Geotechnical Engineering

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Geotechnical Investigation

Proposed Multi-Storey Buildings
21 Huntmar Drive - Ottawa

Prepared For

North American (Goulbourn)
Limited Partnership

July 7, 2020

Report: PG5006-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by North American (Goulbourne) Limited Partnership (North American) to conduct a geotechnical investigation for the proposed multi-storey residential buildings to be located at 21 Huntmar Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

- ☐ determine the subsurface soil and groundwater conditions by means of boreholes and monitoring well program.
- ☐ provide preliminary geotechnical recommendations for the foundation design of the proposed buildings and provide geotechnical construction precautions which may affect the design.

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

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

2.0 Proposed Development

It is understood that the proposed residential development will consist of two, six-storey apartment buildings. Two levels of underground parking are proposed to occupy the majority of the site to serve both residential buildings. Local roadways and at-grade parking areas are also anticipated for the proposed development. It is further anticipated that the site will be serviced by future municipal services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried on July 15 and 16, 2019. At that time, 5 boreholes were extended to a maximum depth of 9.8 m. A previous investigation was carried on July 13, 2009. At that time, 3 boreholes were extended to a maximum depth of 6.4 m. The test hole locations were distributed across the subject site in a manner to provide general coverage of the proposed buildings. The borehole locations were selected and located in the field by Paterson. The test hole locations are shown on Drawing PG5006-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, and using a 50 mm diameter split-spoon sampler and were placed in sealed plastic bags on site for further review. All the samples were transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH1-19 and BH4-19 to a maximum depth of 14.4 and 12.32 m, respectively. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations

Groundwater

Flexible piezometers were installed in all the boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were determined by Paterson personnel and were located and surveyed in the field by Fairhall, Moffatt & Woodland Limited. It is understood that the boreholes were referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG5006-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from our field investigation were examined in our laboratory to collaborate the field findings.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and mainly grass covered across the majority of the subject site. The ground surface was noted to be relatively flat and at grade with Huntmar Drive. It should be noted that a number of existing water monitoring wells installed by others were observed within the subject site.

The site is bordered to the north by Huntmar Drive, to the west by a residential development and to the east by a commercial development. A residential development currently under construction was noted to the south.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the test hole locations consists of a topsoil overlying a hard to very stiff brown silty clay crust followed by a stiff to firm grey silty clay deposit. Glacial till composed of cobbles, gravel and sand in a silty clay matrix was encountered underlying the silty clay deposit. Practical refusal to DCPT was encountered at BH1 and BH4 at depths of 14.4 m and 12.3 m, respectively, below the existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the area is part of the Gull River formation, which consists of interbedded limestone and dolomite. Also, based on available geological mapping, the overburden thickness is expected to range from 5 to 15 m.

4.3 Groundwater

Groundwater level readings were recorded on July 29, 2019 for the current investigation and on July 16, 2009 for the previous investigation at the piezometer locations. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1, and in Table 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater level can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between 3 to 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH1-19	103.24	3.31	99.93	July 29, 2019
BH2-19	102.82	7.44	95.38	July 29, 2019
BH3-19	102.50	3.03	99.47	July 29, 2019
BH4-19	102.24	3.08	99.16	July 29, 2019
BH5-19	102.46	Blocked	-	-
BH1	102.81	1.76	101.05	July 16, 2009
BH2	102.55	2.28	100.27	July 16, 2009

Note: Ground surface elevations at test hole locations were provided by Fairhall, Moffatt and Woodland Ltd. and are understood to be referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed residential buildings. Based on the results of the field program, it is expected that the proposed buildings may be founded on either conventional spread footings or a raft foundation placed on an undisturbed, firm silty clay bearing surface.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Protective Mud Slab

If a raft slab is selected to support the proposed buildings, it is recommended that a lean concrete mud slab be placed on the undisturbed silty clay subgrade surface to protect it from disturbance due to worker traffic. A minimum 50 mm thick lean concrete mud slab (minimum 15 MPa 28-day compressive strength) is recommended to be poured over the undisturbed silty clay surface once exposed and approved by Paterson personnel.

5.3 Foundation Design

Conventional Spread Footings

Pad footings, up to 5 m wide, and strip footings, up to 3 m wide, placed on an undisturbed, firm silty clay bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed, soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in-situ or not, have been removed, prior to the placement of concrete for footings.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

For 2 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 96.0 to 95.5 m. The bearing medium will consist of a firm grey silty clay which is susceptible to disturbance under construction traffic. The bearing surface should be protected to prevent disturbance as described above.

The contact pressure provided considers the stress relief associated with the soil removal required for 2 levels of underground parking. The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 2 levels of underground parking, a bearing resistance value at SLS (contact pressure) of **170 kPa** will be considered acceptable for a raft supported on the undisturbed, compact sandy silt to silty sand or undisturbed, glacial till bearing surface. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **255 kPa**. For this case, the modulus of subgrade reaction was calculated to be **6.8 MPa/m** for a contact pressure of **170 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Recommendations

Based on the undrained shear strength testing results and experience with the local area of the subject site, **it is recommended that a permissible grade raise restriction of 1.2 m be implemented for the subject site.**

5.4 Design for Earthquakes

A seismic site response **Class D** should be used for design of the proposed buildings at the subject site based on seismic site classification in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, the native soil surface, approved by the Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

Should the proposed building be founded upon a raft slab, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. A minimum 50 mm thick layer of a lean concrete mud slab should be placed to protect the native soil from construction equipment and traffic, as per Subsection 5.2, prior to raft slab construction. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

Should the proposed buildings be founded on footings, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill or a mud slab. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

A sub-floor drainage system, consisting of a series of perforated drainage pipes connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas, access lanes and heavy truck parking areas.

