been set at 94.60 m. This FFE represents an estimated grade raise of between 1.8 and 2.5 m.

With the proposed FFE, the weight of the grade raise fill uses all the available 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. In addition, lightweight fill (LWF) will be required within the building area to reduce the potential for slab-on-grade settlement. The geotechnical considerations to site development are further discussed in the following sections.

6.2 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 may have to be removed and replaced with compactive effort if it is to remain under pavements or other structures.

With respect to the granular fill working platform, within the building, it is has been proposed by the architect that the subgrade level for a 0.6 m thick granular layer will be 92.4 m. Our interpretation of the inorganic soil levels are summarized in Table 1, in Appendix 2. It is expected that existing fill remnants will be encountered below the elevation 92.4 m routine subgrade level within much of the building footprint.

It is recommended that one of the following alternatives be implemented to deal with the existing fill. The preferred alternative would be to prepare an inorganic in situ soil subgrade under the entire building footprint, subexcavating where necessary below the 92.4 m routine subgrade level and filling the subexcavations with Granular B Type II.

Another alternative would be to excavate to the routine 92.4 m level and then evaluate 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.

Fill Placement

Fill used for grading beneath building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II, Granular B Type I or select subgrade material. This material should be tested and approved prior to delivery to the site. Testing may consist of the suppliers own Quality Control testing, or samples can be submitted to the geotechnical consultant for testing. Initial acceptance testing can consist of gradation analyses and comparison to OPSS MUNI 1010 (Nov, 2013) gradation limits.

The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Engineered fill placed beneath the buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD) value. Engineered fill placed below the subgrade level for pavements should be compacted to at least 95% of its SPMDD value. The materials comprising the pavement structures should be compacted to at least 100% of their SPMDD values.

The laboratory testing reference for the specified density is ASTM D698-07 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

Site-excavated soil, along with non-specified existing fill, can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If site-excavated soil materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. These materials should meet the requirements for Select Subgrade Material (SSM) under OPSS MUNI 1010 (Nov, 2013). Non-specified existing fill and site-excavated soils, are not suitable as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

6.3 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.

The proposed FFE of the building, at elevation 94.6 m, will require the use of lightweight fill (LWF) instead of granular fill for a portion of the fill materials under the building slab-on-grade.

Light and settlement-tolerant buildings, such as future portable classrooms, can be supported by shallow foundations, such as footings, pads or sills over inorganic native soils or granular fill over native soils.

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 5, 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 5
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 4th 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 a factor of safety of 1.5 to 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 high school building 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 el. 94.6 m will add a sustained load to the soil. For the case where conventional granular fill materials are used, the estimated grade raise of 1.8 to 2.5 m would impart a fill weight stress of the order of 40 to 50 kPa. The ground floor slab of the high school will also have a design live load of 4.8 kPa, of which 50% is representative of the sustained load component.

It is our interpretation that a portion of the conventional fill materials under the slab-on-grade should be replaced with lightweight fill (LWF) material, consisting of expanded polystyrene (EPS) geofoam to reduce the stress and, in turn, reduce the potential for slab-on-grade settlement. More information concerning the geofoam is provided under subsection 6.5 of this report.

All load-bearing walls should be supported on pile foundations. Light partition walls may be suitable for support on slab thickening footings over the subslab fill. The stresses from these slab-supported walls should be distributed sufficiently to not overstress the geofoam.

For example, by using 0.9 m of LWF under the building floor slab, and for a specified distance around the perimeter of the building, the 18 kPa of stress reduction from the grade raise fill can be used to accommodate stresses from footing type foundations.

Footing-Supported Structures

Light and settlement-tolerant buildings, such as future portable classrooms, can be supported by shallow foundations, such as footings, pads or sills over inorganic native soils or granular fill over native soils. These foundations can be designed using a bearing resistance at SLS (serviceability limit states) value of 75 kPa and a factored at ULS (ultimate limit states) value of 120 kPa.

6.4 Seismic Design

The proposed Barrhaven High School building structures is required to be designed according to the requirements of Part 4 of the OBC 2012. This requires that the seismic provisions under Part 4 of the OBC 2012 be considered.

The Barrhaven High School parcel should be classified as Site Class D. The inferred bedrock depths do not exceeding 17 m below the underside of pile cap level. Assuming a conservative shear wave velocity of 120 m/s for the grey clay and 300 m/s for the stiff clay crust and glacial till overburden, and a shear wave velocity of 1,500 m/s for the sandstone and dolomite bedrock, the average shear wave velocity of the upper 30 m profile, Vs_{30}, is estimated to be well in excess of **180 m/s** throughout the Barrhaven High School parcel.

Figure 2, in Appendix 2, provides a summary of a conservative estimate of the Vs_{30}, of 292 m/s, for conditions at the Barrhaven High School parcel, which is within the range of 180 m/s and 360 m/s for Site Class D. Therefore, a seismic **Site Class D** is applicable for the Clarke Lands secondary school development, as per Table 4.1.8.4.A of the OBC 2012.

6.5 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(s) should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the SPMDD.

It is recommended that a 0.9 m thickness of EPS LWF (expanded polystyrene foam lightweight fill) be provided under the entire slab-on-grade for the school. The bottom of the EPS blocks should be well bedded to ensure good support. Where a bedding layer such as sand or crushed screenings is used, a non-woven geotextile may be needed to ensure that the fines from the bedding layer do not migrate into coarser fill materials below. EPS blocks should be butted together using machine cut faces, where possibile, to make a tighter fit. A non-woven geotextile should be placed over the top of the EPS layer to prevent fines from overlying materials from migrating into the joints between the EPS blocks.

Where EPS geofoam LWF is used under the floor, as a replacement for engineered granular fill, the strength of the EPS should be chosen to ensure that the stresses from the overlying fill, the sustained portion of live loads and from any slab-thickening footings, supporting non load-bearing walls, do not exceed the compressive resistance at 1% deformation of the EPS. When sustained loads are kept at or below this value, the EPS will not be susceptible to long-term creep-type deformation.

EPS Type 15 has a compressive resistance at 1% deformation of 25 kPa, and should be suitable for cases where there is a thin granular layer, the slab-on-grade and a portion of the live load over the geofoam and the sustained load is expected to be of the order of 20 kPa.

Where there may be non-load-bearing partition walls to be supported on the slab-on-grade (usually with slab thickenings), the stresses imparted to the EPS from the self weight of these structure should be added to the above-noted 20 kPa value. It is recommended that a 1H:2V stress distribution be used in the fill under the slab thickening and over the EPS to estimate the added stress on the EPS. It is estimated that where partition walls will be supported on the ground floor slab, the increased sustained loading will bump up the sustained stress on the EPS to more than 30 kPa. For these conditions, the use of EPS Type 19, with a compressive resistance at 1% deformation of 40 kPa would probably be suitable.

6.6 Site Servicing

Trench Stability

Trench Support

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

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

Basal Stability

There is a potential for basal heave to occur in deep excavations in firm to soft clay at the site. Our calculations indicate that there is a factor of safety against base heave of 2 for cuts of up to 5.5 metres in depth, and a factor of safety of 1.5 for cuts of 7.5 metres in depth for clays with a shear strength of 25 kPa. Generally minimum measured shear strengths are in excess of 25 kPa.

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

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

Trenching, Supporting and Backfilling Procedures

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

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

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

☐ Employ benching techniques to reduce the effective trench depth.

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

☐ Keep unsupported trench lengths as short as practical.

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

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

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

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

Trench Dewatering

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

Sewer Installation

Pipe Bedding

The bedding for the proposed sewer installations should be constructed according to Ontario Provincial Standard Specifications (OPSS), as amended by City of Ottawa specifications. Class "B" granular bedding is suitable for the support of sewer pipes in trenches placed through the in situ soils. The same material should be used up to the spring line of the pipe. The pipe cover material (i.e. 300 mm or greater above the pipe) should consist of an imported granular backfill material having a maximum stone size no greater than 25 mm.

All granular backfill materials should be uniformly compacted to a minimum of 95 percent of their Standard Proctor maximum dry density (SPMDD) values. However, the uniformity of the density of the bedding and cover materials, especially the bedding under the pipe invert and haunches is often more important to optimizing the structural performance of the pipe as whether the target density is achieved.

Seepage Barriers - Clay Seals

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

The clay seals should be as per Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services Branch, of the City of Ottawa. The basic standard drawing requires a minimum seal length of 1.0 m, but it is recommended by this firm that the seals should be at least 1.5 m long (in the trench direction).

The seals should extend from trench wall to trench wall and should extend from the underside of the road structure, and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay from the crust placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the material's SPMDD.

The clay seals should be placed at the site and/or development boundaries and at strategic locations at no more than 60 m intervals in the service trenches. The seals should not be located at pipe joint locations.

Re-Use of Site Excavated Material

The excavated soils can be used as backfill in the trenches above the engineered pipe cover material. This material should be sufficiently compacted to ensure that the voids are expelled from the soil mass. The ability to compact the silty clay soils will depend in large part on the moisture content of the silty clay soil. Silty clay and clayey silt should be compacted using a padfoot equipment.

The planned use of the site excavated clay as backfill should take into consideration that clay worked in the presence of free water (i.e. rainy weather) can become difficult to manage as backfill material. The upper brown silty clay is preferred to the underlying wetter grey silty clay for use as trench backfill.

The use of site excavated trench backfill will require closer supervision of the backfilling operations, as compared to the use of OPSS Granular B Type I backfill. The compactive effort required to compact the in situ soil and fill is expected to be comparable to that required to compact OPSS Granular B Type I material, provided groundwater influx is not a problem. Taking this into consideration, the use of site excavated soils as trench backfill is a viable alternative, in our opinion.

In areas where service trenches are located within 3 metres of residential foundations, such as rear yard catch basin leads, it will be necessary to backfill the portions of the trench below the foundation level with imported granular materials, ensuring that a compaction of 95 percent or more of its Standard Proctor maximum dry density (SPMDD) is achieved.

Watermain Construction

Bedding and Backfill for Watermains

The bedding and backfill for the proposed watermain installations should be constructed according to Ontario Provincial Standard Specifications (OPSS), as amended by City of Ottawa specifications. The bedding conditions for watermains in the in situ soils within the study area are considered to be favourable. Class "B" bedding is suitable for the support of watermains in trenches through the in situ soils.

Site excavated soils are considered suitable for re-use as backfill for trenches through soil. Reference can be made to the native backfilling recommendations for sewers for guidelines for the watermain trench backfilling operations.

Thrust Blocks

The details of standard thrust blocks are shown on Standard Drawing Nos W25.3 and W25.4 of the City of Ottawa. The blocking details provided require typical soil bearing capacities of 100 to 199 kPa for clays. Because of geotechnical considerations, however, the bearing capacity is dependent on the direction of application of the load.

Thrust blocks resisting lateral loads should be sized on a pro-rata basis (as compared to the 100 to 199 kPa range) using the standard drawings and the allowable lateral soil bearing capacity. Those resisting (downward) vertical loads should be sized based on the allowable vertical soil bearing capacity.

Within the upper stiff silty clay soils, an allowable lateral soil bearing capacity of 125 kPa can be used, based on a soil cover of at least 2.4 metres (below finished grade), and an allowable vertical soil bearing capacity of 150 kPa can be used, which conform to Table 1 in W25.4. Within the deeper, firm to soft soils, an allowable lateral soil bearing capacity of 60 kPa can be used, based on a soil cover of at least 2.4 metres, and an allowable vertical soil bearing capacity of 80 kPa can be used, which requires increasing the dimensions from Table 1 of W25.4.

6.7 Pavement Design

With the complete removal of all topsoil and any loose and/or organic existing fill, the uppermost in situ soils are suitable subgrade media for roadway construction.

Recommended pavement material thicknesses for car parking and access/fire lanes are provided in Tables 6 and 7, on the following page.

All granular base and subbase materials are required to be compacted to a minimum of 100 percent of Standard Proctor maximum dry density (SPMDD). Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

Table 6: Recommended Pavement Material Thicknesses
Medium Duty - Car Parking Areas

Depth in Pavement Profile (mm)		Layer Thickness (mm)	Pavement Material Description
From	To		
0	50	50	**Wear Course:** SP 12.5 Asphaltic Concrete
50	200	150	**BASE:** OPSS Granular A
200	500	300	**SUBBASE:** OPSS Granular B Type II
500	500+	---	**SUBGRADE:** In situ soil and/or well compacted inorganic existing fill and/or native trench fill materials.

Table 7: Recommended Pavement Material Thicknesses
Heavy Duty - Access Roads and Fire Lanes

Depth in Pavement Profile (mm)		Layer Thickness (mm)	Pavement Material Description
From	To		
0	40	40	**Wear Course:** SP 12.5 Asphaltic Concrete
40	90	50	**Binder Course:** SP 19.0 Asphaltic Concrete
90	240	150	**BASE:** OPSS Granular A
240	640	400	**SUBBASE:** OPSS Granular B Type II
640	640+	---	**SUBGRADE:** In situ soil and/or well compacted inorganic existing fill and/or native trench fill materials.

It can be foreseen that, depending on the time between the installation of services and the ensuing pavement construction, localized or extensive soft areas could be present over service trenches. In these localized areas, it may be necessary to subexcavate deleterious materials, and replace them with suitable, well compacted, OPSS select subgrade material and/or imported granular fill, providing tapers or transition point treatment, where applicable.

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

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

7.0 Design and Construction Precautions

7.1 Foundation Drainage and Backfill

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

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

The use of a foundation drainage system is mandatory for cases where EPS LWF is used under the floors of the building in order to ensure that buoyancy of the EPS does not occur, with associated lifting of the overlying fill and floor slab.

7.2 Protection of Footings Against Frost Action

Perimeter pile caps and grade beams 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 and grade beams, such as those for isolated exterior piers and wing walls may require more soil cover or a combination of soil cover and insulation.

7.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the building excavations to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to minimize the trench width for services, the excavation can be carried out within the confines of a fully braced steel trench box or other acceptable shoring systems.

Unsupported excavation side slopes, extending to a maximum depth of 3.0 m, should be cut back at 1H:1V, or shallower. Unsupported slopes in excess of 3.0 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

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.

7.4 Groundwater Control

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

It is anticipated that groundwater infiltration into the excavations should be low to moderate to high depending upon depth of excavation. Pumping from open or cased sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The use of non-woven geotextile and clear stone may be necessary to control silt and prevent clogging of submersible pumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of 2 to 4 weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

7.5 Precautions for EPS LWF Material

The use of the EPS LWF requires certain precautions to be considered, including the following. After the driving of the piles and before or after the construction of the pile caps and grade beams, the Granular B Type II working pad should be scarified, regraded and compacted. It also may be necessary to locally remove and replace soil-contaminated granular fill. As and end result, the EPS blocks should be well bedded on the granular fill.

Field-cutting of the EPS blocks should be limited and machine (manufacturer) cut surfaces should be used wherever possible to provide the best block to block fit. In order to prevent the loss of fines into cracks between the EPS blocks, the use of a non-woven geotextile (Terrafix 270R or equivalent) sheet over the EPS should be considered.

The installation of building services should be coordinated with the EPS placement, as it will not be practical to complete the EPS placement and then have a plumber come in after with a backhoe and arbitrarily dig through the EPS for pipe runs. The presence of the granular fill over the EPS should alleviate much of this issue except on long pipe runs.

The EPS should be properly stored on site to prevent damage. EPS deteriorates under prolonged UV exposure. EPS should be kept away from hydrocarbons, such as fuel, that will act as a solvent.

7.6 Winter Construction

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

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

Trench excavation and backfilling and pavement construction are also difficult to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Additional information could be provided, if required.

7.7 Corrosion Potential and Sulphate

Two soil samples from the current investigation have been submitted for analytical testing for corrosion potential and sulphate. The test results were not available at the time of this report, and will be submitted under separate cover. The results of analytical testing conducted on samples recovered from elsewhere in the development show that the sulphate content is less than 0.1% (1 mg/g) in all the samples. This result is indicative that Type GU (general use) cement, as per CSA A23.1 Section 4.2.1.1.2, is appropriate for this site.

The chloride content of the samples are less than 400 mg/g and the pH of the samples is greater than 5. These results indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site. However, the tested resistivity values are indicative of an aggressive to very aggressive corrosive environment for ferrous metals at this site.

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.

7.8 Landscaping Considerations

The subject site is located in an area of moderately sensitive marine silty clay (SMC) deposits with regards to tree planting. The Atterberg Limits test results indicate that the plasticity index is generally below 40%, so the silty clay soils should be considered to be within the low to medium plasticity range with respect to tree planting restrictions. For the proposed development, it is expected that final grade raises will be approximately between 1.5 to 2.5 m above original ground surface grades. Therefore, it is expected that the combination of the proposed finished grades and the thickness of the underlying weathered clay crust will provide approximately 4 to 5 m thick buffer to the underlying firm grey silty clay deposit.

In our opinion, tree planting for this subject development should be limited to low water demand trees. The minimum permissible distance from the foundation will depend on the nature of the tree, the depth of the clay crust and the final grade raise in relation to the permissible grade raise. For low water demand trees, the minimum permissible distance can be set at 4.5 for this site.

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

8.0 Material Testing and Observation Services

The following geotechnical review, material testing and field review observation program should be performed by the geotechnical consultant:

☐ Geotechnical review of any revisions to the project grading plan.

☐ Full time field review services during the driving of piles, including recording pile lengths, refusal criteria, relative pile location and plumbness, and reviewing the results of dynamic monitoring (by others).

☐ Observation of all bearing surfaces prior to the placement of concrete for footings, where footings are used.

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

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

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

☐ Observation of all subgrades prior to backfilling and follow-up field density tests to ensure that the specified level of compaction has been achieved.

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon demand, based on the completion of a satisfactory materials testing and field review observation program by the geotechnical consultant.

9.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be retained to review the grading plans, and foundation plans, once prepared, for conformance to our recommendations.

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The 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 the Conseil des écoles publiques de l’Est de l’Ontario or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

PATERSON GROUP INC.

AJ Tovell

LICENSED PROFESSIONAL ENGINEER
Apr. 09-19
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS:

BH1-19 to BH6-19, Inclusive (2019)
BH2-15 to BH8-15, Inclusive (2015)
BH13-16, BH14-16, BH 14A-16 and BH23-16 (2016)
BH7 to BH10, Inclusive (2011)

Symbols and Terms

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.

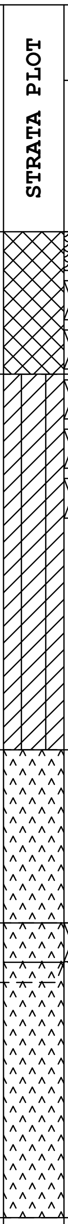
Inferred **GLACIAL TILL**

15.22

End of Borehole

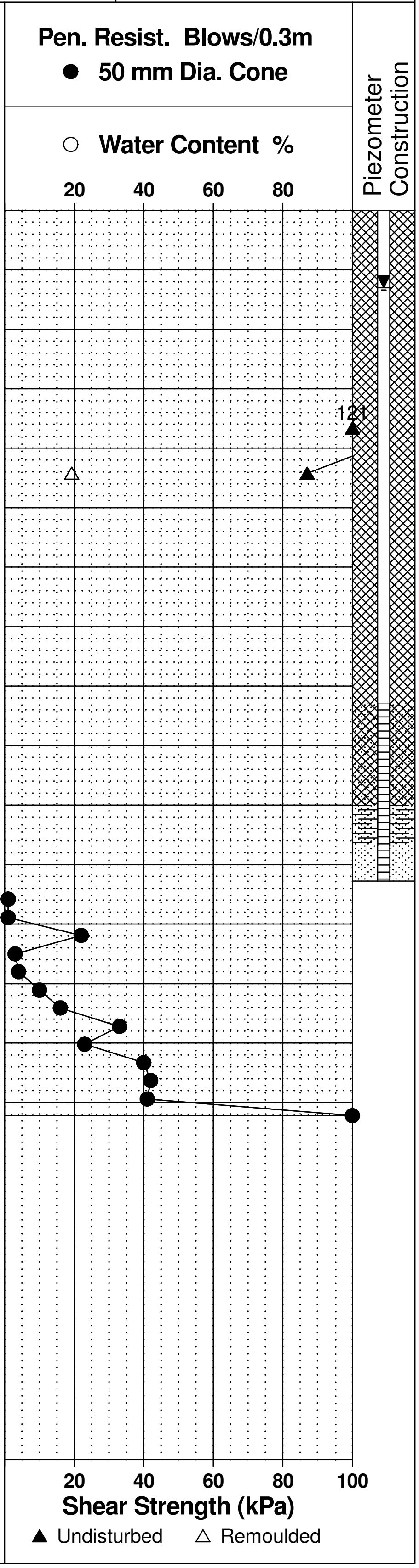
Practical DCPT refusal at 15.22m depth

(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



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

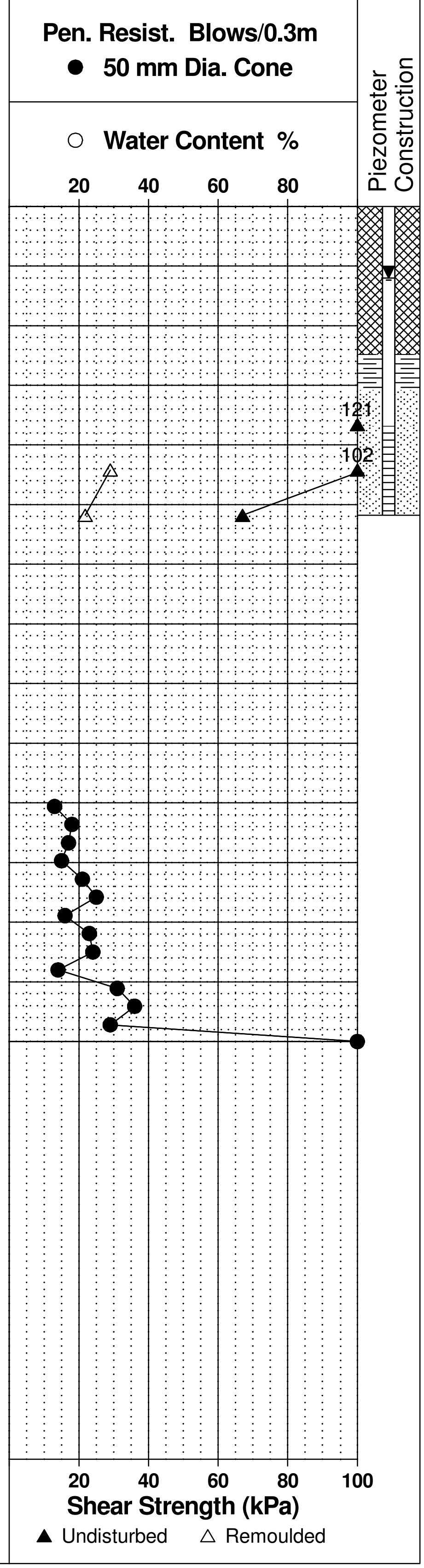
End of Borehole

Practical DCPT refusal at 14.00m depth

(GWL@ 1.20m - March 29, 2019)