Table 2 - Recommended Pavement Structure - Car Only Parking

<table>
<tr><th>Thickness
(mm)</th><th>Material Description</th></tr>
<tr><td>50</td><td>Wear Course - HL 3 or Superpave 12.5 Asphaltic Concrete</td></tr>
<tr><td>150</td><td>BASE - OPSS Granular A Crushed Stone</td></tr>
<tr><td>300</td><td>SUBBASE - OPSS Granular B Type II</td></tr>
<tr><td colspan="2">SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill</td></tr>
</table>

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

<table>
<tr><th>Thickness
(mm)</th><th>Material Description</th></tr>
<tr><td>40</td><td>Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete</td></tr>
<tr><td>50</td><td>Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete</td></tr>
<tr><td>150</td><td>BASE - OPSS Granular A Crushed Stone</td></tr>
<tr><td>400</td><td>SUBBASE - OPSS Granular B Type II</td></tr>
<tr><td colspan="2">SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill</td></tr>
</table>

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels, it is expected that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 3 m below finished grade down the foundation walls to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation (underside of footing or raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

It is also recommended that a secondary perimeter foundation drainage system be provided at a depth of 2 m below any frost heave sensitive areas, such as exterior sidewalks for the proposed structure, to control any perched water within the backfill material. The perimeter drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, if considered, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Underfloor Drainage

It is anticipated that underfloor drainage may be required to control water infiltration below the underground parking structure. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

It is recommended that 150 mm diameter sleeves, spaced at 6 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular A, should be used for this purpose..

Adverse Effects of Dewatering on Adjacent Properties

Due to the low permeability of the subsoils profile, any minor dewatering will be considered temporary and limited to the local area of the proposed building during the construction period. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to any groundwater lowering.

6.2 Protection Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

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

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system.

The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 4 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 $K\gamma H$ for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of $K\gamma H$ for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Excavation Base Stability

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

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

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems. The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

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

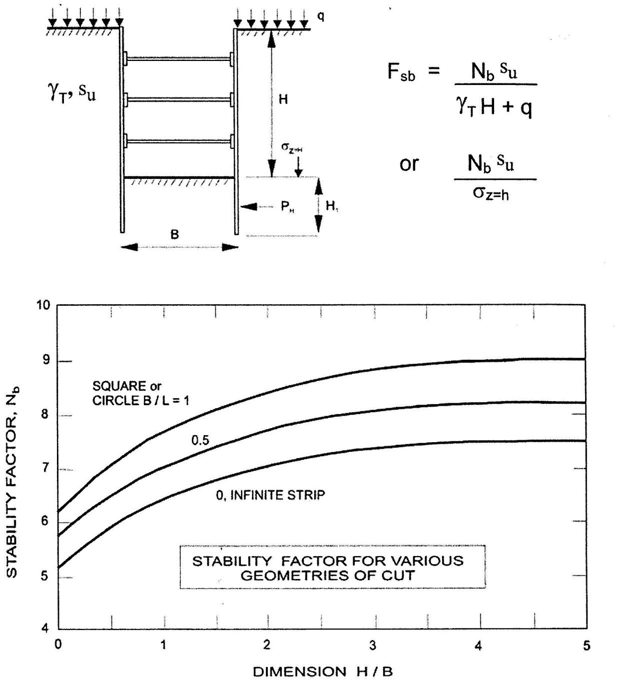


Figure 1 - Stability Factor for Various Geometries of Cut

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

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

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

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

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

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

6.5 Groundwater Control

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

Permit to Take Water

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

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

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.

Impacts on Neighboring Structures

Based on our observations, the long term groundwater level is expected at a depth ranging from 3 to 4 m below the existing grade and within the silty clay deposit. Based on our observations, no groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighboring structures are expected to be founded within the silty clay deposit. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the resistivity indicate the presence of a low to slightly aggressive environment for exposed ferrous metals at this site, which is typical of silty clay samples submitted for the subject area. It is anticipated that standard measures for corrosion protection are sufficient for services placed within the silty clay deposit.

7.0 Recommendations

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

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

☐ Review the Contractor’s design of the temporary shoring system.

☐ Review of waterproofing details for elevator shaft and building sump pits.

☐ Review and inspection of the foundation waterproofing system and all foundation drainage systems.

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

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

☐ Observation of all subgrades prior to placing backfilling materials.

☐ Observation of clay seal placement at specified locations.

☐ Field density tests to ensure that the specified level of compaction has been achieved.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than North American (Goulbourn) L.P. or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 7, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

☐ North American (Goulbourn) L.P.
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 21 Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt and Woodland Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 July 15

FILE NO. PG5006

HOLE NO. BH 1-19

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.46

Very stiff to stiff, brown **SILTY CLAY,** some sand and gravel

- grey by 4.7m depth

8.84

GLACIAL TILL: Compact, grey silty sand with gravel 9.75

Dynamic Cone Penetration Test commenced at 9.75m depth.