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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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

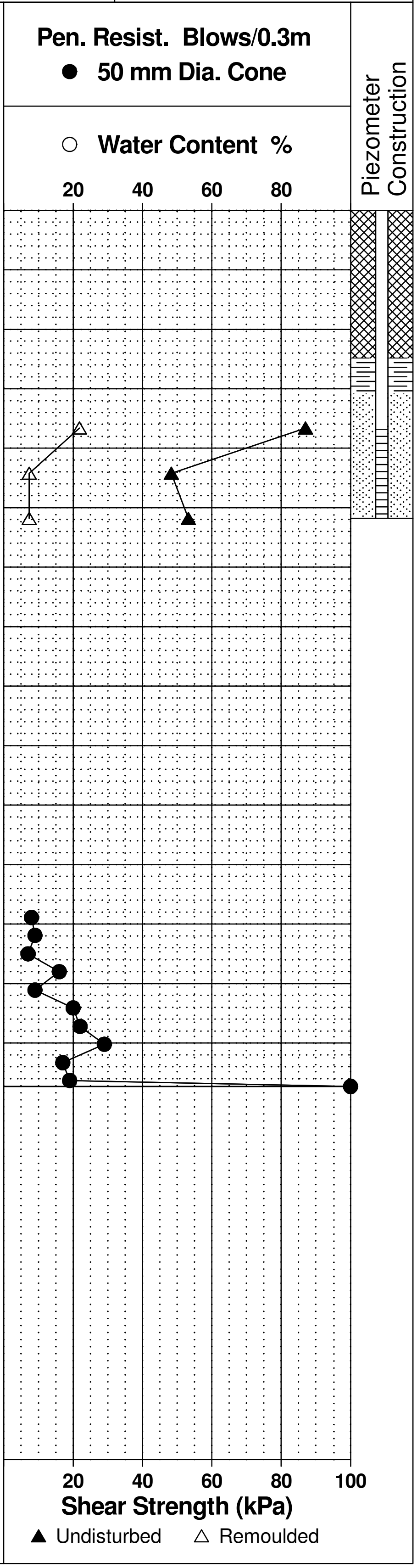
DATE March 22, 2019

FILE NO. PG3968

HOLE NO. BH 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel, cobbles and organics		AU	1		
1.20		SS	2	58	9
Stiff, brown **CLAYEY SANDY SILT**		SS	3	75	5
2.30					
Stiff, brown **SILTY CLAY**		SS	4	100	3
		SS	5	100	2
- grey by 3.8m depth		SS	6	100	2
5.18		SS	7	100	2
Dynamic Cone Penetration Test commenced at 5.18m depth. Cone pushed to 11.3m depth. Inferred **SILTY CLAY**					
11.30					
Inferred **GLACIAL TILL**					
14.73					
End of Borehole Practical DCPT refusal at 14.73m depth (Piezometer blocked - March 29, 2019)					

DEPTH (m)	ELEV. (m)
0	93.36
1	92.36
2	91.36
3	90.36
4	89.36
5	88.36
6	87.36
7	86.36
8	85.36
9	84.36
10	83.36
11	82.36
12	81.36
13	80.36
14	79.36



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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

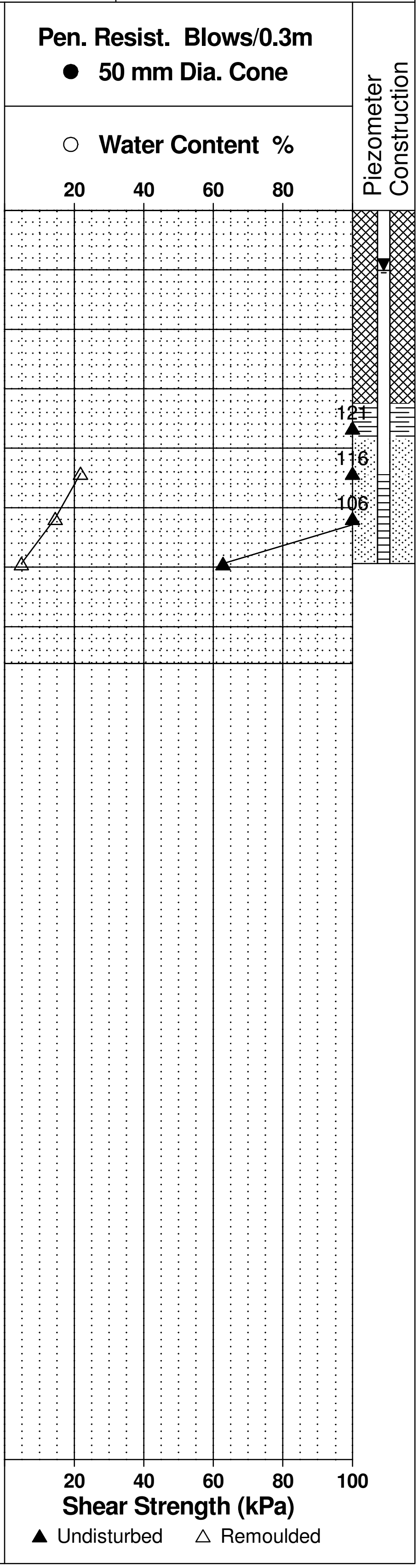
SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand, some clay, gravel, cobbles and boulders, trace organics 1.50	AU	1		
	SS	2	67	50+
Brown **CLAYEY SANDY SILT** 2.95	SS	3	38	9
	SS	4	58	14
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m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

End of Borehole

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

(GWL @ 1.01m - March 29, 2019)

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



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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

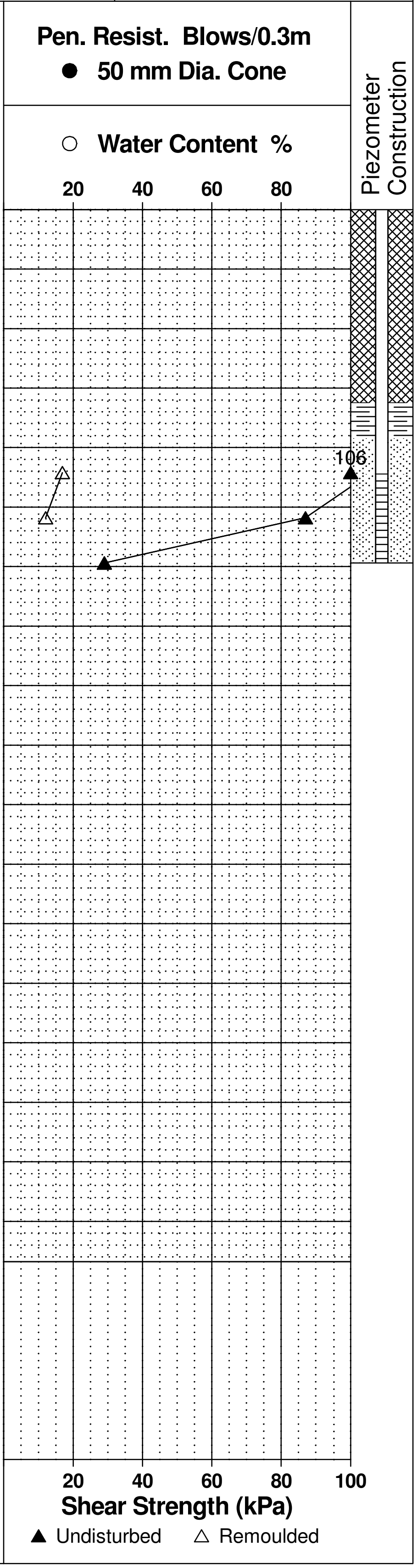
DATE March 20, 2019

FILE NO. PG3968

HOLE NO. BH 5-19

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel and clay, some cobbles	AU	1		
	SS	2	67	27
	SS	3	50	50+
2.30				
Brown **CLAYEY SANDY SILT**	SS	4	29	11
2.95				
Very stiff to stiff, brown **SILTY CLAY**	SS	5	62	7
	SS	6	100	2
- grey by 5.0m depth	SS	7	100	2
	SS	8	100	2
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)				

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



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

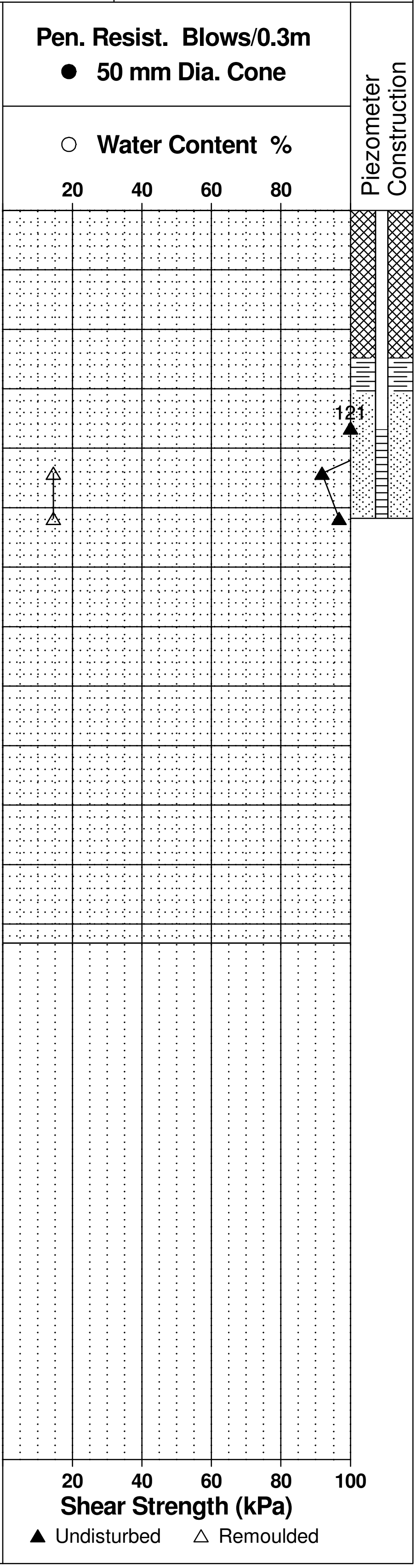
HOLE NO. BH 6-19

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel, trace clay	AU	1		
	SS	2	21	14
1.35 — Brown **CLAYEY SANDY SILT,** trace roots	SS	3	29	11
2.30 — Very stiff to stiff, brown **SILTY CLAY**	SS	4	79	6
	SS	5	100	2
- grey by 3.8m depth	SS	6	38	2
5.18	SS	7	100	2
Borehole augered from 5.18 to 12.32m depth.				
Inferred **SILTY CLAY**				
11.60 — Drill chatter commencing at 11.6m depth				
12.32 — Inferred **GLACIAL TILL**				
End of Borehole				

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

(Piezometer sunk below grade - March 29, 2019)

DEPTH (m)	ELEV. (m)
0	93.95
1	92.95
2	91.95
3	90.95
4	89.95
5	88.95
6	87.95
7	86.95
8	85.95
9	84.95
10	83.95
11	82.95
12	81.95



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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**

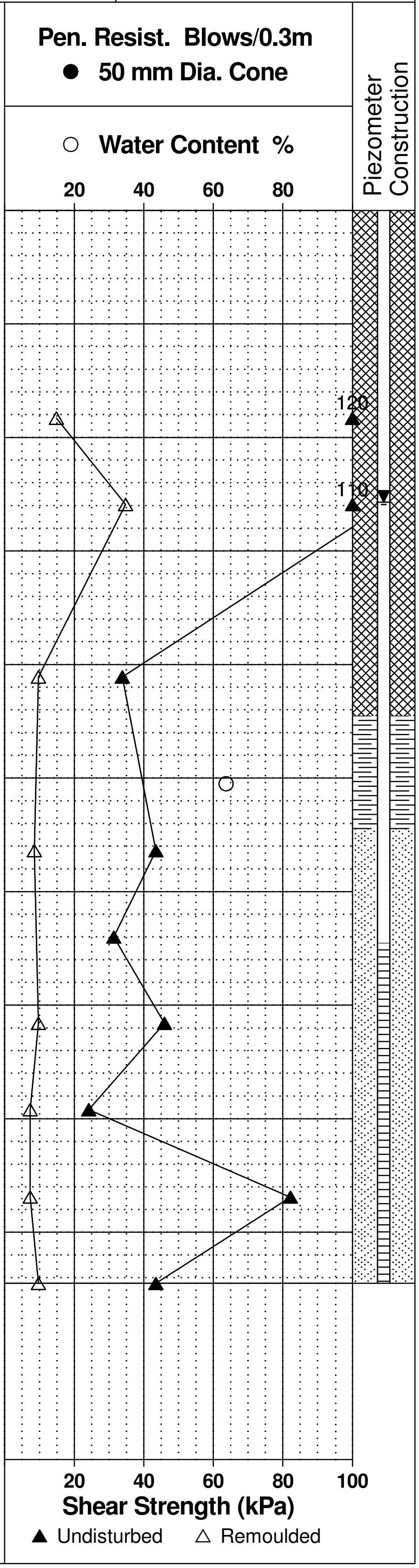
REMARKS

HOLE NO. **BH 2-15**

BORINGS BY CME 75 Power Auger

DATE March 4, 2015

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

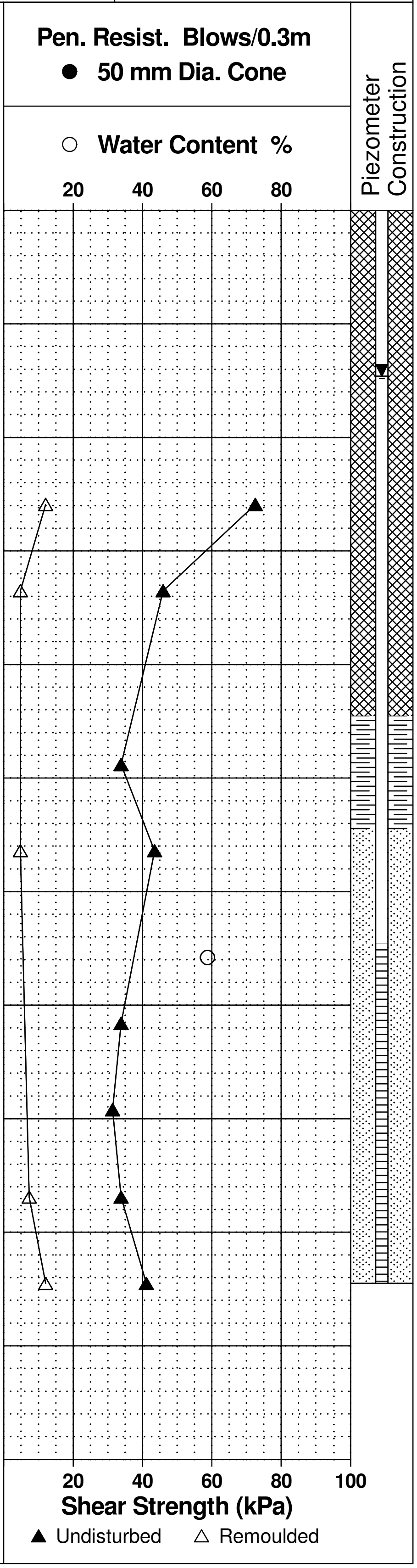
REMARKS

HOLE NO. BH 3-15

BORINGS BY CME 75 Power Auger

DATE March 5, 2015

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



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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

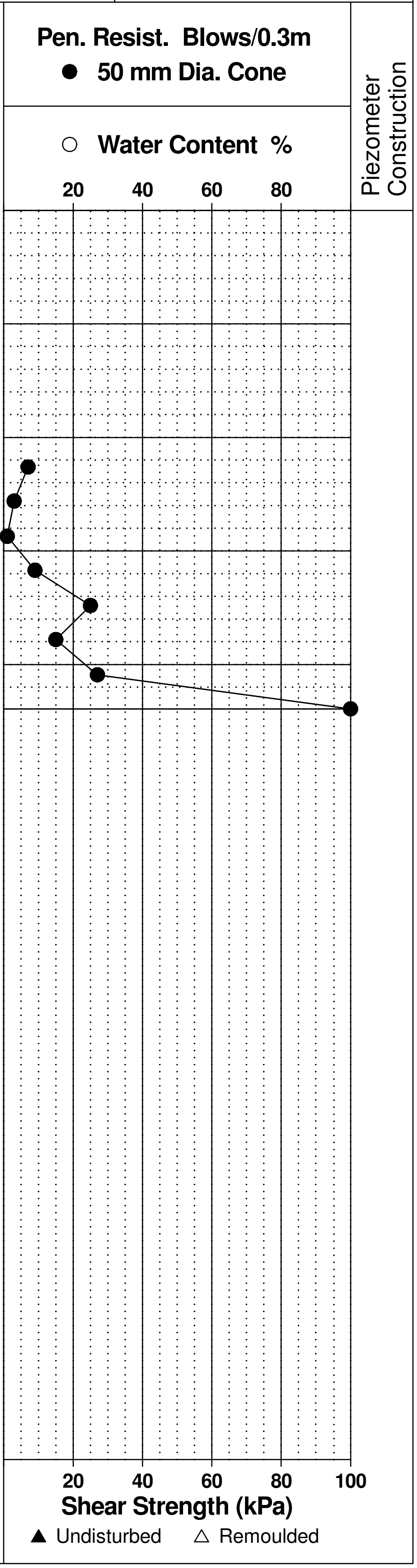
REMARKS

HOLE NO. BH 3-15

BORINGS BY CME 75 Power Auger

DATE March 5, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
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

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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

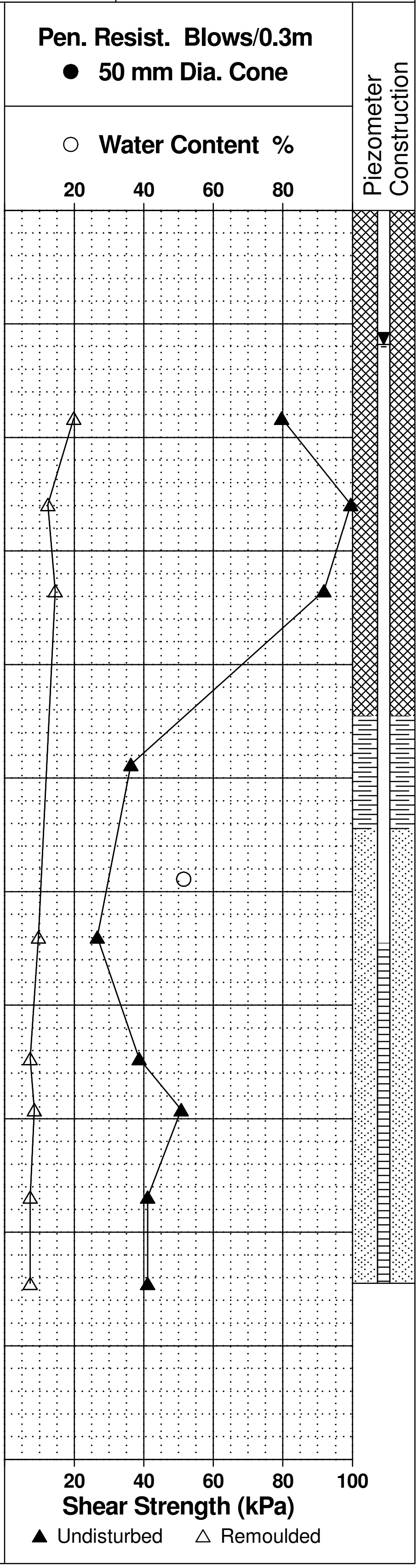
DATE March 5, 2015

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					
Very stiff to stiff, brown **SILTY CLAY**	▨	AU	1		
	▨	SS	2	100	6
3.40					
Firm to stiff, grey **SILTY CLAY**	▨	TW	3	100	
	▨	TW	4	92	
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



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

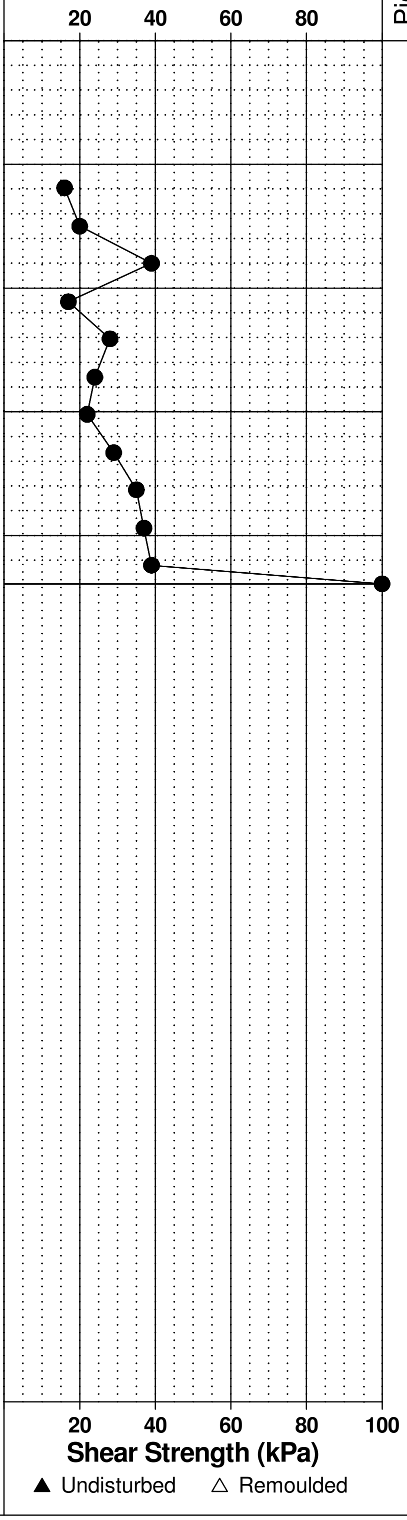
BORINGS BY CME 75 Power Auger

DATE March 5, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						11	80.84
						12	79.84
Inferred firm to stiff cohesive soil over compact to dense glacial till						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)							

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

○ Water Content %



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

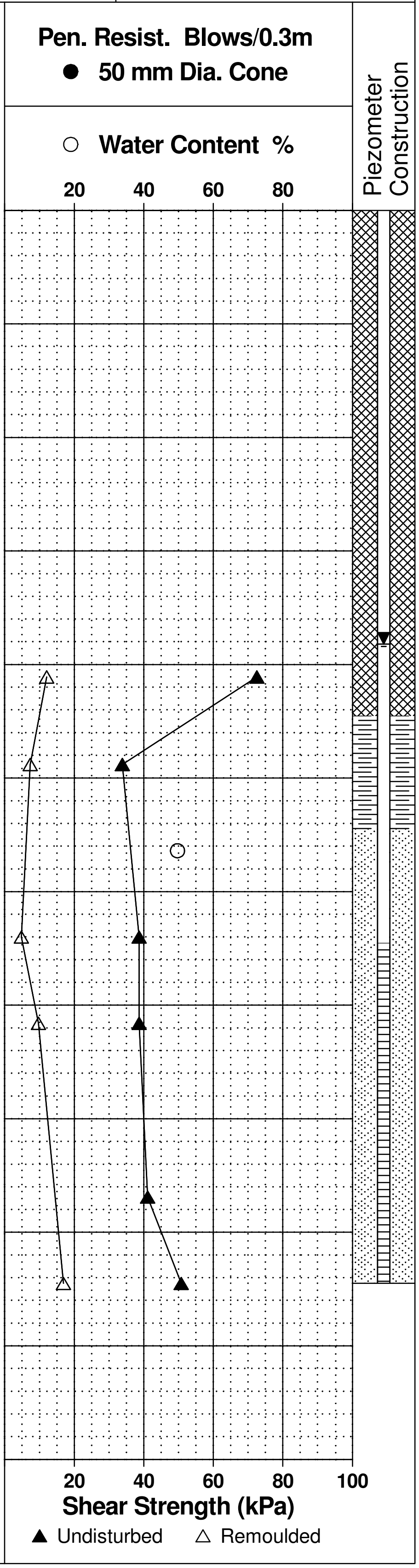
REMARKS

HOLE NO. BH 5-15

BORINGS BY CME 75 Power Auger

DATE March 6, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	93.44
FILL: Brown silty clay, some sand and cobbles		AU	1				
		SS	2	75	9	1	92.44
1.42							
Very stiff to stiff, brown **SILTY CLAY,** some sand		SS	3	100	12		
						2	91.44
		SS	4	100	8		
						3	90.44
		SS	5	100	5		
						4	89.44
4.50							
Firm to stiff, grey **SILTY CLAY**						5	88.44
		TW	6	97			
						6	87.44
						7	86.44
		TW	7	98			
						8	85.44
						9	84.44
9.45							
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 11.0m depth.						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**

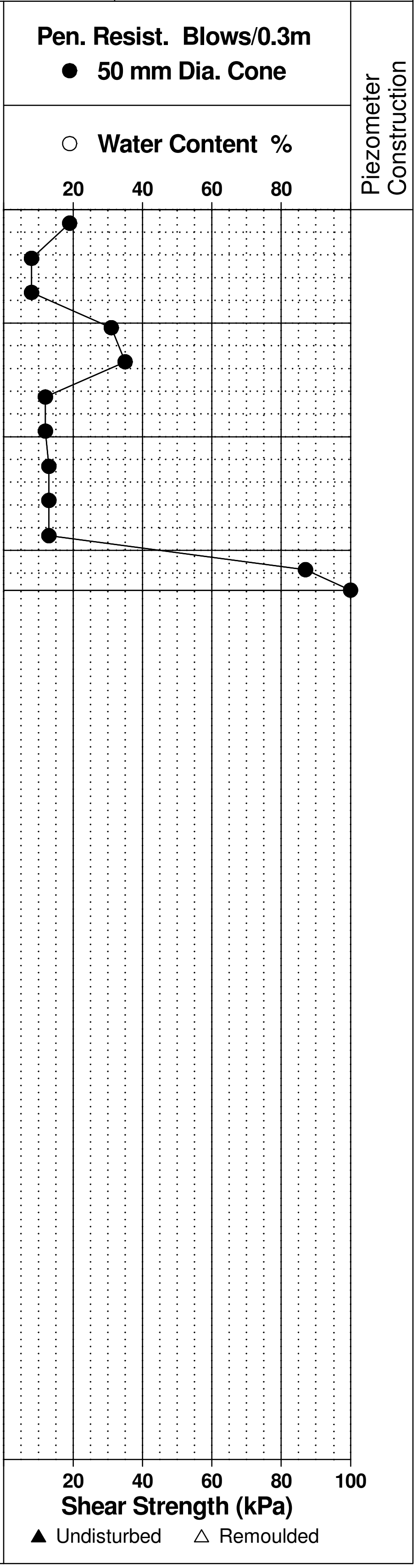
REMARKS

HOLE NO. **BH 5-15**

BORINGS BY CME 75 Power Auger

DATE March 6, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						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)							



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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 6-15**

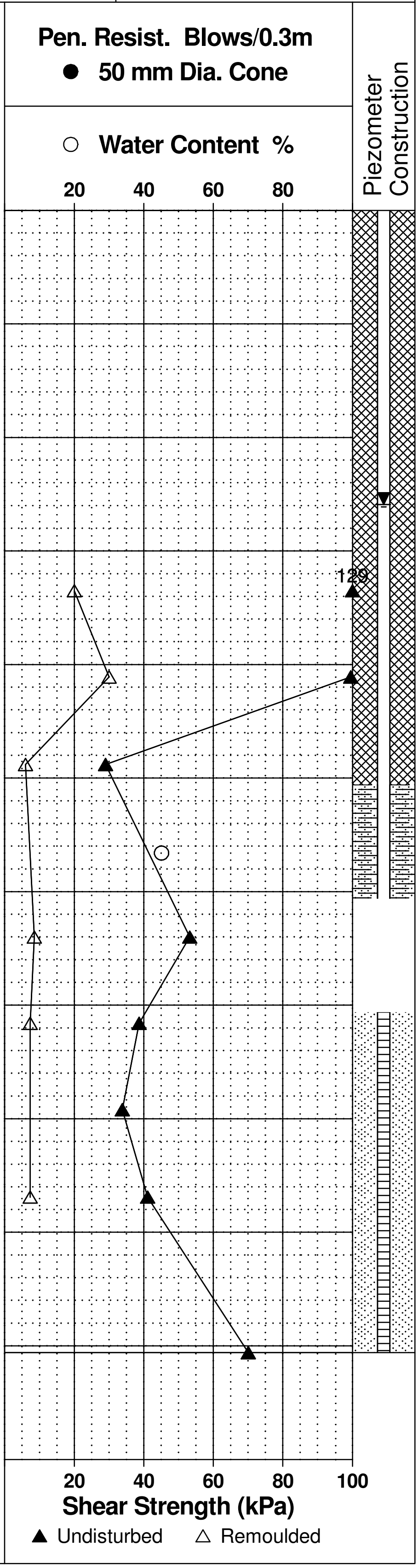
BORINGS BY CME 75 Power Auger

DATE March 6, 2015

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, trace cobbles	1.45
FILL: Brown silty clay, some sand and gravel	1.98
Very stiff to stiff, brown **SILTY CLAY,** some sand	4.60
Firm, grey **SILTY CLAY**	10.06
End of Borehole	
(GWL @ 2.59m-March 19, 2015)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	60	50+
SS	3	71	12
SS	4	75	7
TW	5	95	
TW	6	100	

DEPTH (m)	ELEV. (m)
0	94.30
1	93.30
2	92.30
3	91.30
4	90.30
5	89.30
6	88.30
7	87.30
8	86.30
9	85.30
10	84.30



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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.

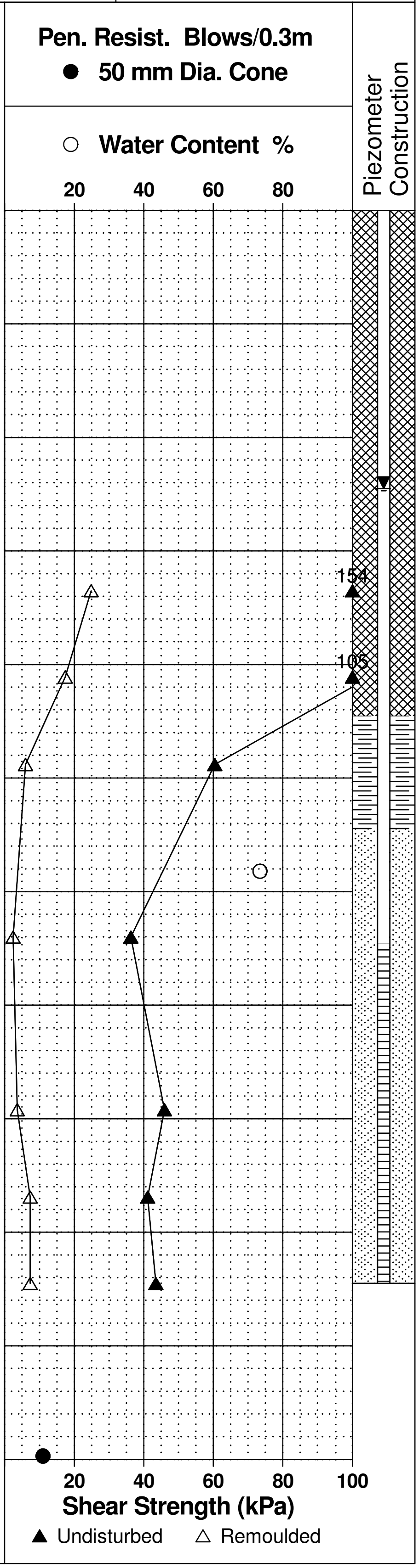
FILE NO. **PG1984**

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						0	94.05
FILL: Brown silty sand with gravel, trace cobbles		AU	1				
		SS	2	100	79	1	93.05
1.98		SS	3	75	12		
						2	92.05
Very stiff to stiff, brown **SILTY CLAY,** some sand		SS	4	100	6		
						3	91.05
						4	90.05
5.00						5	89.05
Firm, grey **SILTY CLAY**		TW	5	96			
						6	88.05
		TW	6	60			
						7	87.05
						8	86.05
						9	85.05
9.45							
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 10.8m depth.							
						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

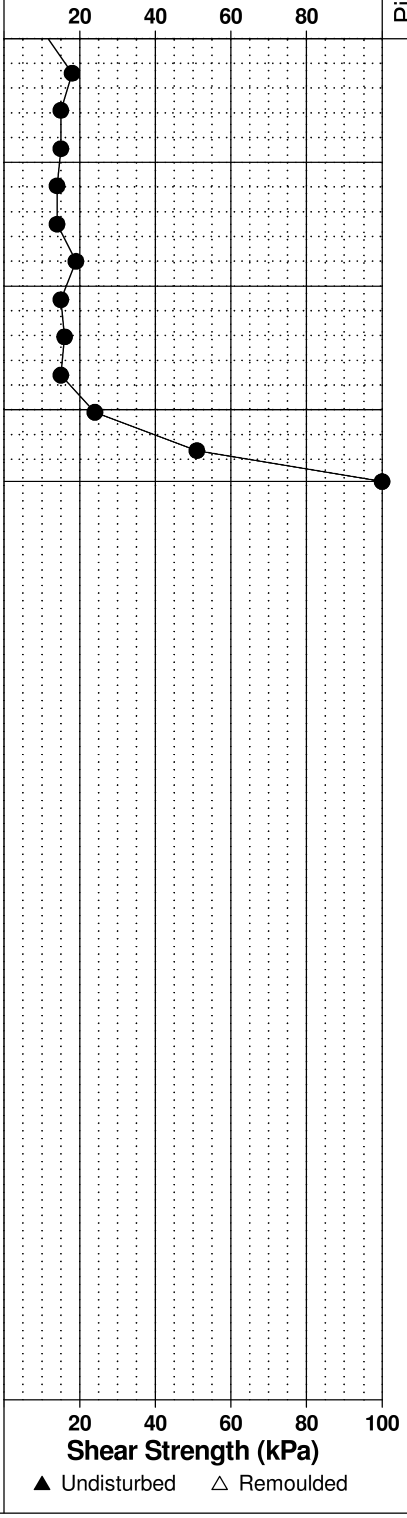
DATE March 9, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						11	83.05
Inferred firm to stiff cohesive soil over compact to dense glacial till						12	82.05
						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



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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**

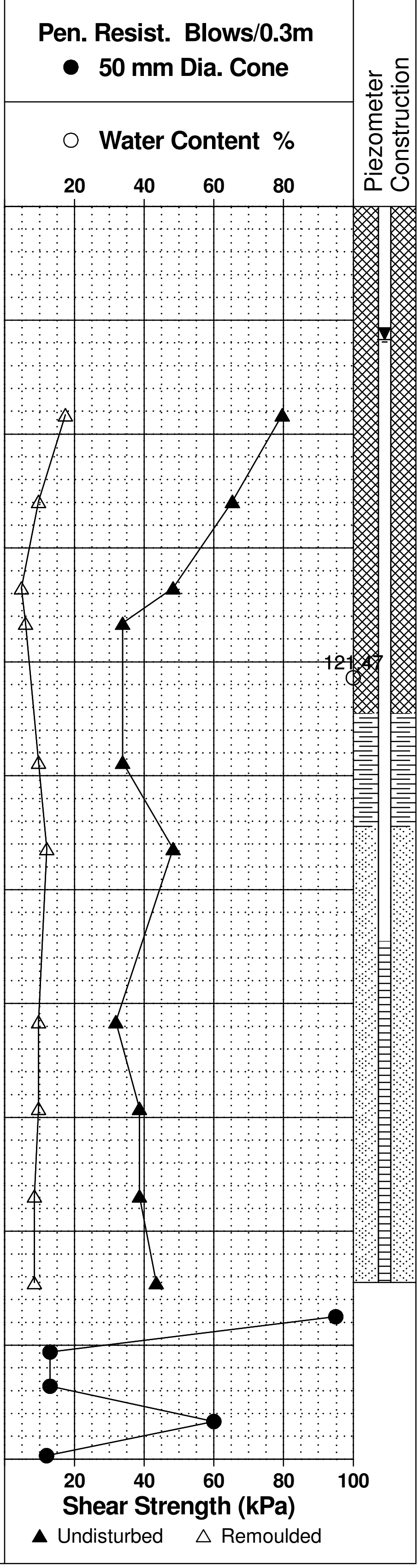
REMARKS

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)
GROUND SURFACE						0	92.71
Very stiff to stiff, brown **SILTY CLAY,** some sand	▨	AU	1				
	▨	SS	2	100	7	1	91.71
	▨					2	90.71
3.00							
Firm, grey **SILTY CLAY**	▨					3	89.71
	▨	TW	3	93		4	88.71
	▨					5	87.71
	▨					6	86.71
	▨	TW	4	98			
	▨					7	85.71
	▨					8	84.71
	▨					9	83.71
9.45							
Dynamic Cone Penetration Test commenced at 9.45m depth.						10	82.71
						11	81.71



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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

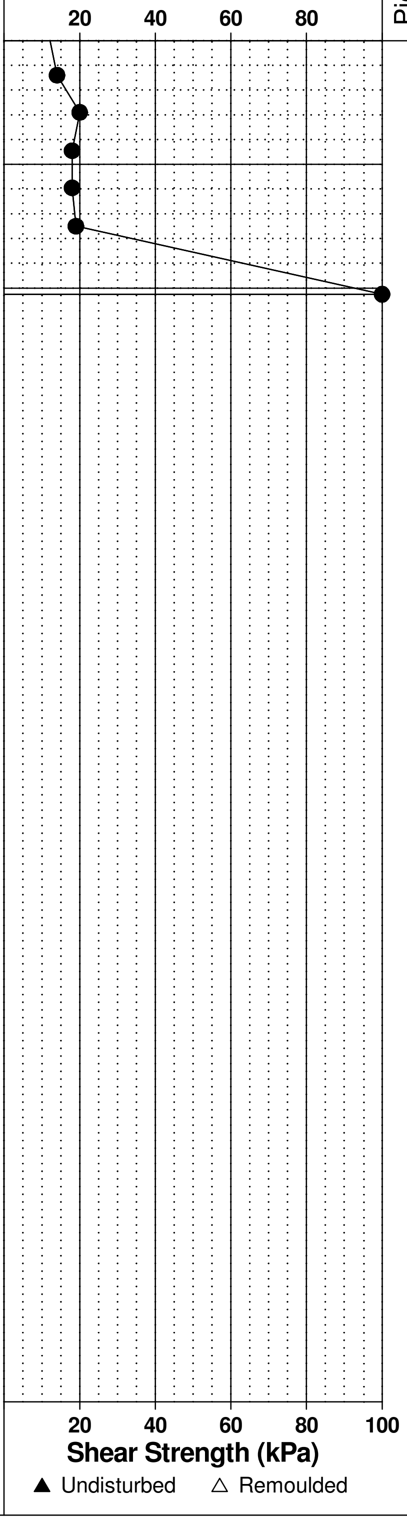
DATE March 9, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
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)							

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

○ Water Content %

Piezometer Construction



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

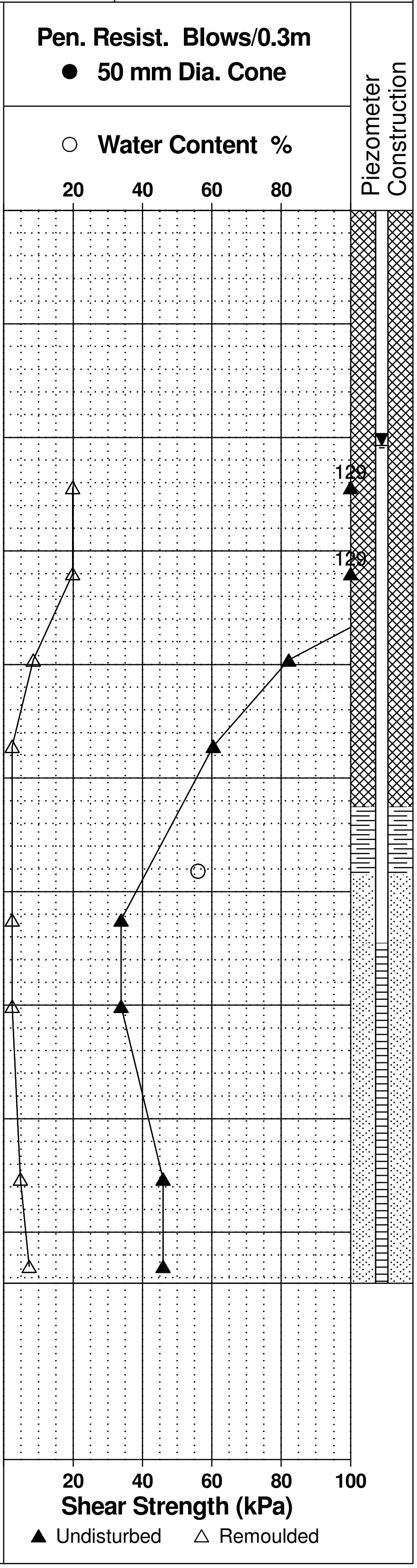
BORINGS BY CME 55 Power Auger

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 (0.60 to 1.83)	SS	2	0	9
	SS	3	25	4
Very stiff, brown **SILTY CLAY,** trace sand (1.83 to 3.80)	G	4		
Stiff to firm, grey **SILTY CLAY** (3.80 to 9.45)	G	5		
	TW	6	100	
	G	7		
	TW	8	100	
	G	9		
End of Borehole (9.45)				

(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**

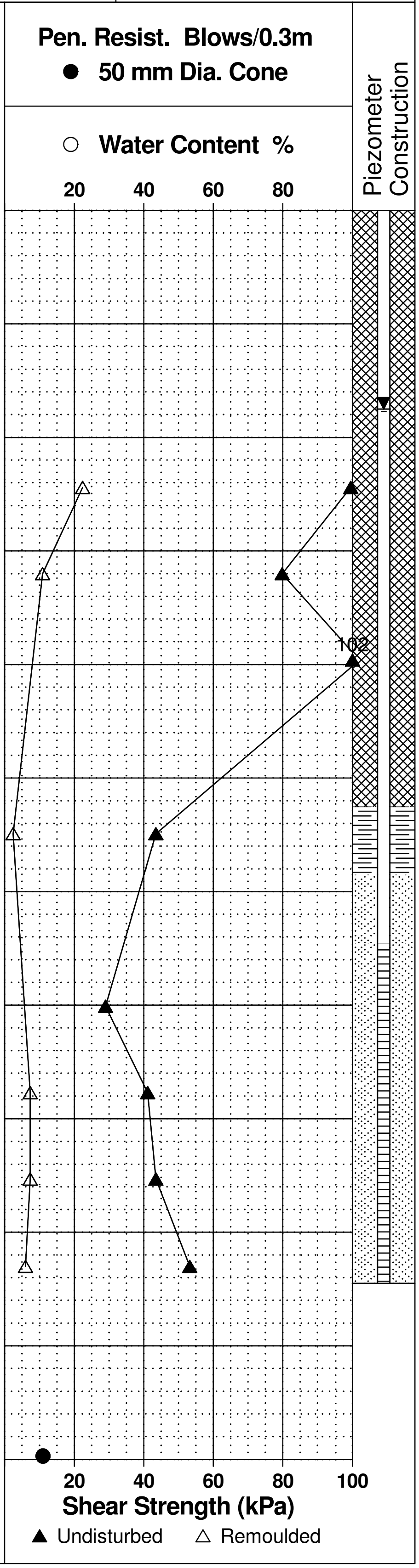
BORINGS BY CME 55 Power Auger

DATE April 21, 2016

SOIL DESCRIPTION	DEPTH (m)
GROUND SURFACE	0
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



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

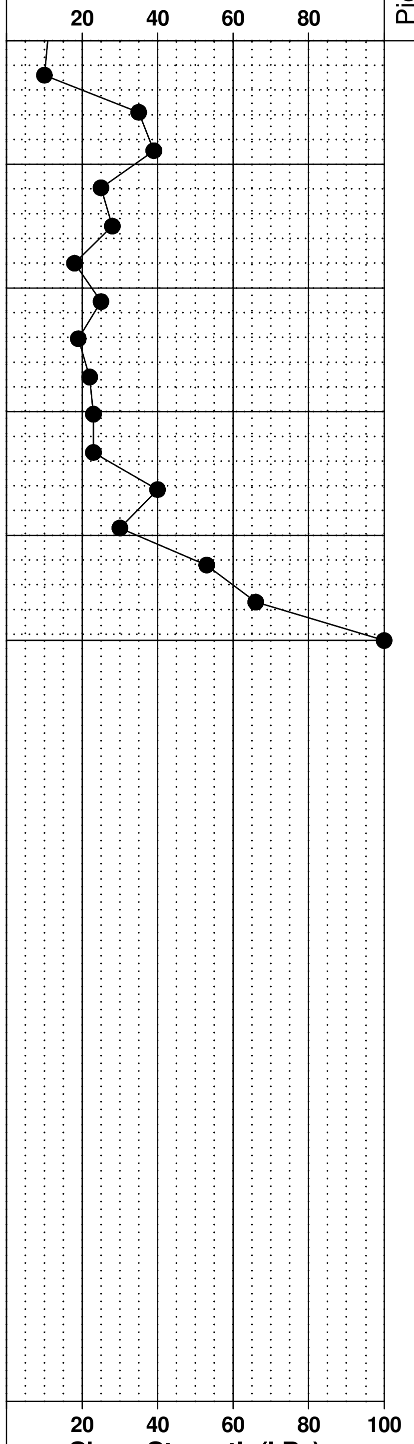
BORINGS BY CME 55 Power Auger

DATE April 21, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						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)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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. **BH14A-16**