Inferred **GLACIAL TILL**

14.38

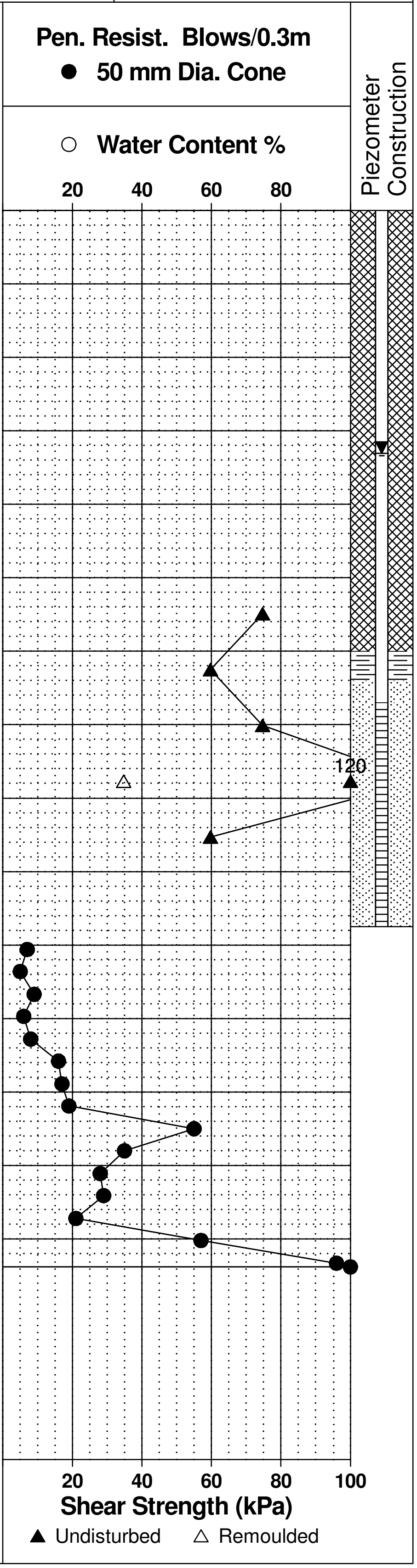
End of Borehole

Practical DCPT refusal at 14.38m depth.

(GWL @ 3.31m - July 29, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	11
SS	3	42	8
SS	4	75	6
SS	5	79	7
SS	6	88	5
SS	7	92	W
SS	8	38	25

DEPTH (m)	ELEV. (m)
0	103.24
1	102.24
2	101.24
3	100.24
4	99.24
5	98.24
6	97.24
7	96.24
8	95.24
9	94.24
10	93.24
11	92.24
12	91.24
13	90.24
14	89.24



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 21 Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt and Woodland Ltd.

FILE NO. **PG5006**

REMARKS

HOLE NO. **BH 2-19**

BORINGS BY CME 55 Power Auger

DATE 2019 July 15

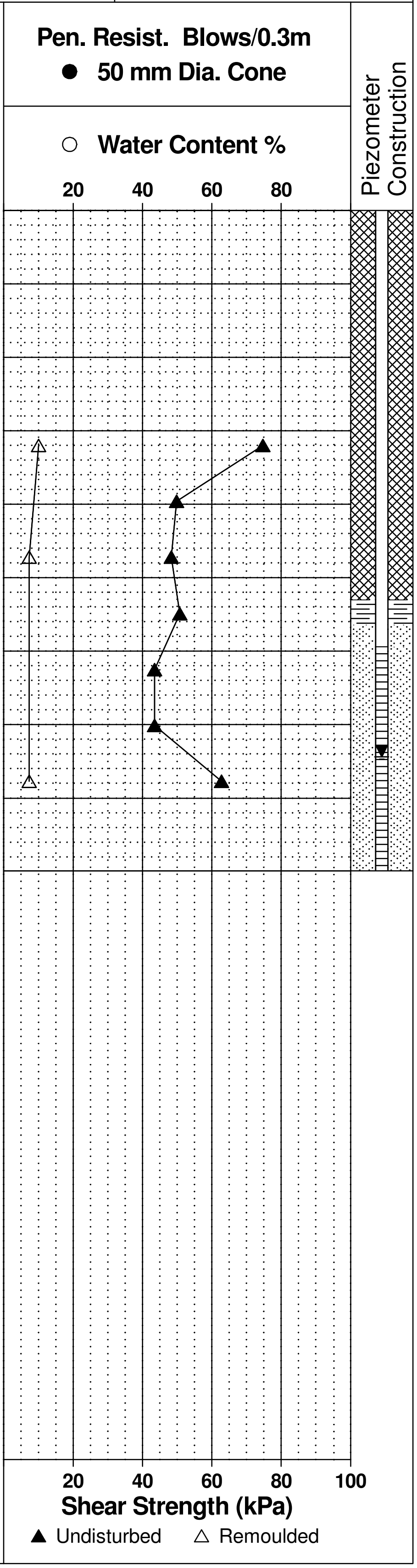
SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.51	AU	1		
Stiff, brown **SILTY CLAY,** some sand - grey by 3.3m depth	SS	2	83	5
	SS	3	100	5
	SS	4	100	5
8.38 **GLACIAL TILL:** Very dense, grey silty sand with gravel, cobbles and boulders 8.99	SS	5	53	50+

End of Borehole

Practical refusal to augering on inferred boulder at 8.99m depth

(GWL @ 7.44m - July 29, 2019)

DEPTH (m)	ELEV. (m)
0	102.82
1	101.82
2	100.82
3	99.82
4	98.82
5	97.82
6	96.82
7	95.82
8	94.82



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 21 Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt and Woodland Ltd.