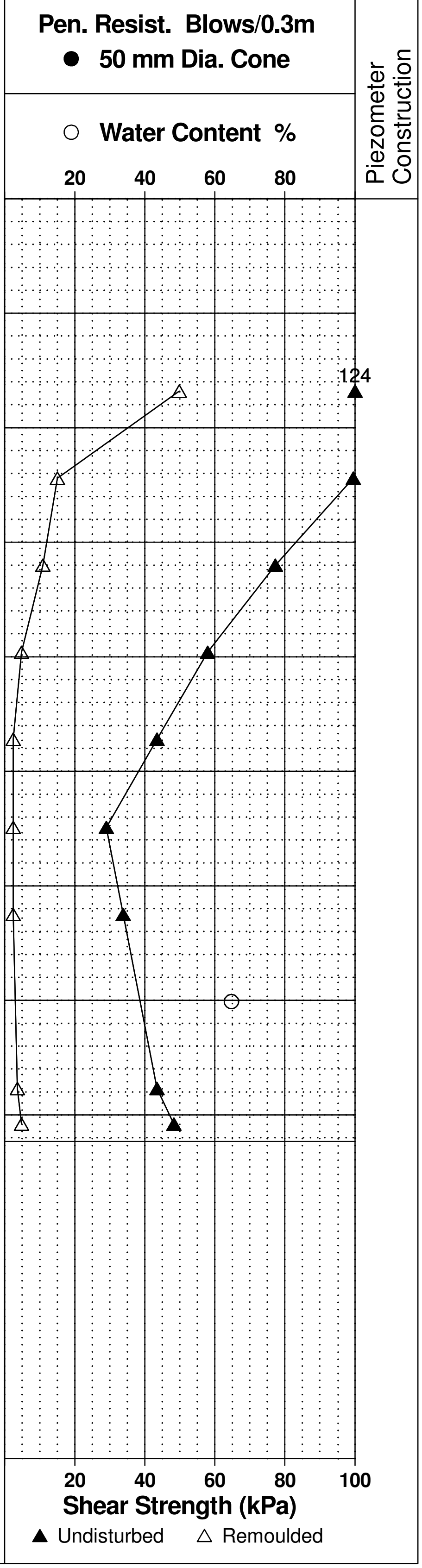
BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: Brown silty clay, some gravel and cobbles	1.68
Very stiff to stiff, brown **SILTY CLAY,** some sand seams	3.80
Stiff to firm, grey **SILTY CLAY**	8.23
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	25	6
TW	3	100	

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



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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**

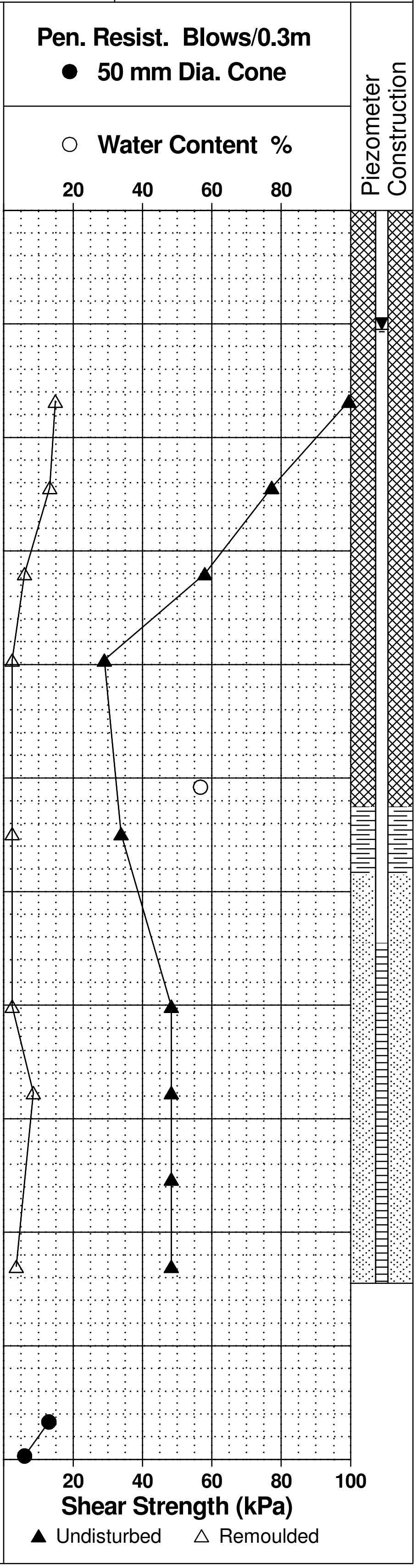
BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
Very stiff to stiff, brown **SILTY CLAY,** some sand	3.40
Stiff to firm, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 10.4m depth.	

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

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



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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

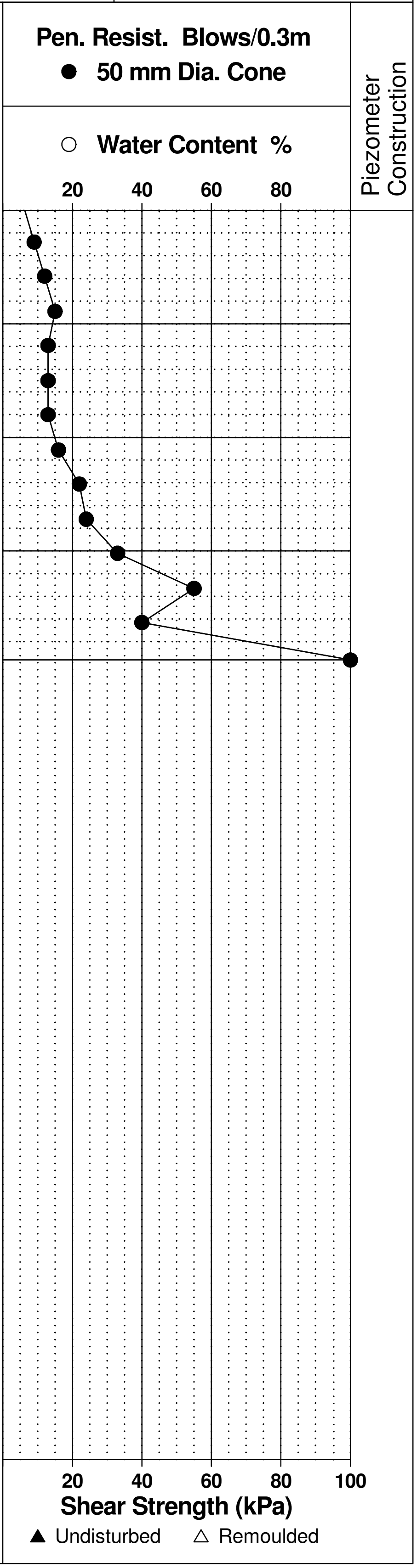
REMARKS

HOLE NO. BH23-16

BORINGS BY CME 55 Power Auger

DATE April 28, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						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.

REMARKS

BORINGS BY CME 75 Power Auger

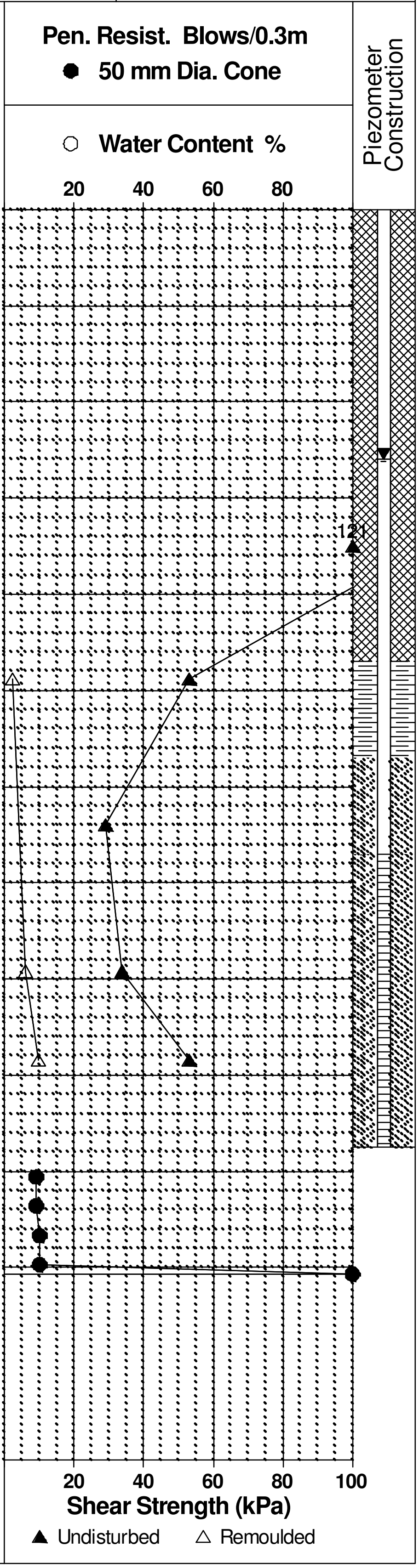
DATE 4 February 2011

FILE NO. PG1984

HOLE NO. BH 7

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel, trace clay	1.45	SS	1	100	70
		SS	2	100	11
Very stiff to stiff, brown **SILTY CLAY**	4.80	SS	3		7
		SS	4		4
		SS	5		W
Firm to stiff, grey **SILTY CLAY**	9.75	TW	6		
		TW	7		
		TW	8		
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

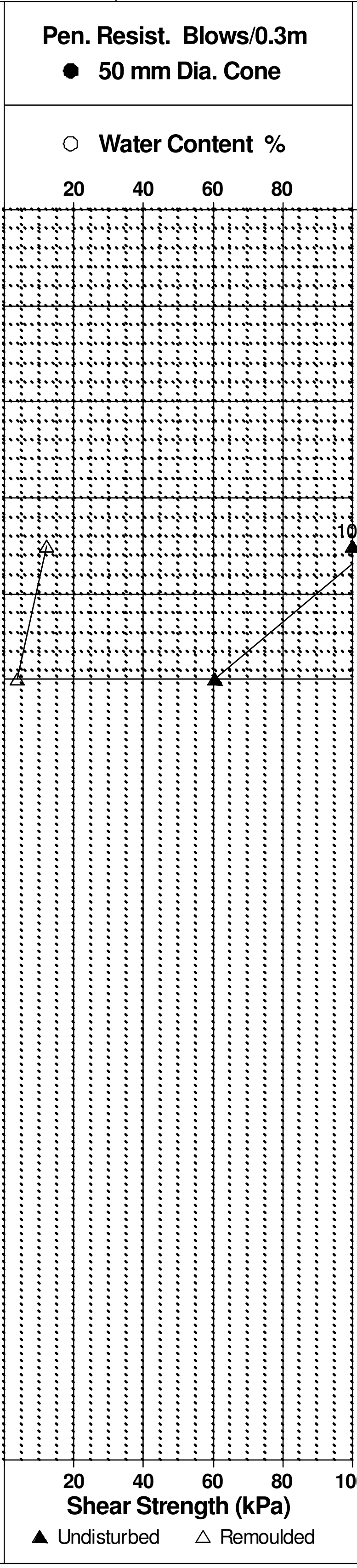
BORINGS BY CME 75 Power Auger

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)
GROUND SURFACE						0	
FILL: Brown silty sand with gravel, trace clay 1.45	▨					1	
Very stiff to stiff, brown **SILTY CLAY** 4.88	▨	TW	6			2 3 4	
End of Borehole							



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.

FILE NO. **PG1984**

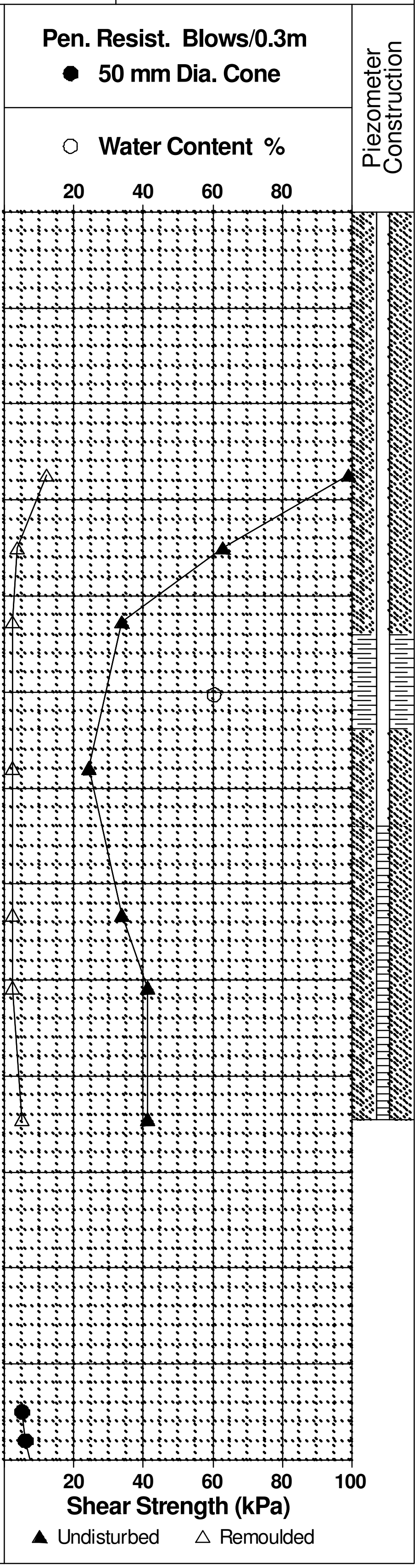
REMARKS

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	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	92.29
Loose to compact, brown **SILTY SAND** 0.60	SS	1	92	30		
Very stiff to stiff, brown **SILTY CLAY**	SS	2	92	7	1	91.29
	SS	3		3	2	90.29
3.60					3	89.29
Firm, grey **SILTY CLAY**					4	88.29
	TW	4			5	87.29
	TW	5			6	86.29
					7	85.29
	TW	6			8	84.29
9.45					9	83.29
Dynamic Cone Penetration Test commenced @ 9.45m depth. Cone pushed to 12.2m depth					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.

REMARKS

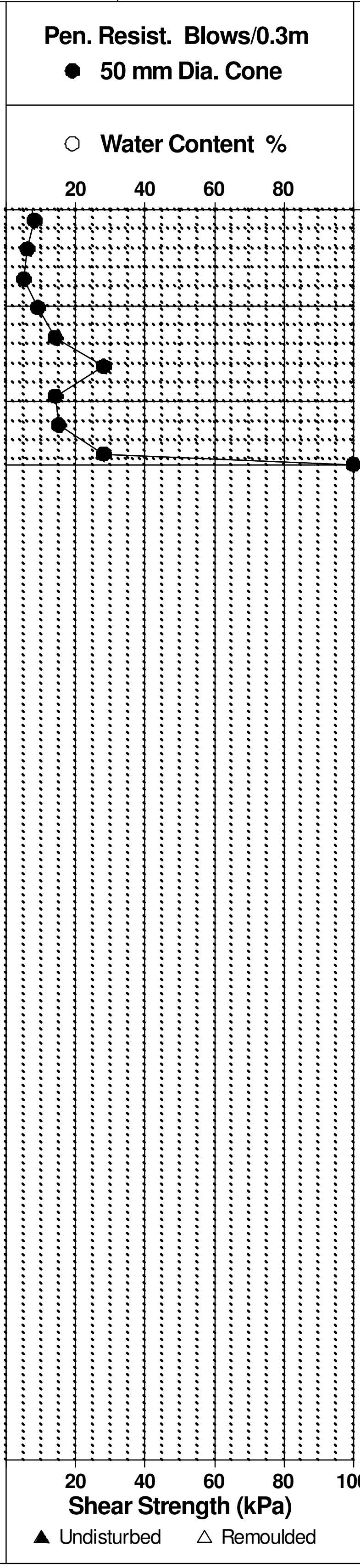
BORINGS BY CME 75 Power Auger

DATE 4 February 2011

FILE NO. PG1984

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
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)								



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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

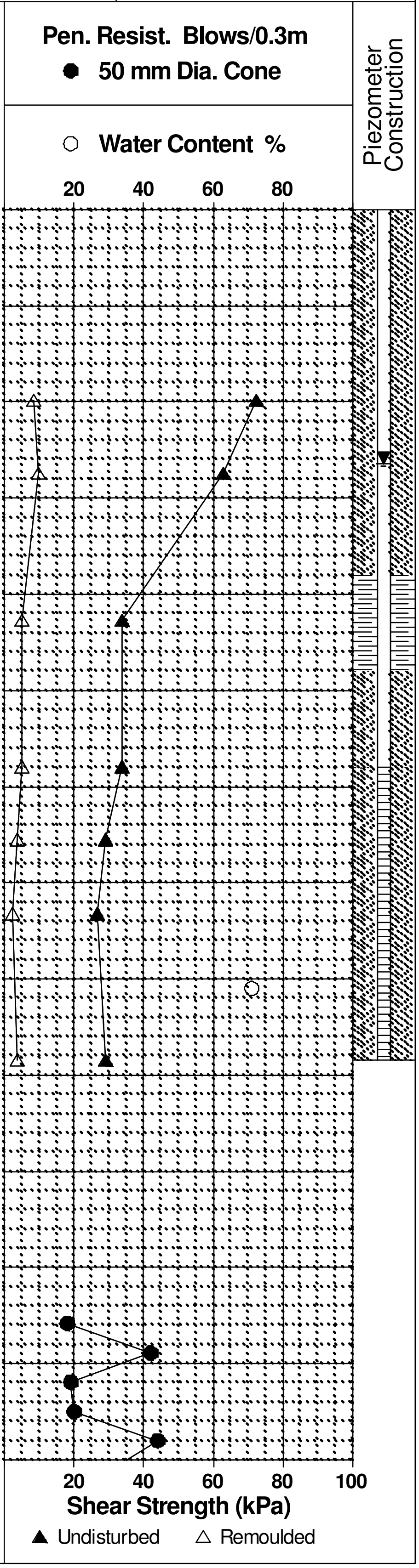
DATE 3 February 2011

FILE NO. PG1984

HOLE NO. BH 9

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Compact to loose, brown **SILTY SAND,** trace clay	SS	1		16
	SS	2		6
1.45				
Very stiff to stiff, brown **SILTY CLAY**				
2.90				
Frim, grey **SILTY CLAY**	TW	3		
	TW	4		
	TW	5		
8.84				
Dynamic Cone Penetration Test commenced @ 8.84m depth. Cone pushed to 11.3m depth.				

DEPTH (m)	ELEV. (m)
0	92.06
1	91.06
2	90.06
3	89.06
4	88.06
5	87.06
6	86.06
7	85.06
8	84.06
9	83.06
10	82.06
11	81.06
12	80.06
13	79.06



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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

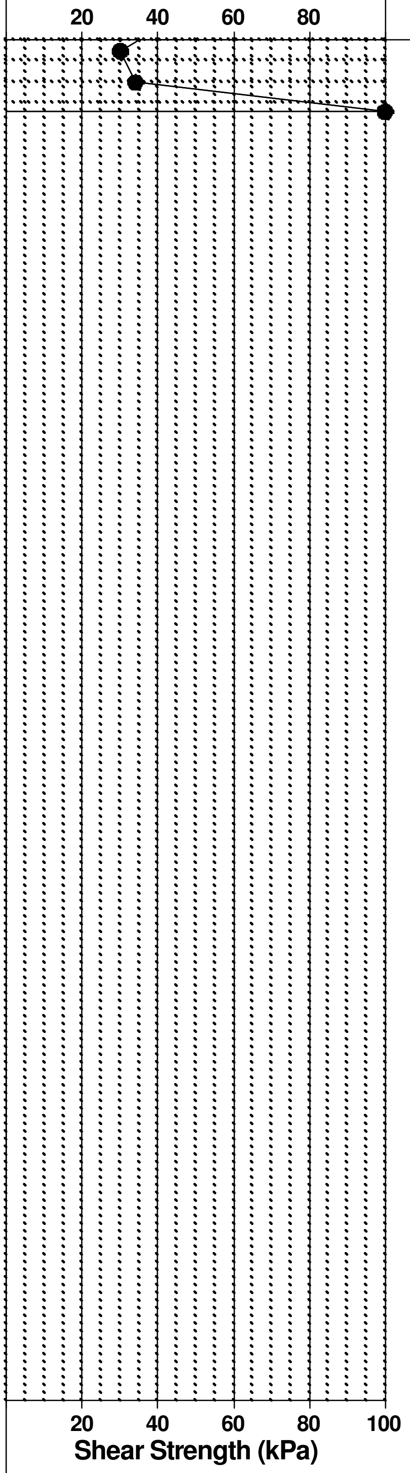
BORINGS BY CME 75 Power Auger

DATE 3 February 2011

FILE NO. PG1984

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						13	79.06		
13.69									
End of Borehole Practical DCPT refusal @ 13.69m depth. (GWL @ 2.64m-Feb. 10/11)									



▲ 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

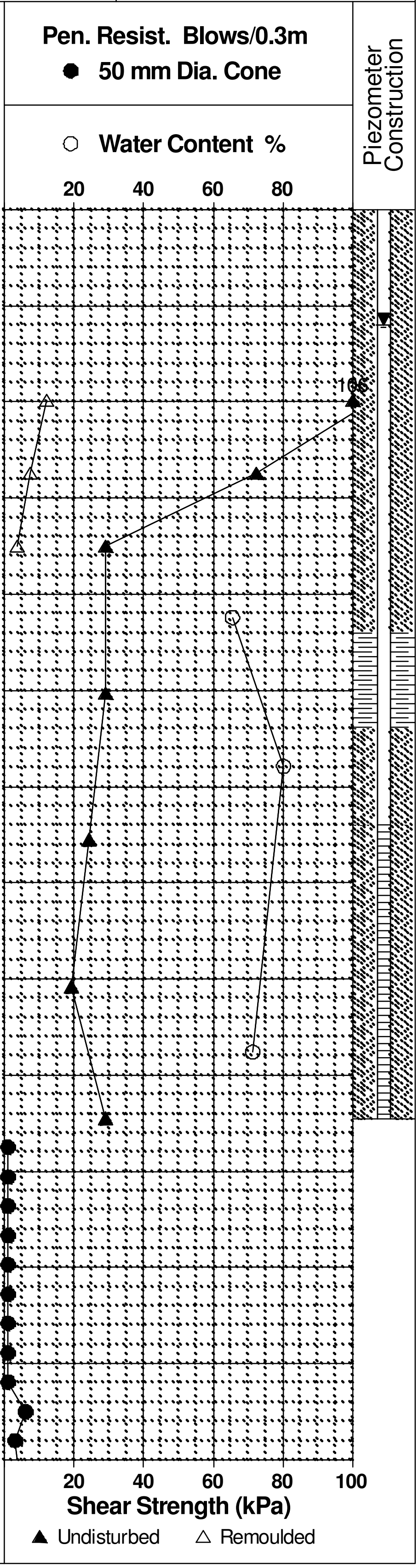
BORINGS BY CME 75 Power Auger

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	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.09
Compact, brown **SILTY SAND**		SS	1	92	16		
0.60							
		SS	2		2	1	91.09
Very stiff to stiff, brown **SILTY CLAY**						2	90.09
2.90							
						3	89.09
		TW	3			4	88.09
						5	87.09
		TW	4				
Firm to soft, grey **SILTY CLAY**						6	86.09
						7	85.09
						8	84.09
		TW	5				
						9	83.09
9.45							
Dynamic Cone Penetration Test commenced @ 9.45m depth.						10	82.09
						11	81.09
						12	80.09
						13	79.09



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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. BH10

BORINGS BY CME 75 Power Auger

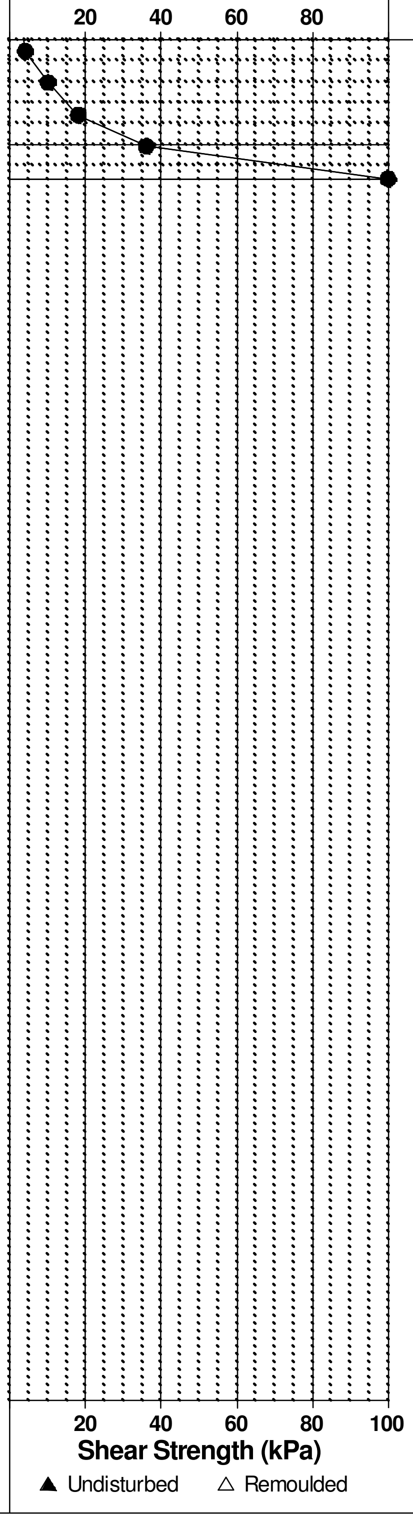
DATE 28 January 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
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)							

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

○ **Water Content %**

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)

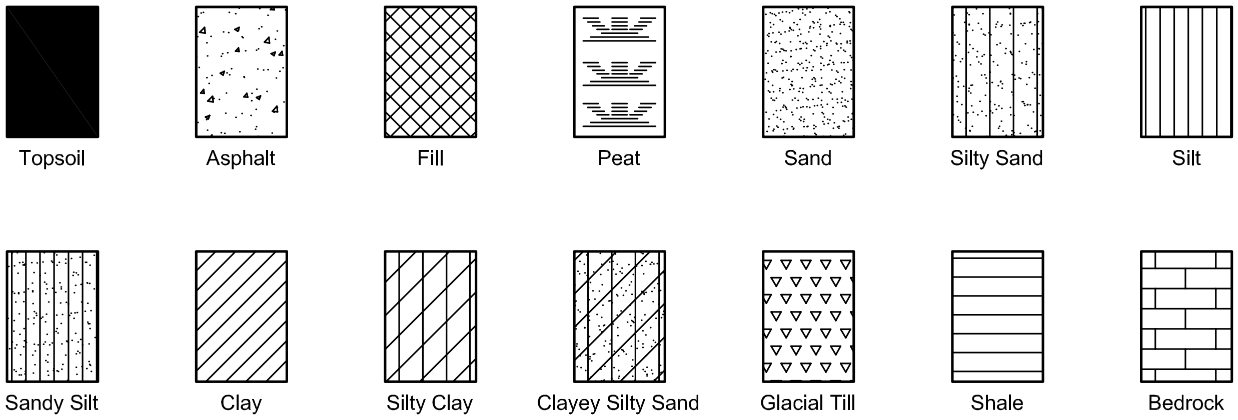
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

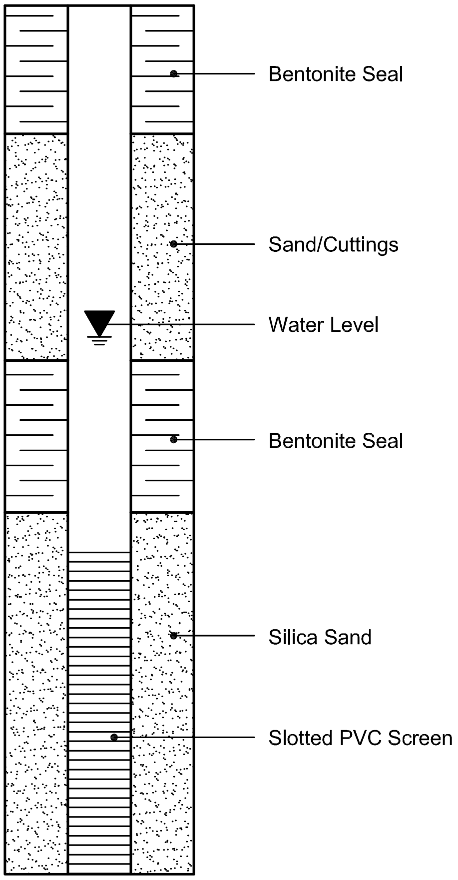
SYMBOLS AND TERMS (continued)

STRATA PLOT

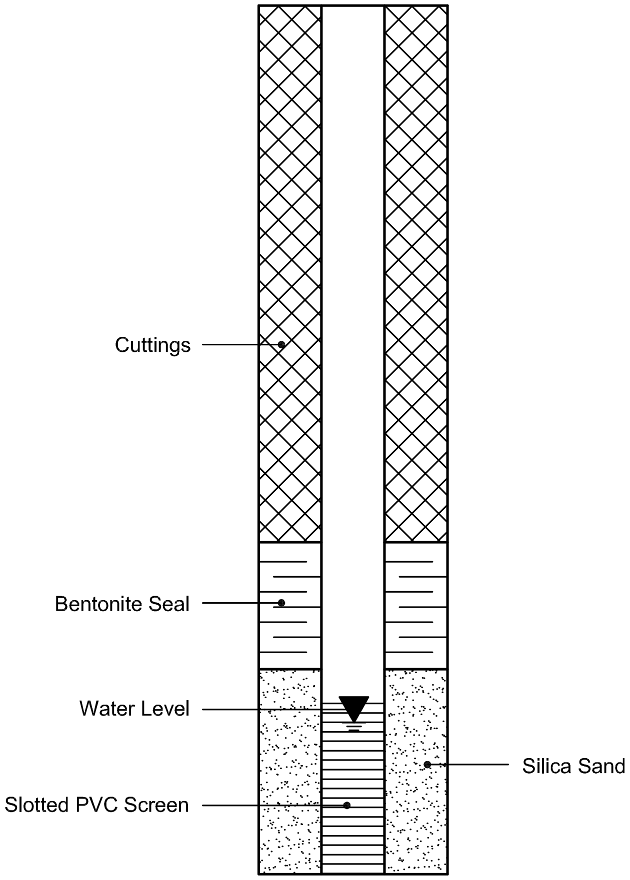


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

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:

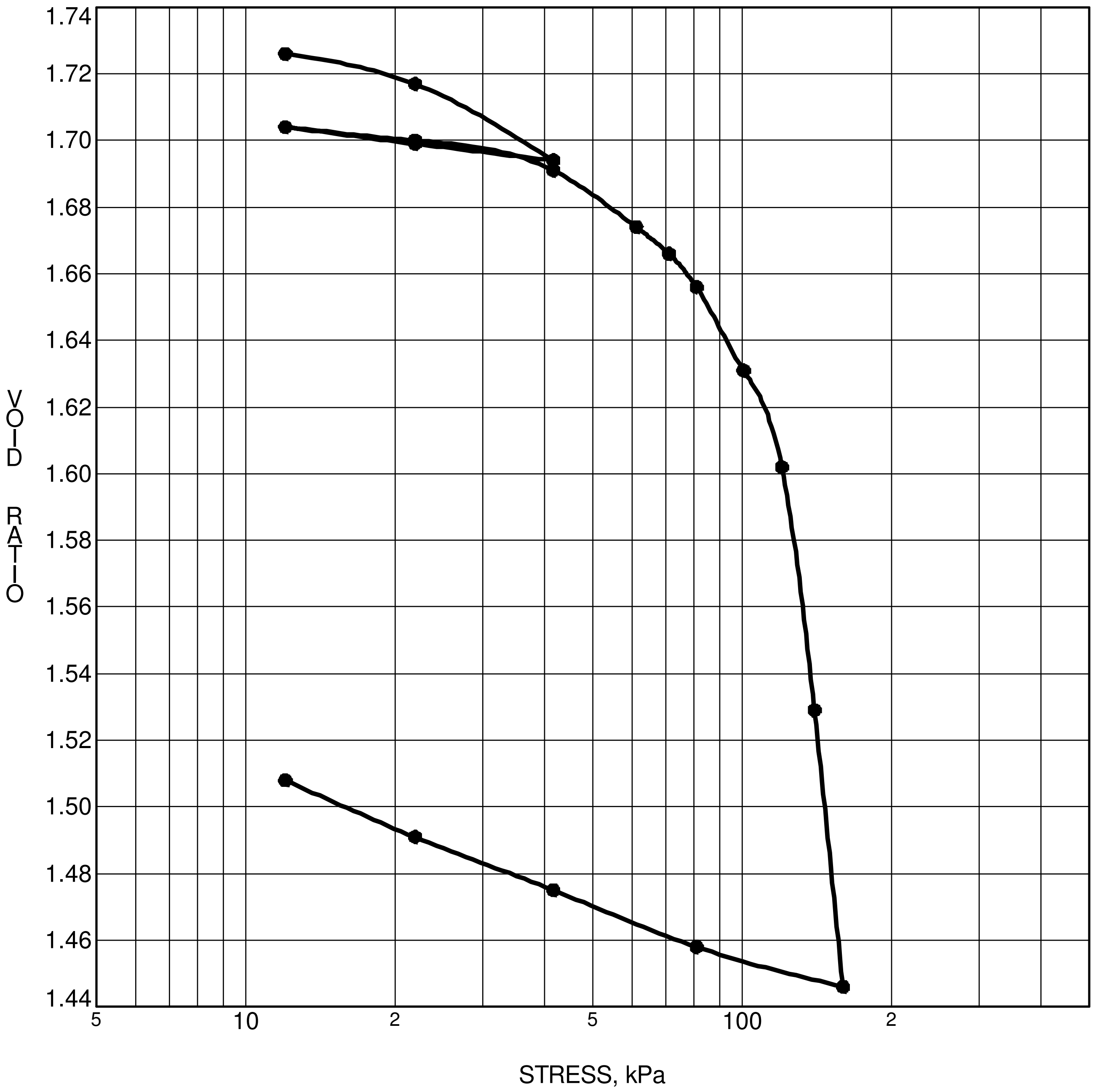
1. Effective overburden pressure, p'_o, is based on an average crust thickness of 3.0 m, an estimated long-term low groundwater depth of 2.5 m and a mean unit weight for the grey clay of 16.0 kN/m^3. The original ground elevation is used for p'_o estimates.
2. The last column presents the quality assessment 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**

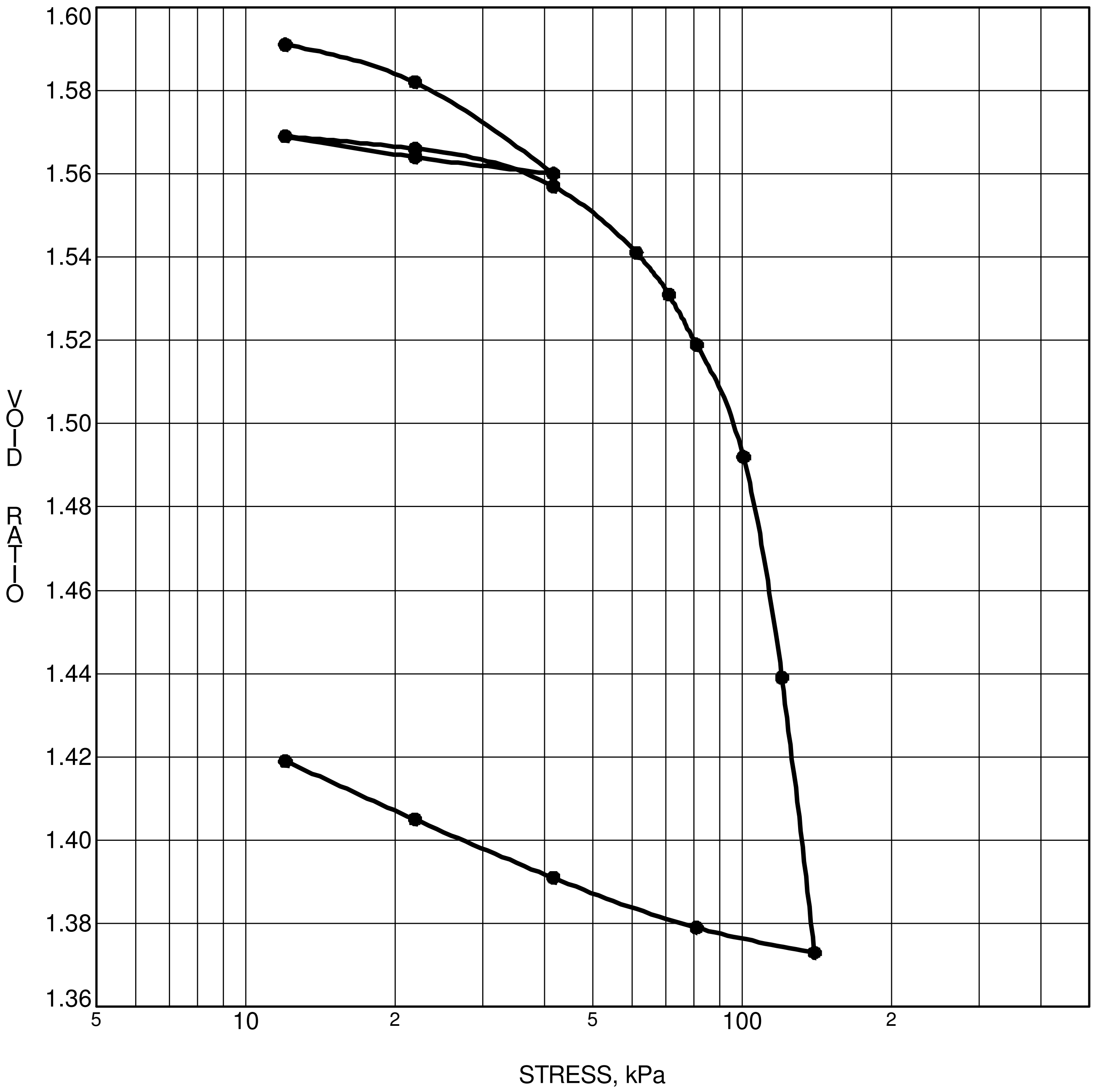
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**

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.**

FILE NO. **PG1984**

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

DATE **12/03/2015**

patersongroup **Consulting Engineers**

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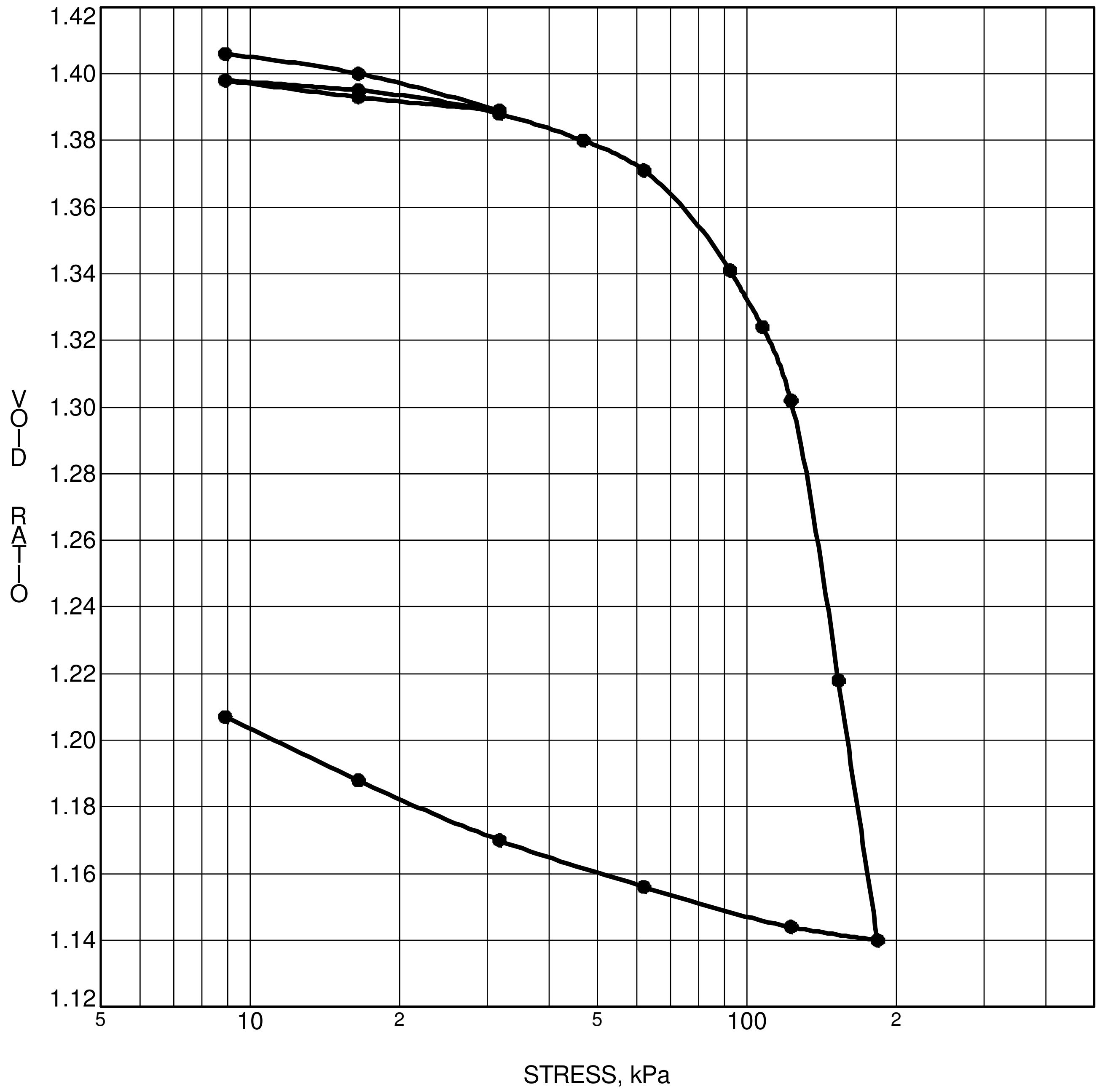

CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 4-15**	p'_o	**67** kPa	Ccr	**0.017**
Sample No.	**TW 4**	p'_c	**110** kPa	Cc	**0.973**
Sample Depth	**5.89** m	OC Ratio	**1.6**	Wo	**51.5** %
Sample Elev.	**85.95** m	Void Ratio	**1.416**	Unit Wt.	**17.0** kN/m ^{3}

CLIENT **Minto Communities Inc.**
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DATE **12/03/2015**

patersongroup **Consulting Engineers**
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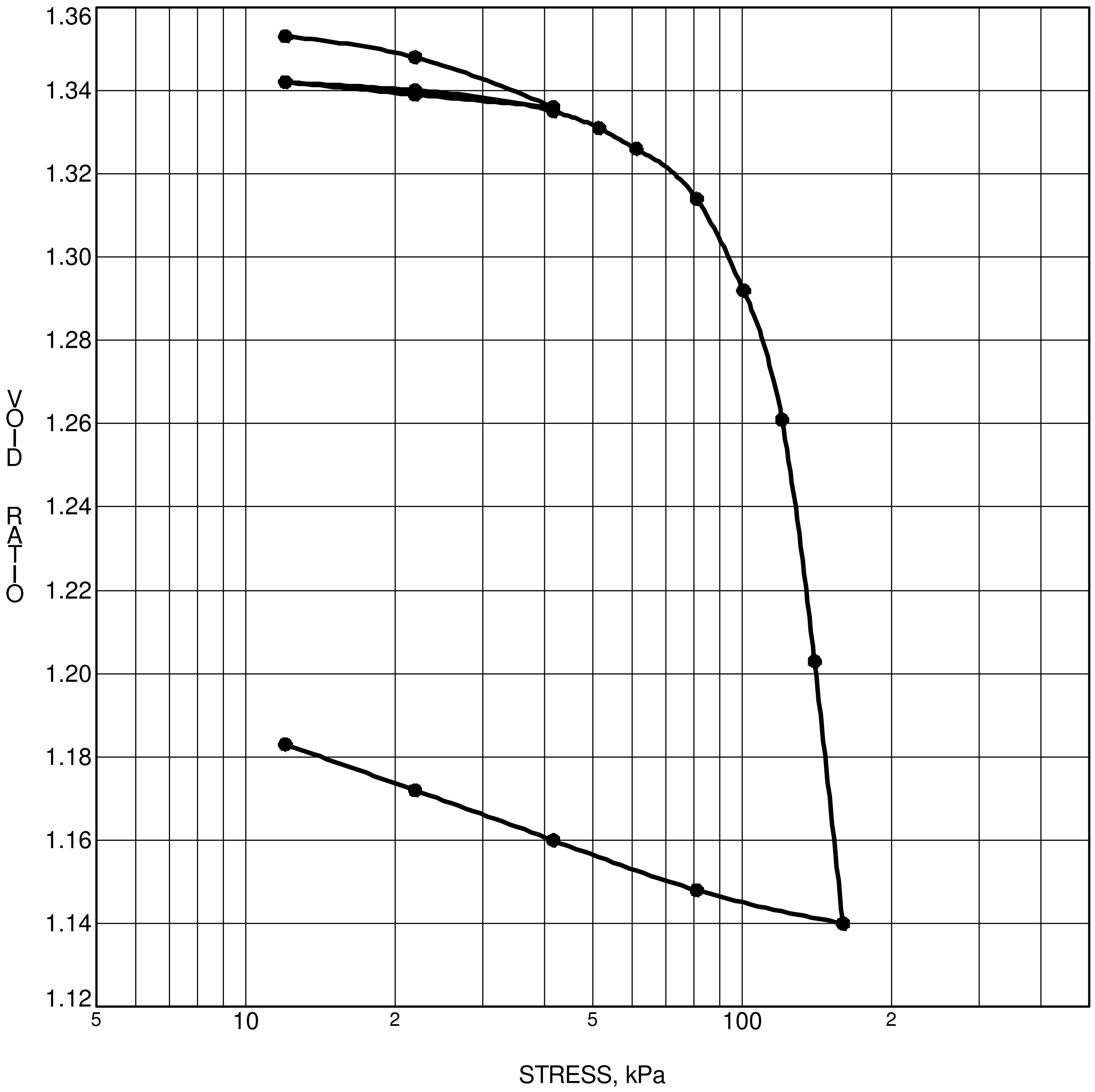
CONSOLIDATION TEST