REMARKS

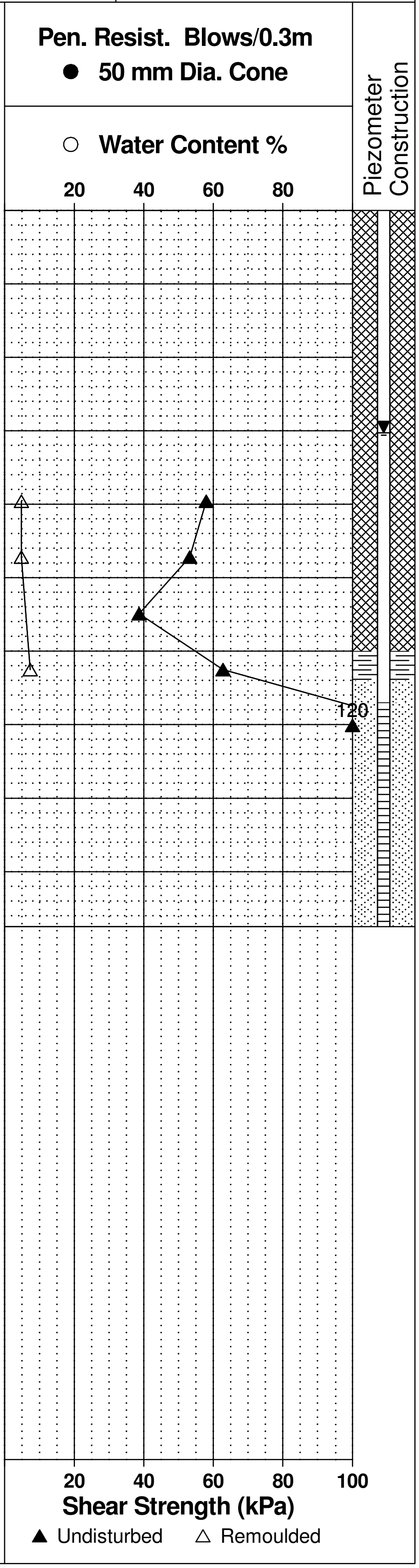
BORINGS BY CME 55 Power Auger

DATE 2019 July 15

FILE NO. PG5006

HOLE NO. BH 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.50
TOPSOIL 0.56		AU	1				
		SS	2	100	5	1	101.50
		SS	3	100	5	2	100.50
		SS	4	75	5		
						3	99.50
Stiff, brown **SILTY CLAY,** some sand		SS	5	100	6		
						4	98.50
- grey by 4.0m depth							
						5	97.50
						6	96.50
6.86							
						7	95.50
Very stiff, grey **CLAYEY SILT**							
		SS	7	58	18		
8.56						8	94.50
GLACIAL TILL: Compact to very dense, grey silty sand with gravel, cobbles and boulders		SS	8		19		
						9	93.50
9.75		SS	9		50+		
End of Borehole							
(GWL @ 3.03m - July 29, 2019)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 21 Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt and Woodland Ltd.

FILE NO. **PG5006**

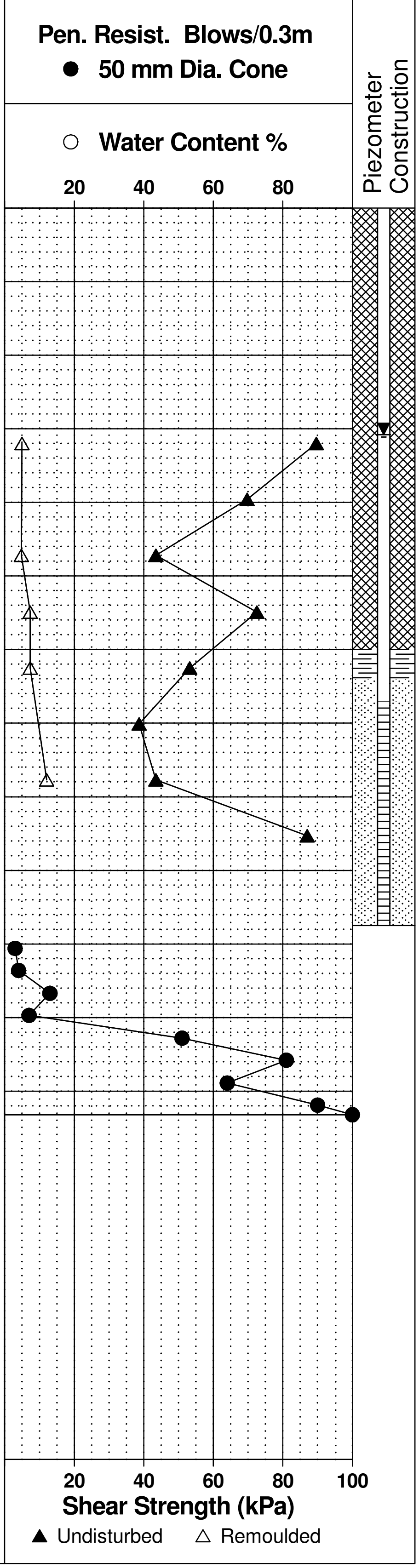
REMARKS

HOLE NO. **BH 4-19**

BORINGS BY CME 55 Power Auger

DATE 2019 July 15

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.24
TOPSOIL 0.51	AU	1				
	SS	2	67	4	1	101.24
	SS	3	54	3	2	100.24
	SS	4	100	4		
Stiff, brown **SILTY CLAY,** some sand					3	99.24
					4	98.24
- firm and grey by 4.6m depth					5	97.24
					6	96.24
					7	95.24
					8	94.24
9.14					9	93.24
Grey **CLAYEY SILT,** trace sand 9.75	SS	6	67	6		
Dynamic Cone Penetration Test commenced at 9.75m depth					10	92.24
					11	91.24
					12	90.24
12.32 End of Borehole Practical DCPT refusal at 12.32m depth (GWL @ 3.08m - July 29, 2019)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 21 Huntmar Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt and Woodland Ltd.