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**

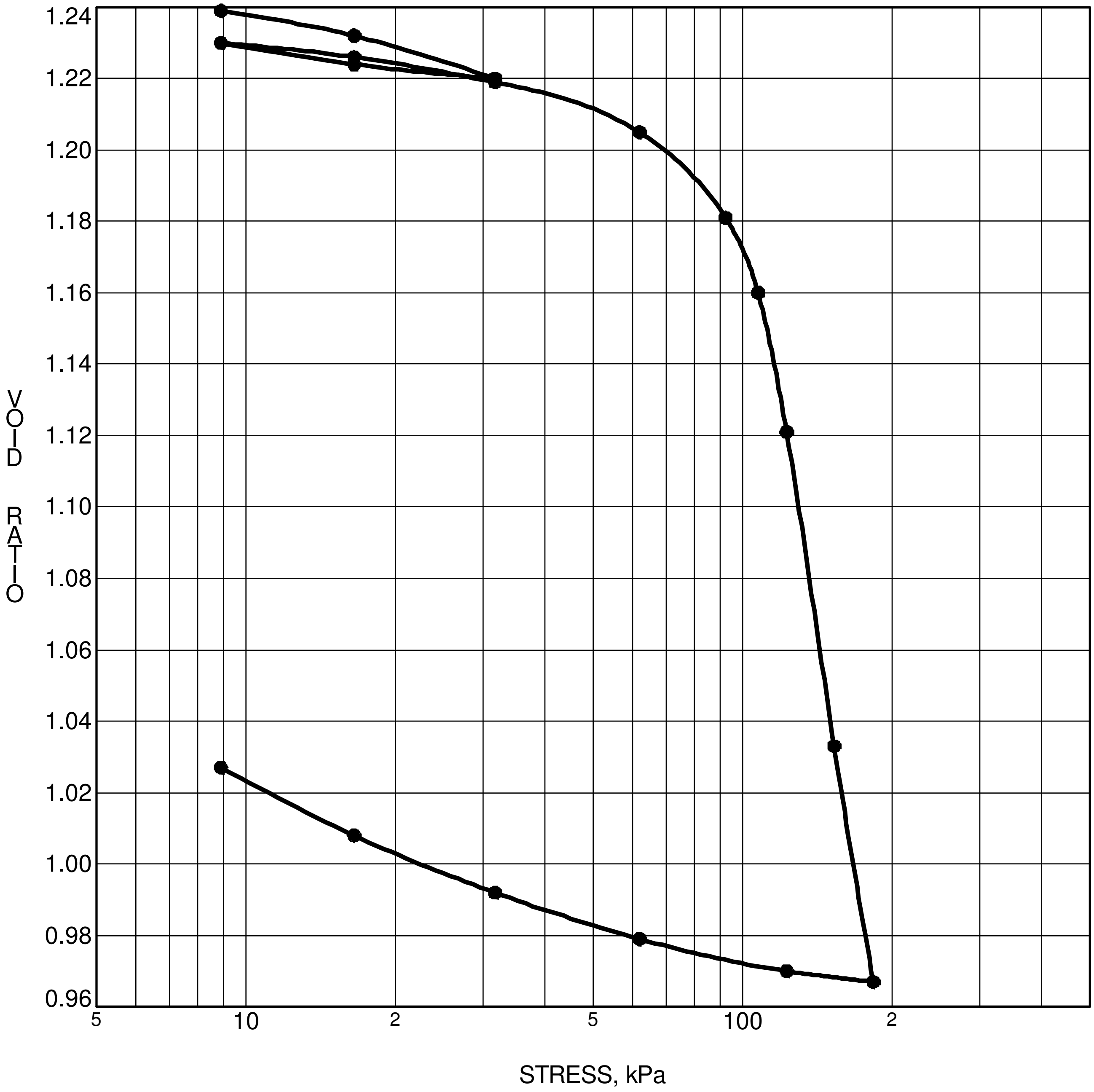
PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke & Mion Lands**

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**

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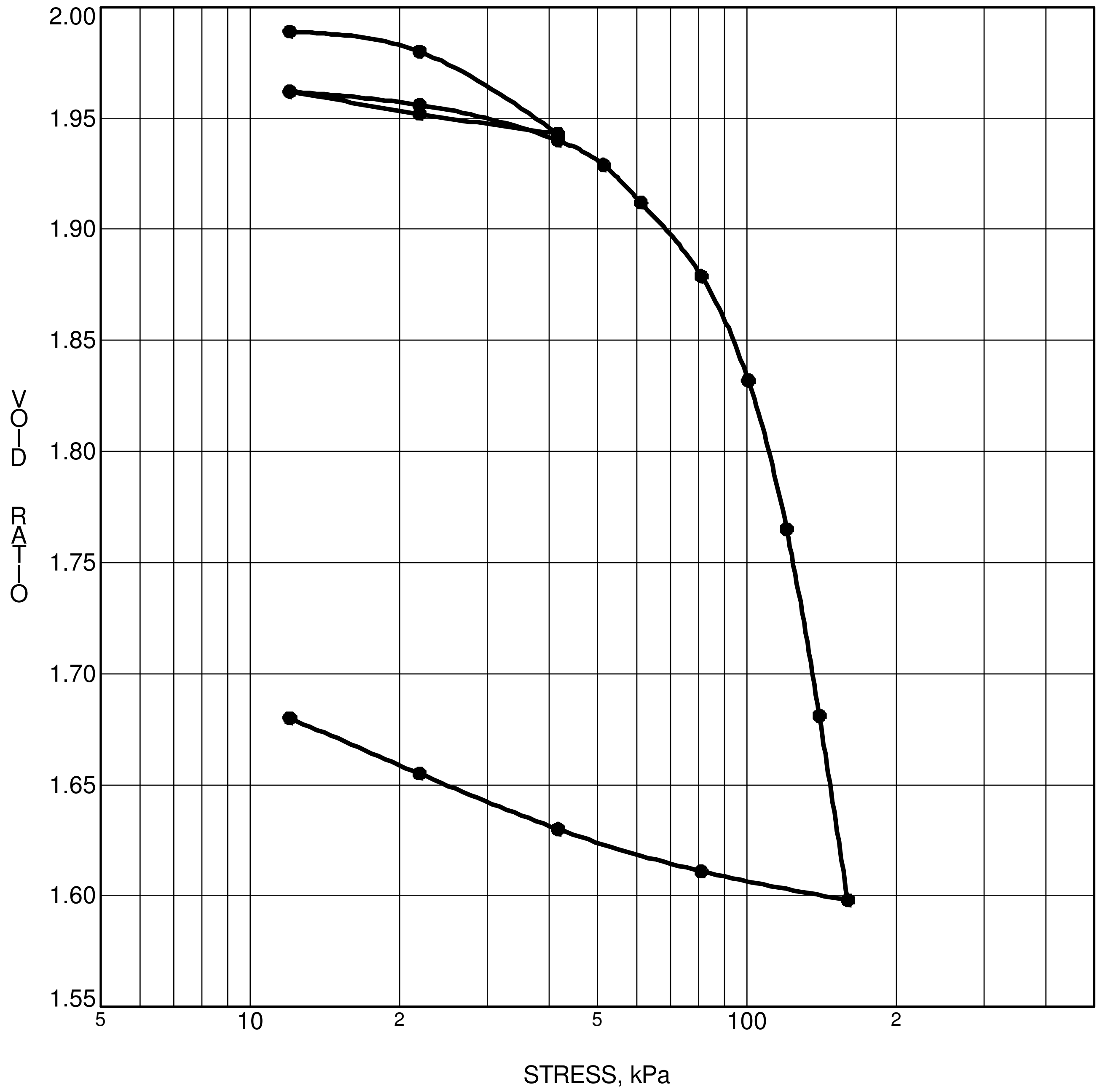
CONSOLIDATION TEST



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**

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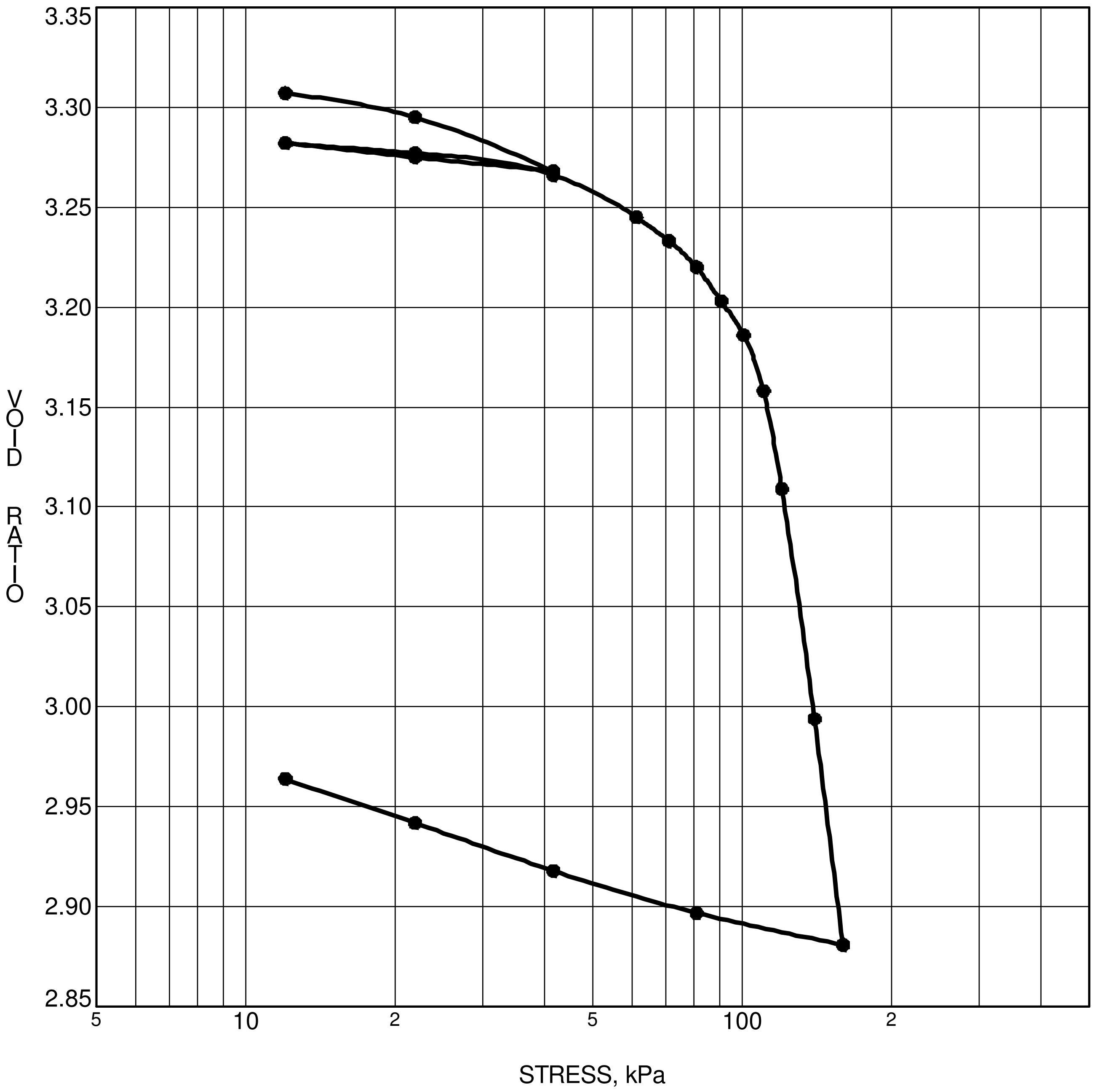
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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**

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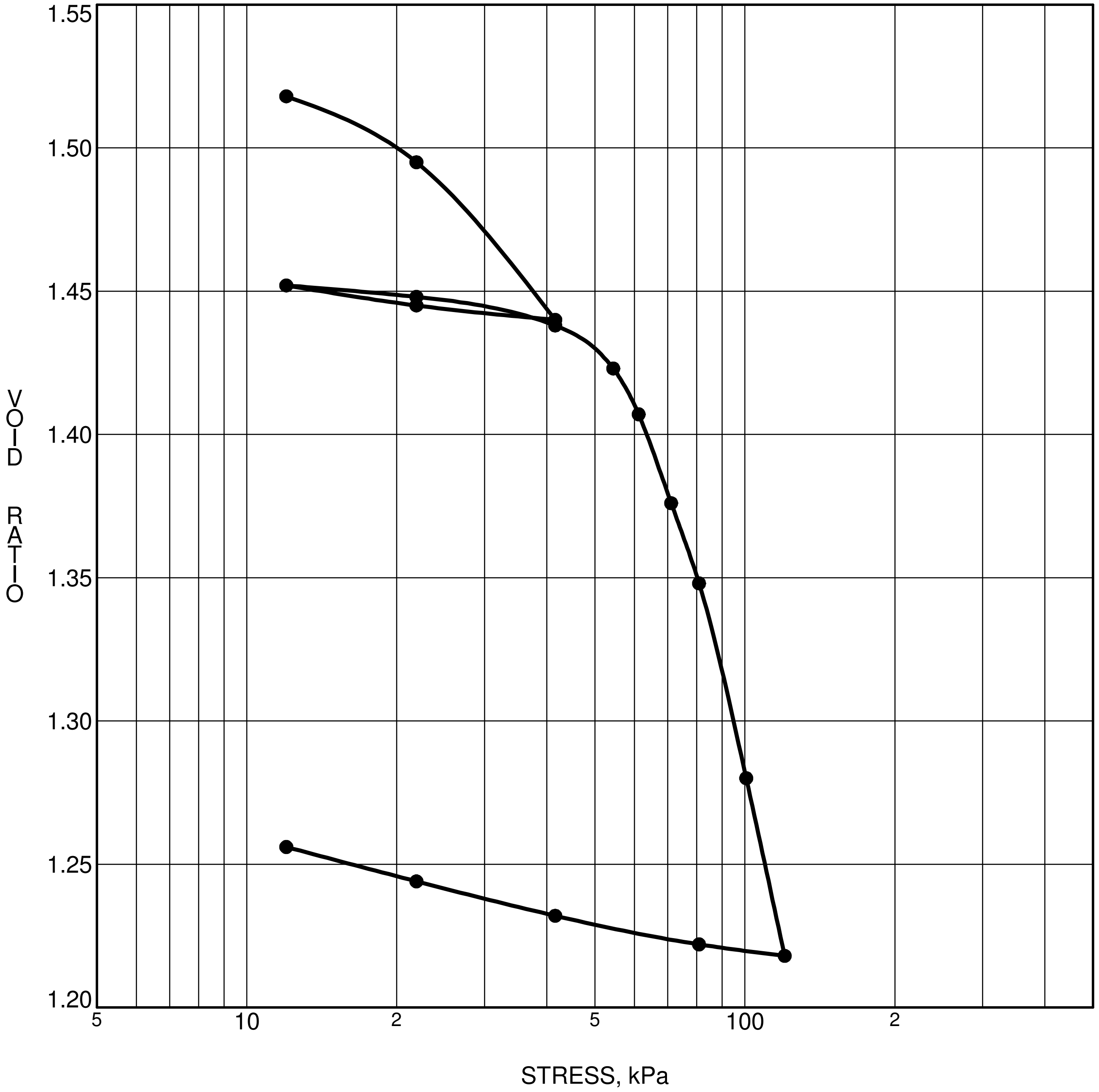
CONSOLIDATION TEST



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**

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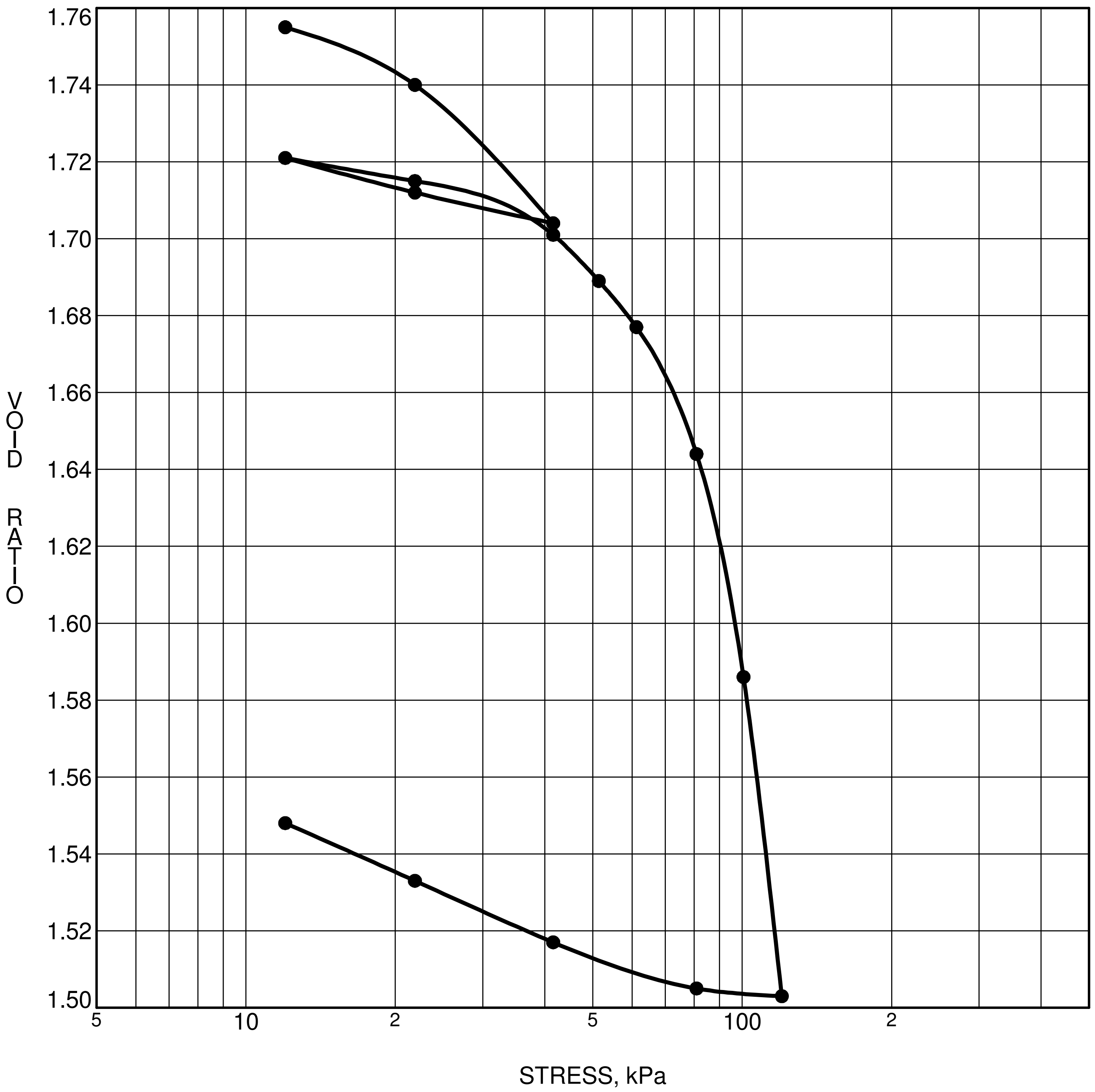
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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**

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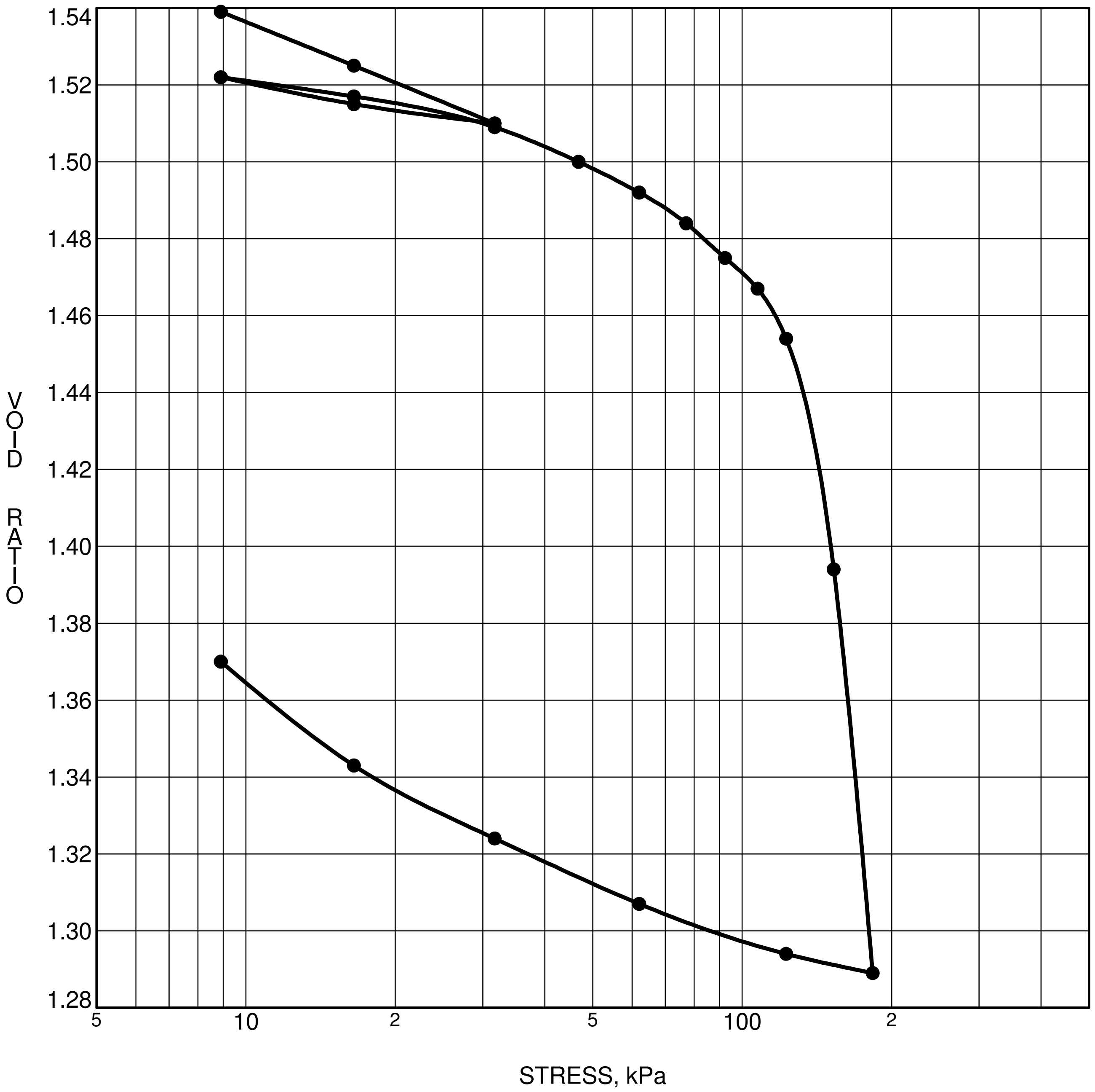

CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH23-16**	p'_o	**60** kPa	Ccr	**0.023**
Sample No.	**TW 4**	p'_c	**137** kPa	Cc	**1.366**
Sample Depth	**5.08** m	OC Ratio	**2.3**	Wo	**56.7** %
Sample Elev.	**87.21** m	Void Ratio	**1.56**	Unit Wt.	**16.5** kN/m ^{3}

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

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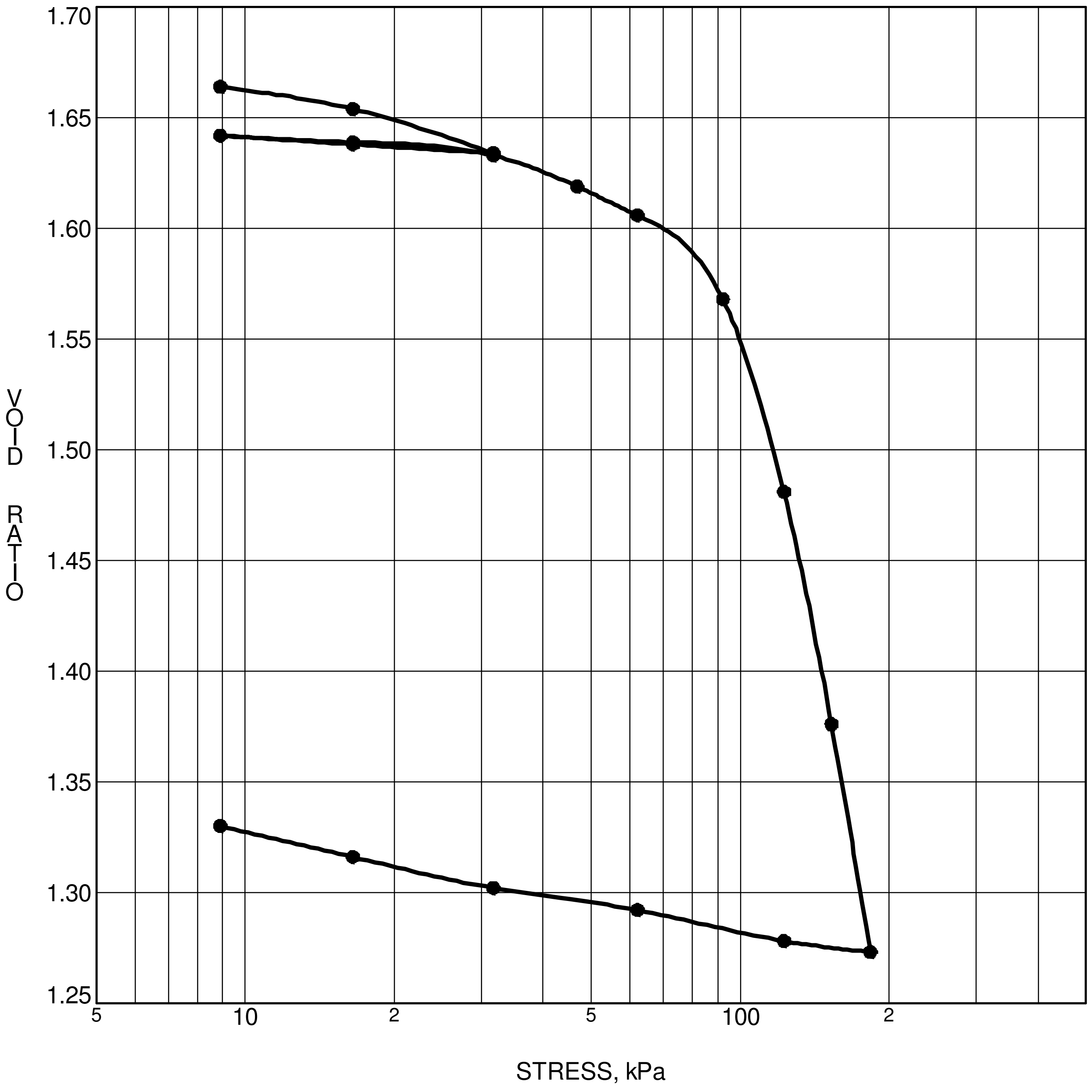


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 7**	p'_o	**65** kPa	Ccr	**0.016**
Sample No.	**TW 7**	p'_c	**100** kPa	Cc	**1.199**
Sample Depth	**7.31** m	OC Ratio	**1.5**	Wo	**61.2** %
Sample Elev.	**86.65** m	Void Ratio	**1.684**	Unit Wt.	**16.2** kN/m ^{3}

CLIENT **Minto Communities Inc.**

FILE NO. **PG1984**

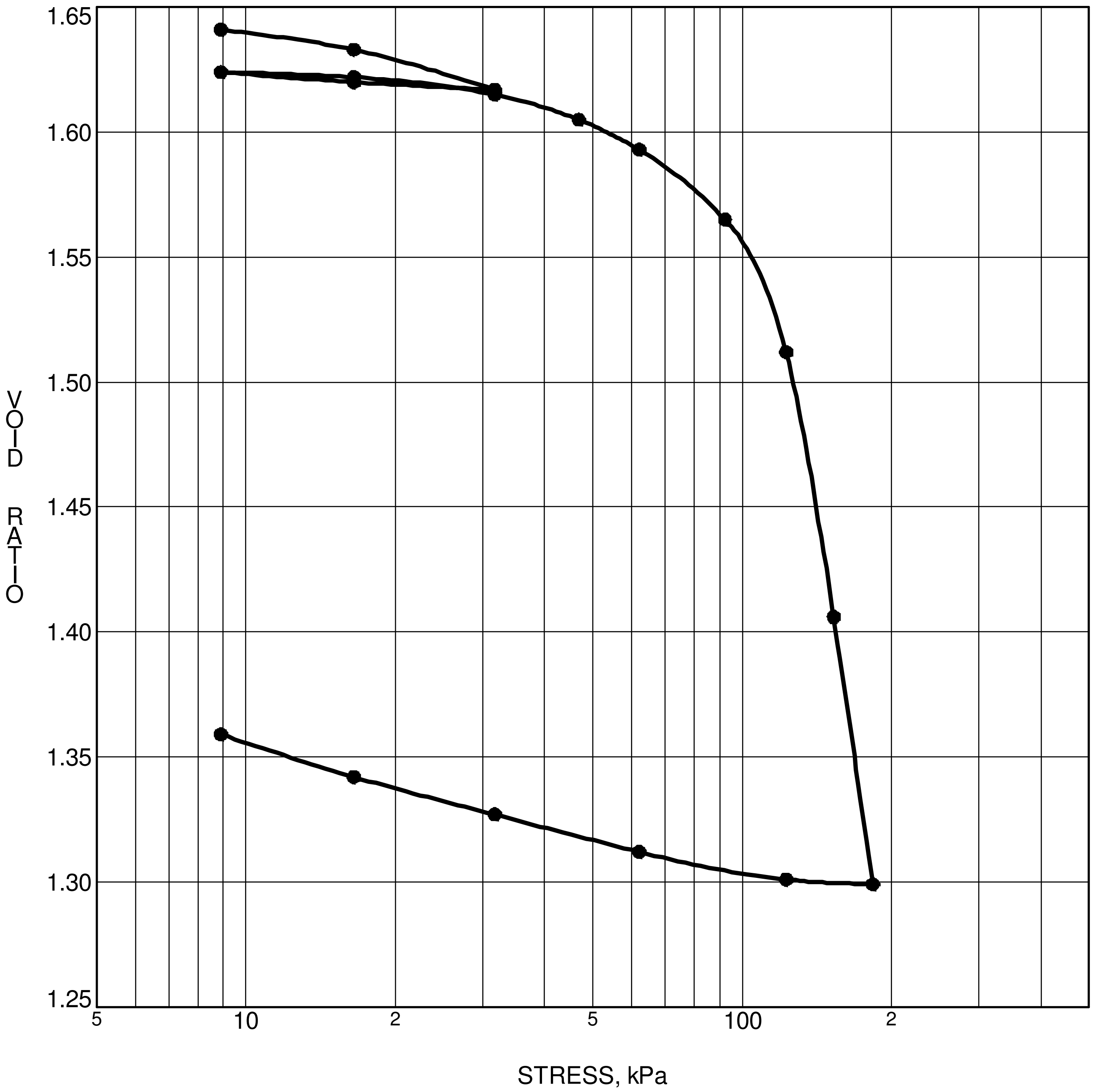
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

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 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**

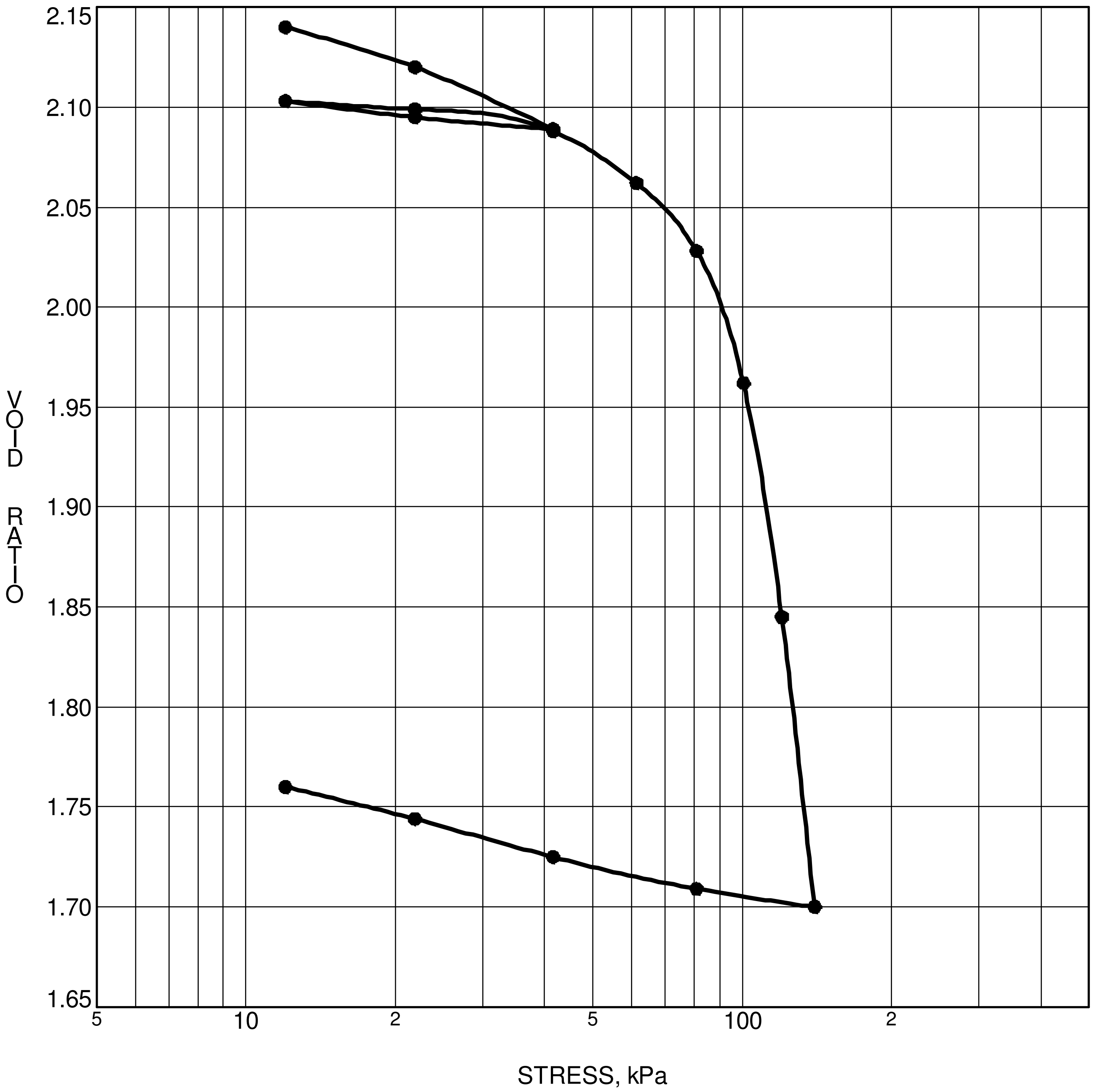
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

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**

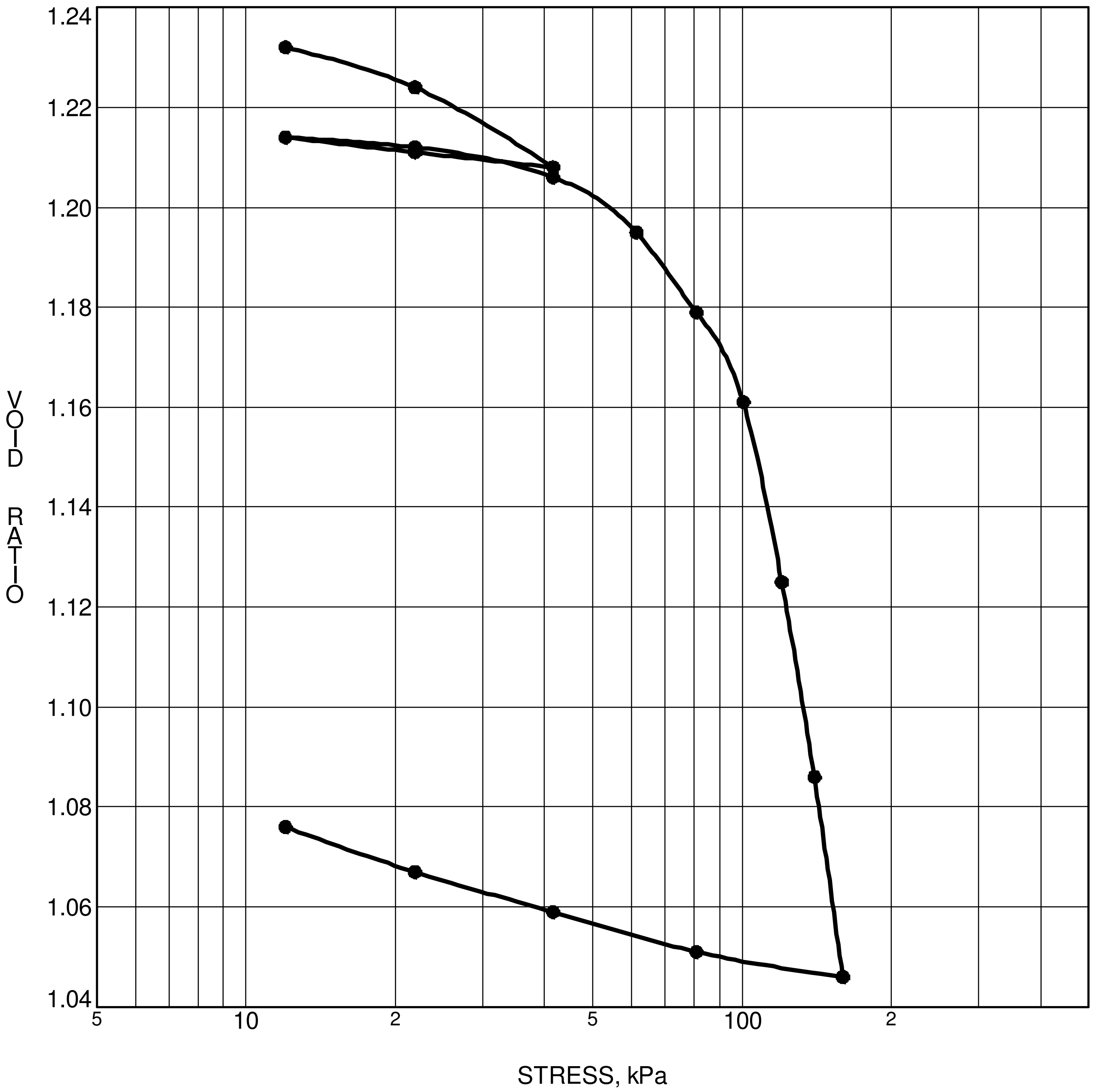
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

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**

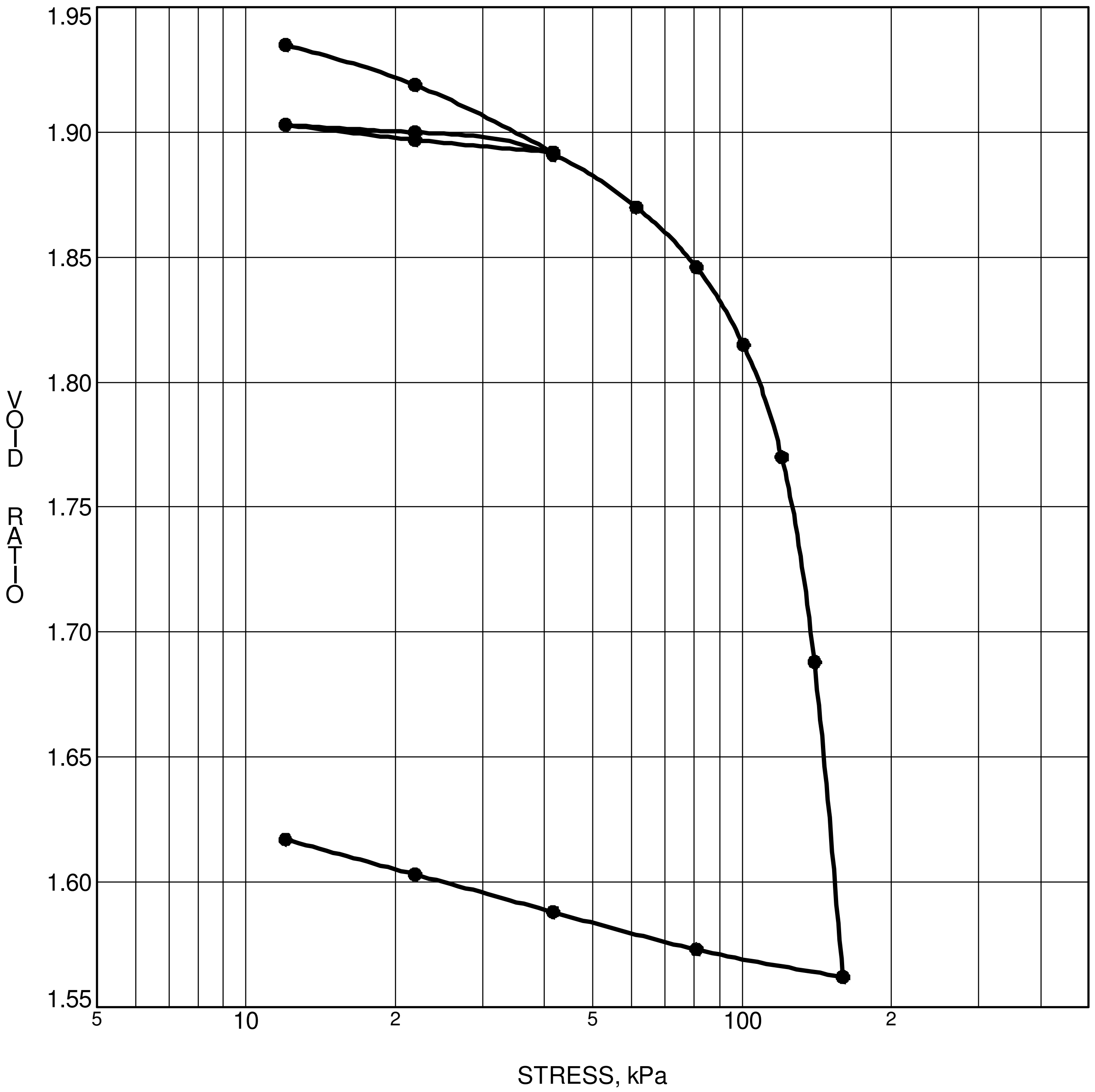
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

DATE **02/25/2011**

patersongroup **Consulting Engineers**

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

CONSOLIDATION TEST



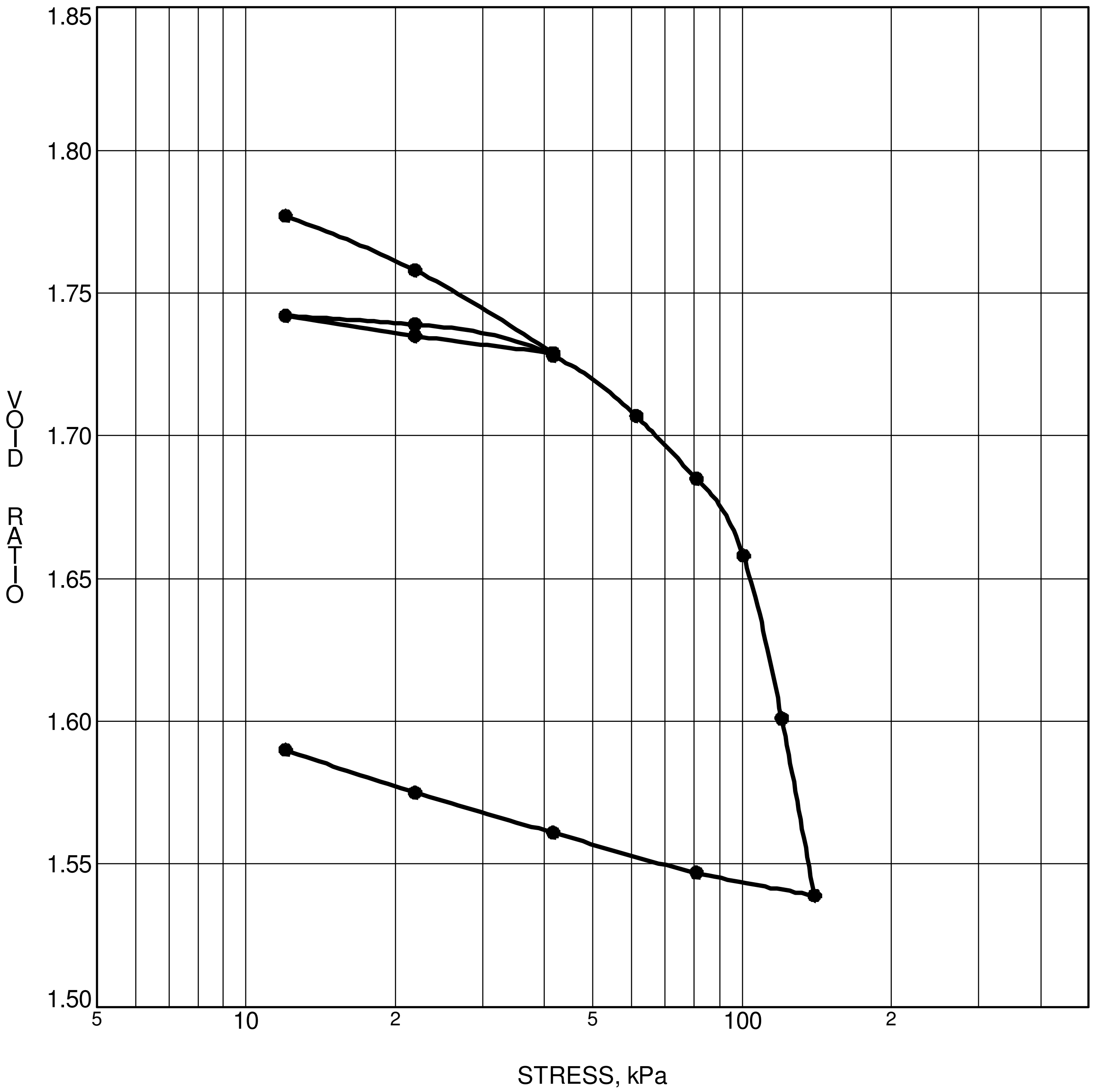
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 9**	p'_o	**79** kPa	Ccr	**0.023**
Sample No.	**TW 5**	p'_c	**112** kPa	Cc	**1.717**
Sample Depth	**8.10** m	OC Ratio	**1.4**	Wo	**71.1** %
Sample Elev.	**83.96** m	Void Ratio	**1.955**	Unit Wt.	**15.6** kN/m^3