FILE NO. **PG5006**

REMARKS

HOLE NO. **BH 5-19**

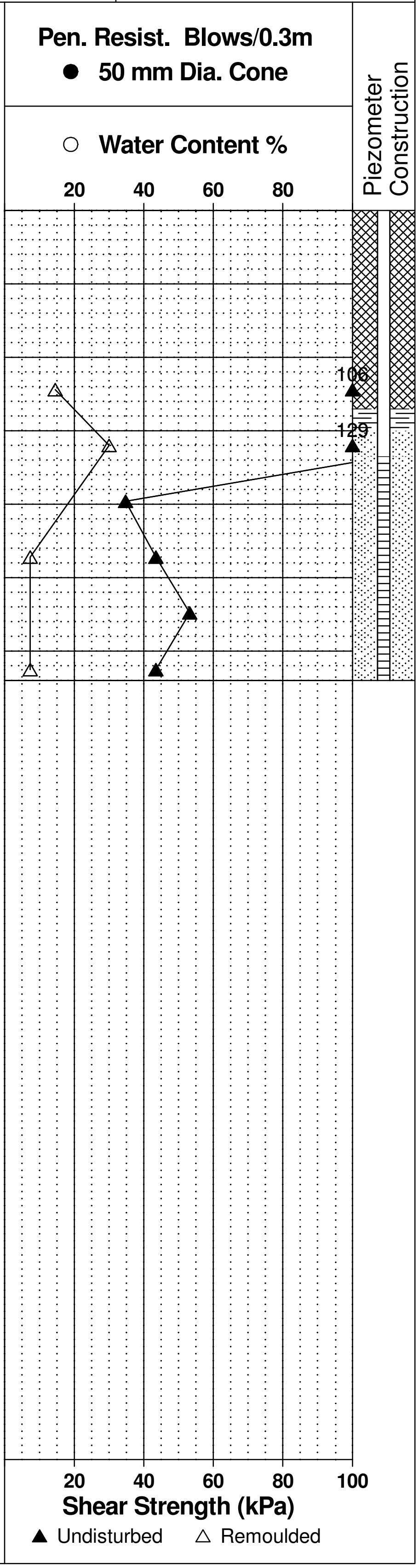
BORINGS BY CME 55 Power Auger

DATE 2019 July 15

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.46
TOPSOIL 0.30	AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	100	3	1	101.46
	SS	3	100	3	2	100.46
					3	99.46
- firm and grey by 3.8m depth					4	98.46
					5	97.46
6.40					6	96.46

End of Borehole

(Piezometer dry/blocked - July 29, 2019)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development-Hazeldean Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt & Woodland Ltd.

REMARKS

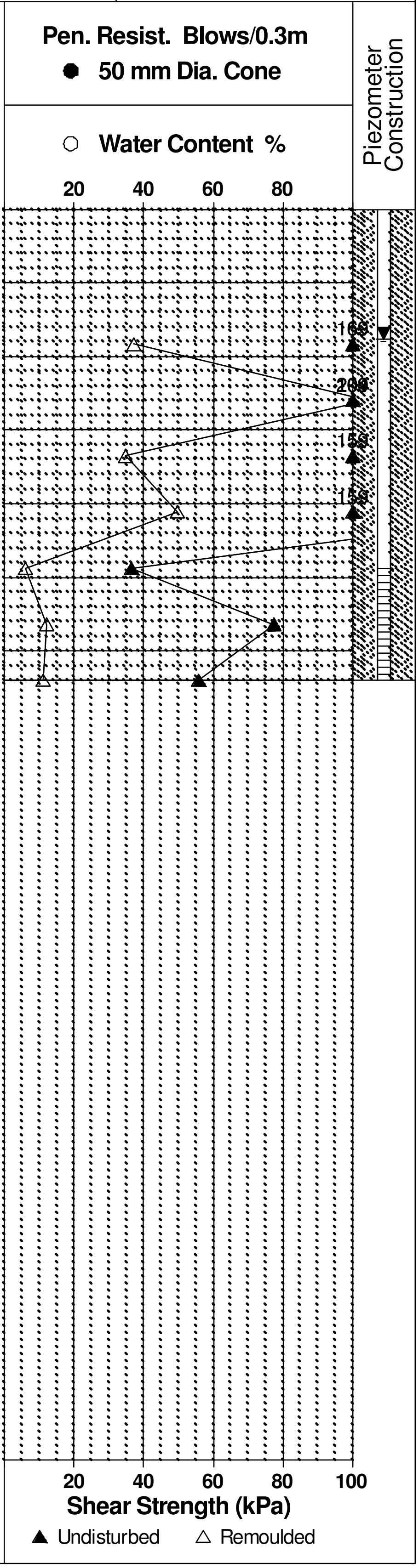
BORINGS BY CME 55 Power Auger

DATE 13 July 2009

FILE NO. PG1899

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.81
TOPSOIL 0.13		AU	1				
		SS	2	58	8	1	101.81
Hard to very stiff, brown **SILTY CLAY**, trace sand and sand seams						2	100.81
						3	99.81
- very stiff to stiff and grey by 2.8m depth						4	98.81
						5	97.81
6.40						6	96.81
End of Borehole (GWL @ 1.76m-July 16/09)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development-Hazeldean Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt & Woodland Ltd.

FILE NO. PG1899

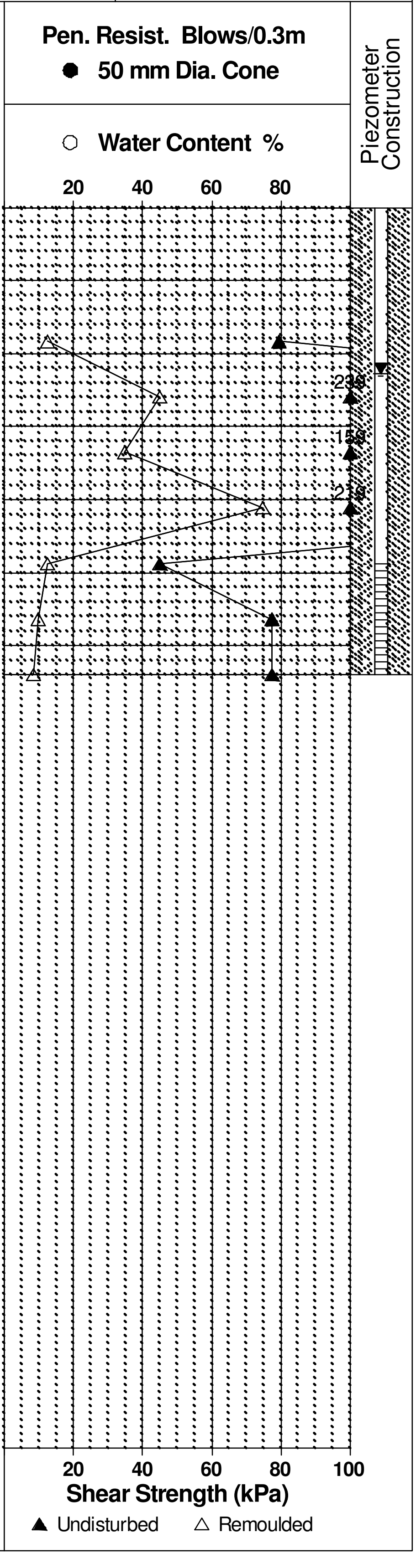
REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE 13 July 2009