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**

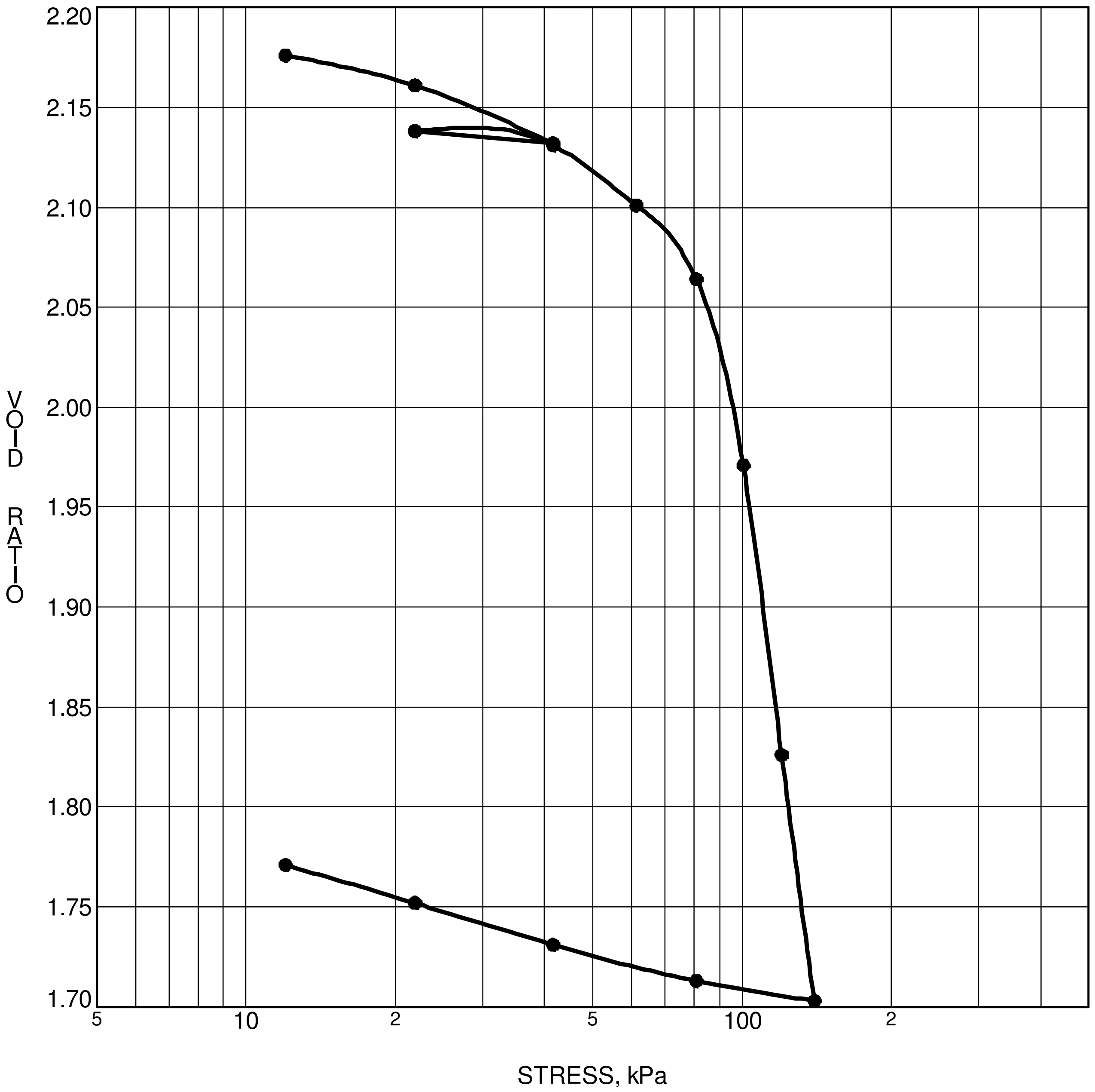
FILE NO. **PG1984**

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.**

FILE NO. **PG1984**

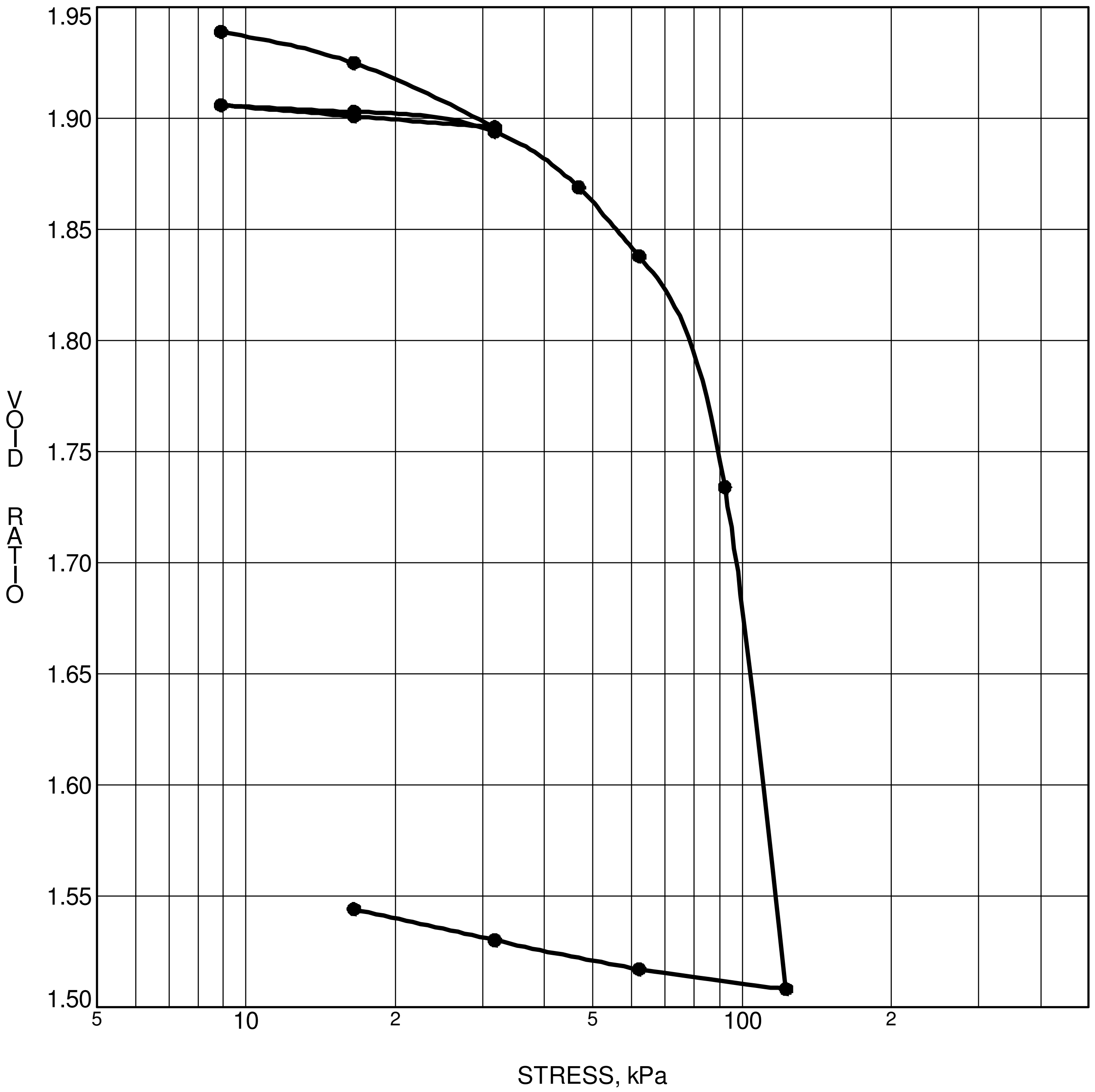
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

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	**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**

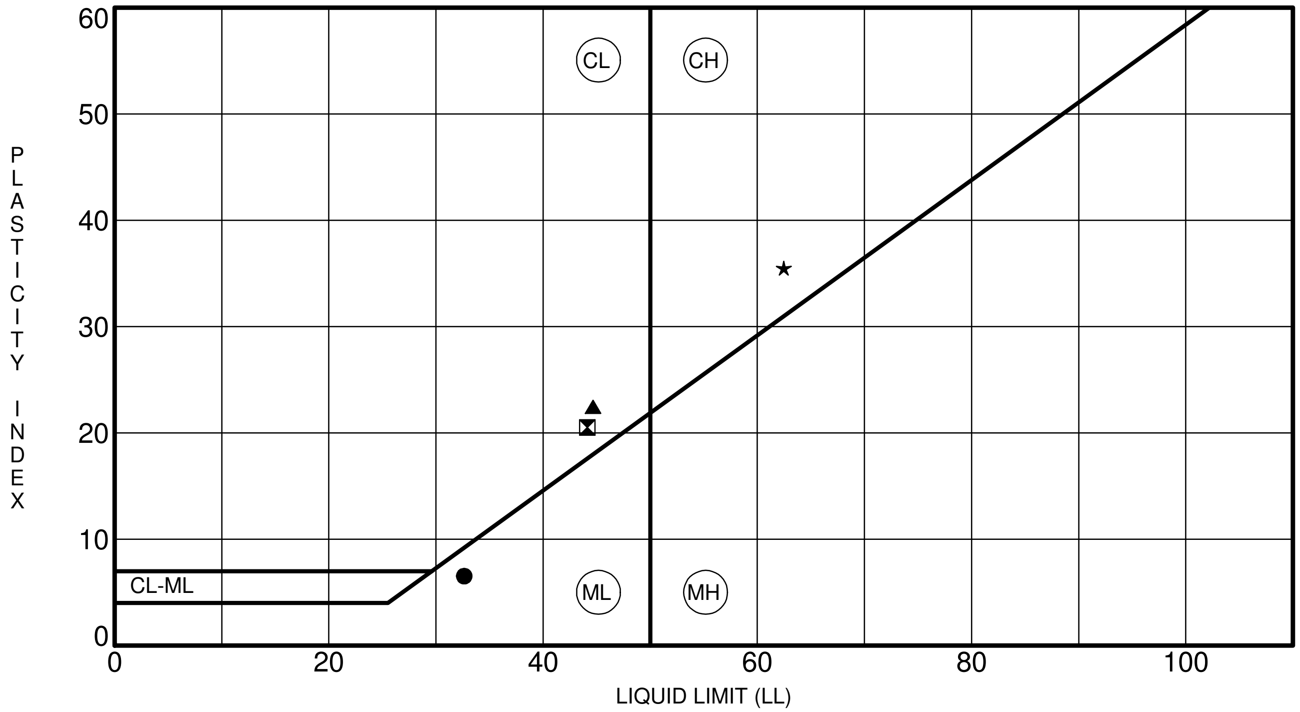
PROJECT **Geotechnical Investigation - Proposed Residential Development - Clarke Lands**

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.**

FILE NO. **PG1984**

PROJECT **Geotechnical Investigation - Prop. Residential Development - Clarke Lands Stage 2**

DATE **28 Jan 11**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Figure 2: Seismic Site Class - OBC 2012

Project: CEPEO Barrhaven High School - Conservative Values - Pile Foundation
File No: PG3968-REP.02 **Date:** 08-Apr-19

PGA 0.32 **Region:** Ottawa (Nepean)

Layer Description	**Layer Properties**		**Cumulative**	**Thickness**
	Vs	**Thickness**	**Thickness**	**Vs**
silty clay crust	300	3.0	3.0	0.0100
grey silty clay (Vs by eqn)	126.8	0.0	3.0	0.0000
Vs = 125 + 1.1667*Z				
grey silty clay (assumed)	120	8.0	11.0	0.0667
post-glacial clay	200	0.0	11.0	0.0000
glacial till	300	5.0	16.0	0.0167
weak or weathered bedrock	1200	1.0	17.0	0.0008
sound bedrock	1500	13.0	30.0	0.0087
Totals	**N/A**	**30.0**	**N/A**	**0.1028**

Average Shear Wave Velocity = **291.7**

Site Class for Seismic Response = **Class** **D**

Site Class	**Description**	**Vs Min.**	**Vs Max.**	**N60 Range**	**Cu Range**
A	Hard rock	1500	>1500	N/A	N/A
B	Rock	760	1500	N/A	N/A
C	Soft rk VD soil	360	760	N>50	Cu>100
D	Stiff soil	180	360	15<N<50	50<Cu<100
E	Soft soil	0	180	N<15	Cu<50

APPENDIX 3

Figure 1 - Key Plan

Drawing PG3968-1 - Revision No. 1
Test Hole Location Plan

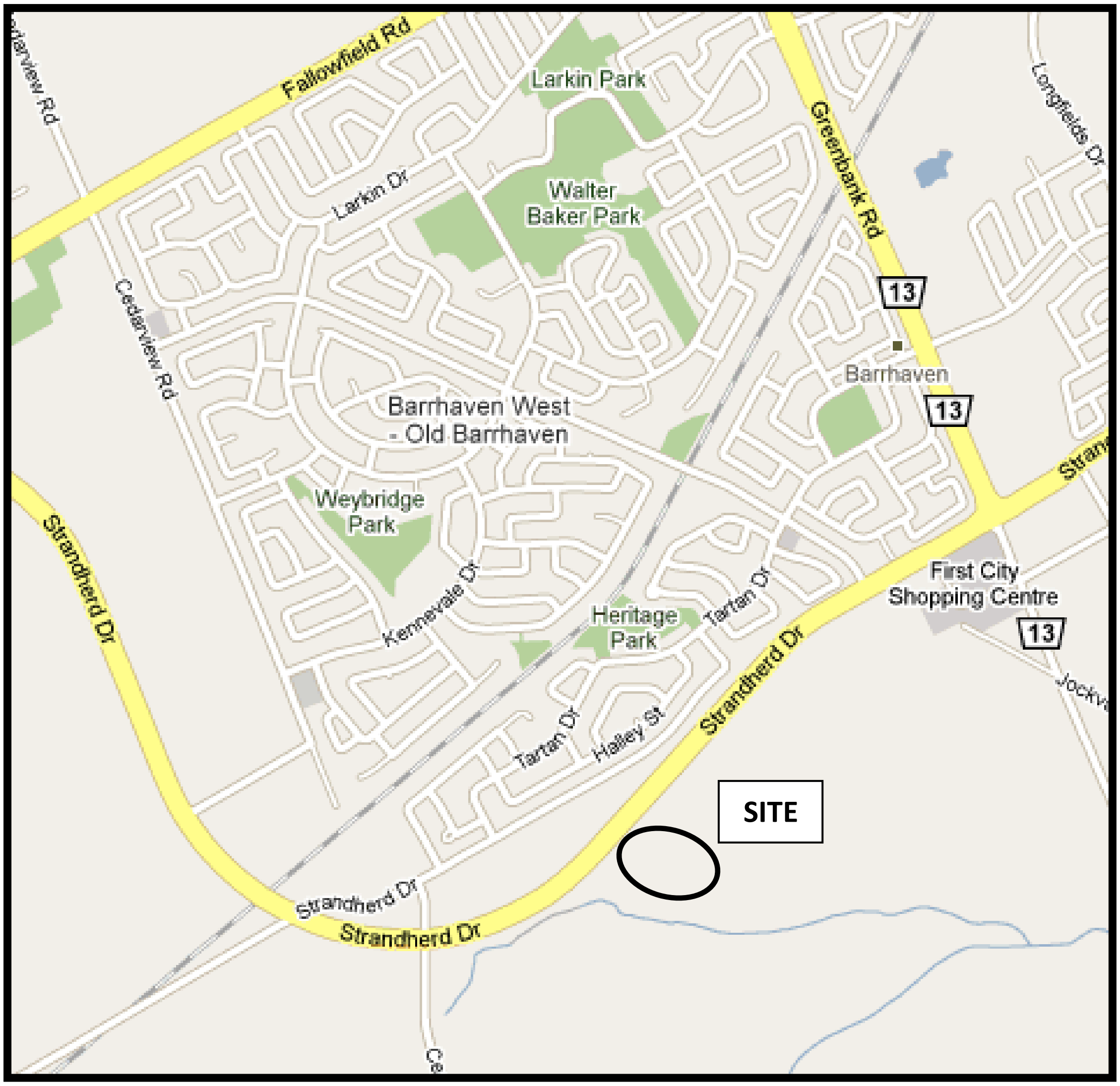
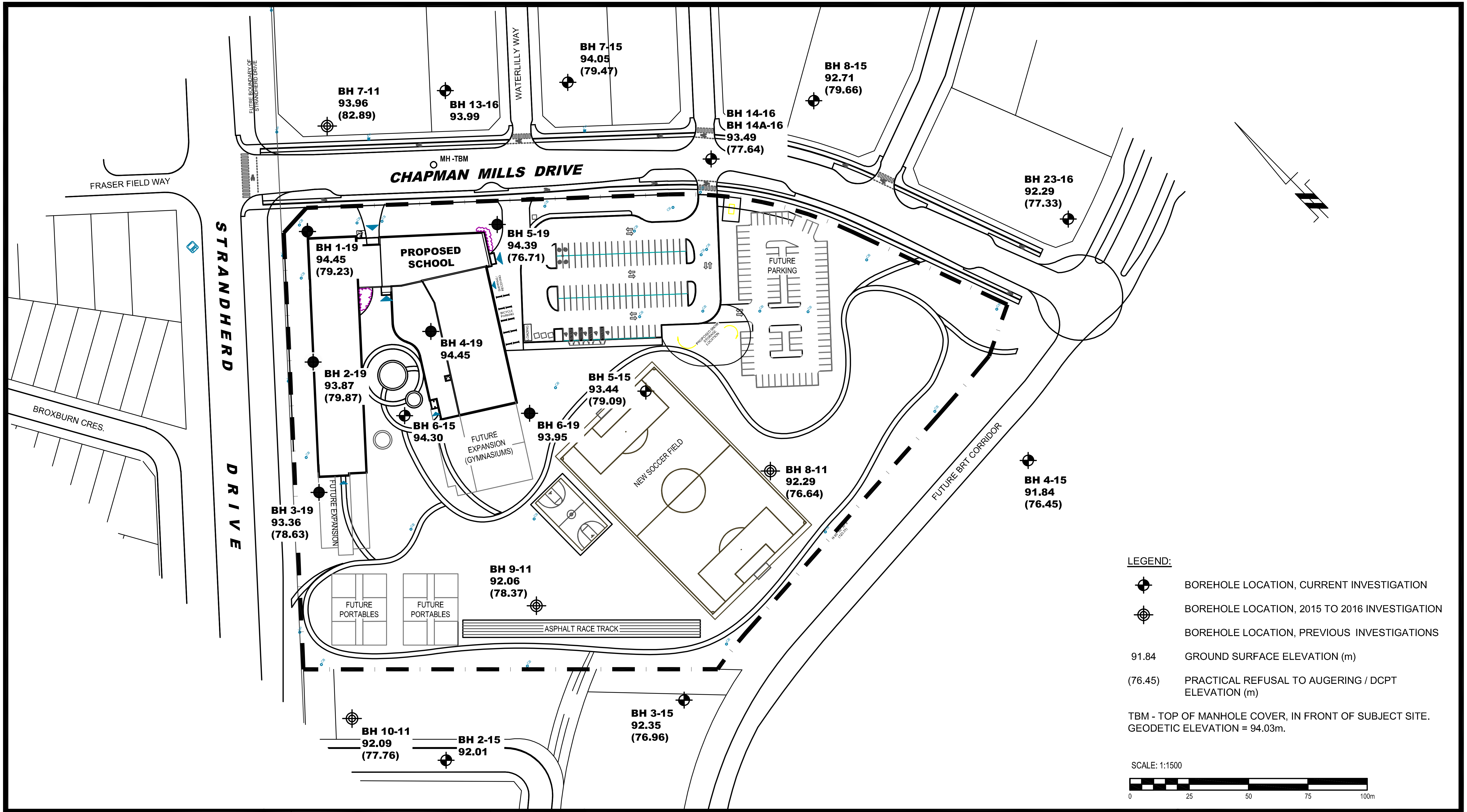


FIGURE 1:

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, 2015 TO 2016 INVESTIGATION
	BOREHOLE LOCATION, PREVIOUS INVESTIGATIONS
91.84	GROUND SURFACE ELEVATION (m)
(76.45)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

TBM - TOP OF MANHOLE COVER, IN FRONT OF SUBJECT SITE.
GEODETIC ELEVATION = 94.03m.

SCALE: 1:1500

0 25 50 75 100m

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NO.	REVISIONS	DATE	INITIAL
1	BASE PLAN UPDATED & 2019 BOREHOLES ADDED	28/03/2019	SD

CEPEO
FINAL GEOTECHNICAL INVESTIGATION
PROPOSED BARRHAVEN HIGH SCHOOL - STRANDHERD DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1500
Date: 11/2016
Drawn by: MPG
Report No.: PG3968-2
Checked by: SD
Dwg. No.: PG3968-1
Approved by: AJT
Revision No.: 1

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