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.55
TOPSOIL 0.15							
		SS	1		5	1	101.55
						2	100.55
Hard to very stiff, brown **SILTY CLAY**, trace sand and sand seams						3	99.55
- firm to stiff and grey by 4.3m depth						4	98.55
						5	97.55
						6	96.55
6.40							
End of Borehole (GWL @ 2.28m-July 16/09)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development-Hazeldean Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Fairhall, Moffatt & Woodland Ltd.

FILE NO. PG1899

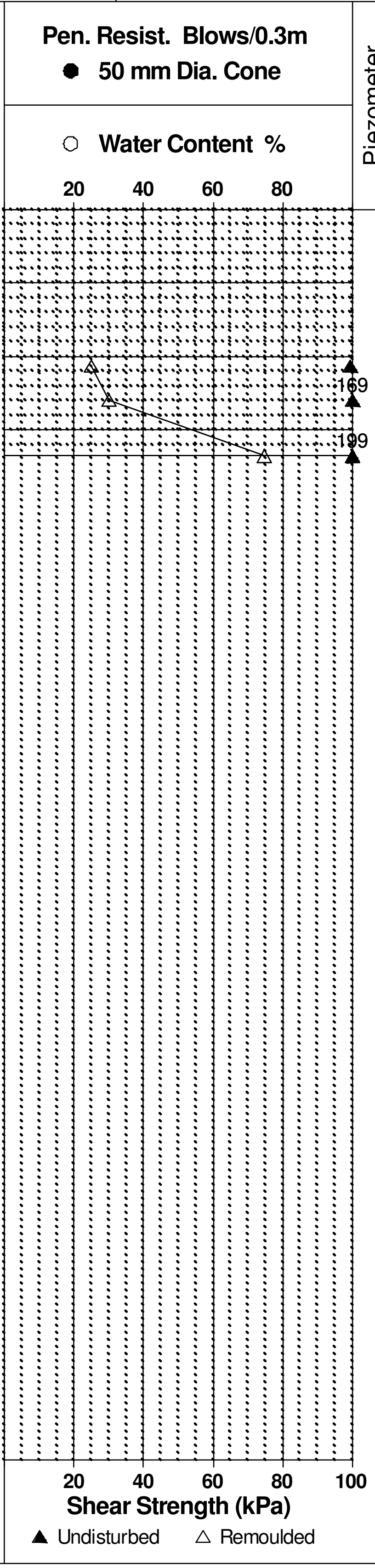
REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger

DATE 13 July 2009

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	102.59	
TOPSOIL 0.15								
Brown **CLAYEY SILT**, some sand 0.69		SS	1		8	1	101.59	
Very stiff, brown **SILTY CLAY**						2	100.59	
3.35						3	99.59	
End of Borehole								



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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

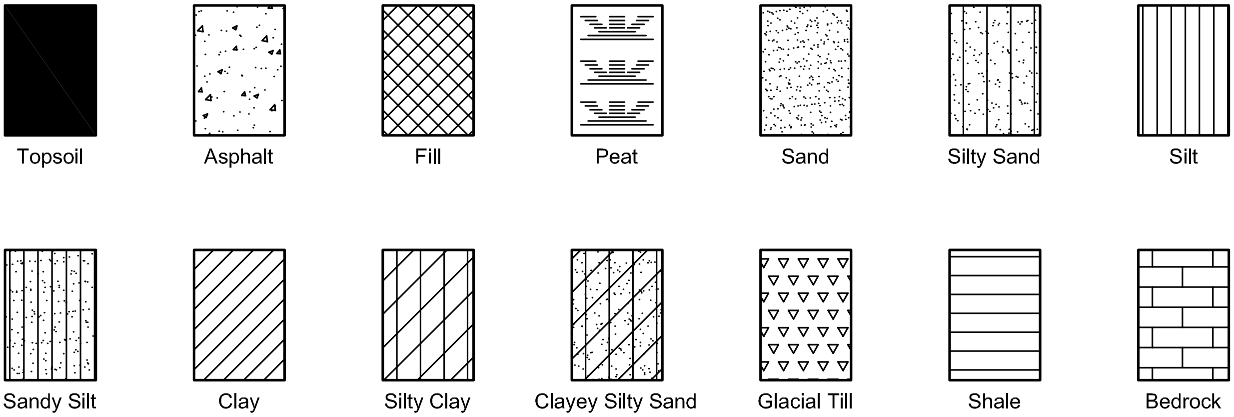
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

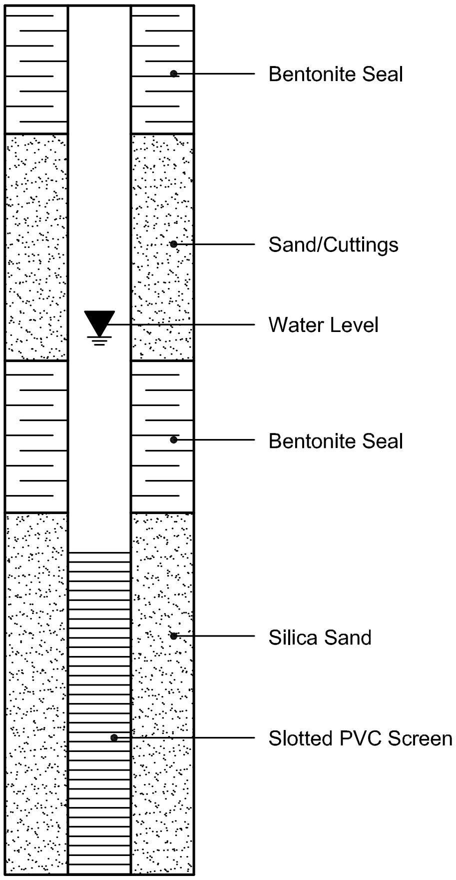
SYMBOLS AND TERMS (continued)

STRATA PLOT

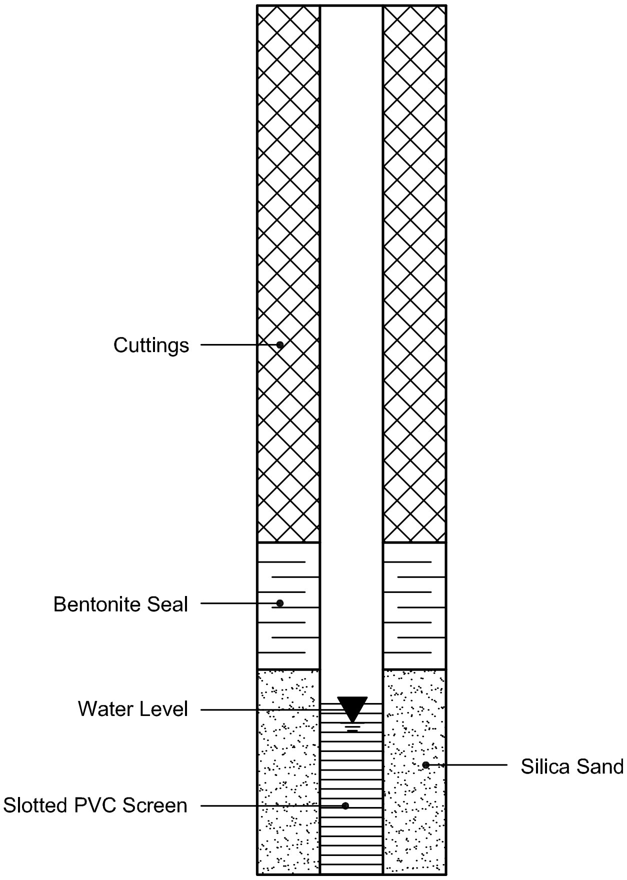


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25602

Report Date: 22-Jul-2019

Order Date: 16-Jul-2019

Project Description: PG1899

	Client ID:	BH5-SS3	-	-	-
	Sample Date:	16-Jul-19 10:00	-	-	-
	Sample ID:	1929245-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.42	-	-	-
Resistivity	0.10 Ohm.m	61.4	-	-	-
Anions					
Chloride	5 ug/g dry	19	-	-	-
Sulphate	5 ug/g dry	5	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5006-1 - TEST HOLE LOCATION PLAN

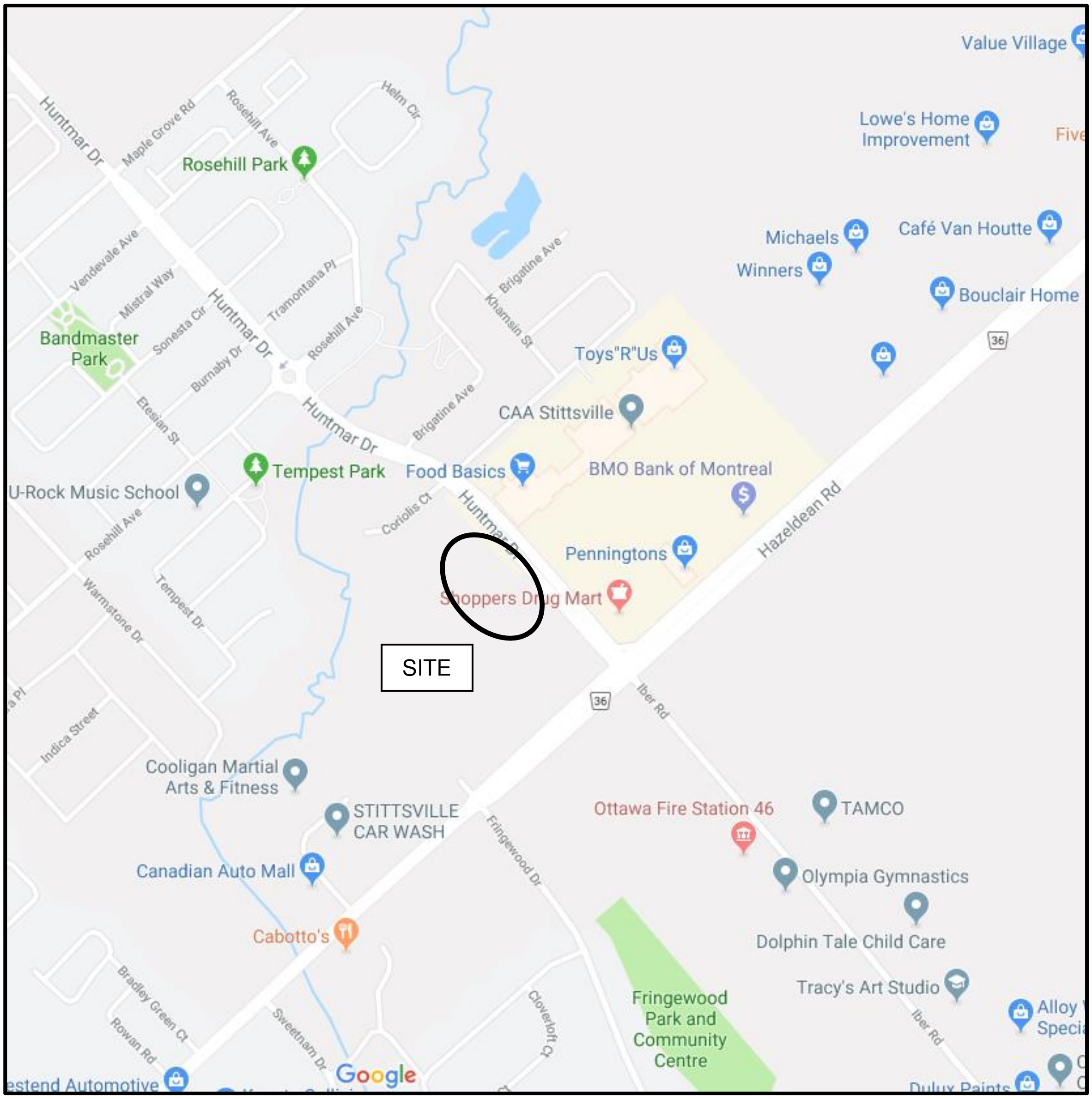
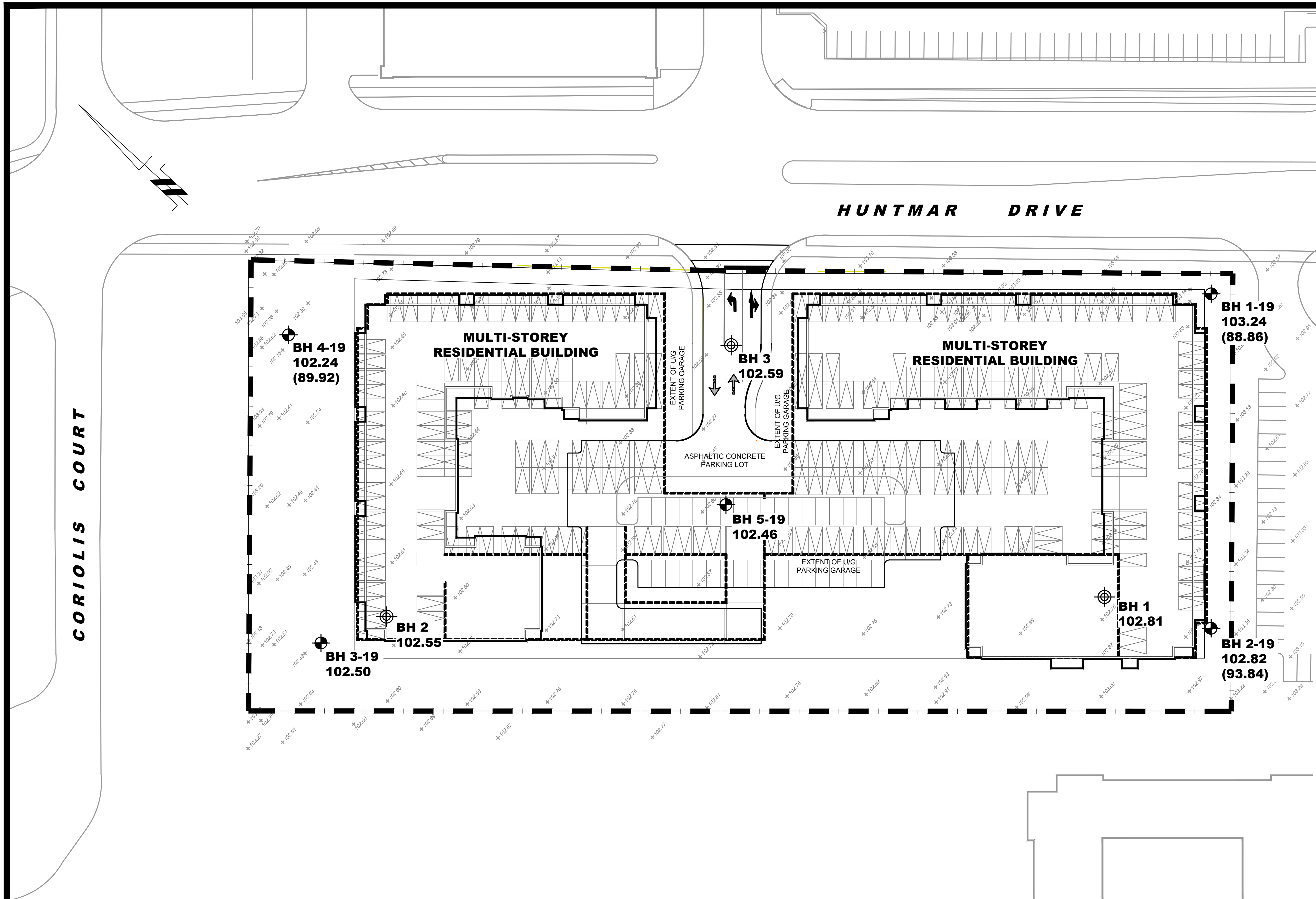


FIGURE 1

KEY PLAN



LEGEND:

◈	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PG1899, 2009
102.82	GROUND SURFACE ELEVATION (m)
(93.84)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY FAIRHALL, MOFFATT AND WOODLAND LTD.
CONCEPTUAL PLAN PROVIDED BY Q4A ARCHITECTS

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST CONCEPTUAL PLAN	30/06/2020	DP

NORTH AMERICAN (GOULBOURN) LIMITED PARTNERSHIP
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDINGS - 21 HUNTMAR DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750
Drawn by: RCG
Checked by: DP
Approved by: DJG

Date: 08/2019
Report No.: PG5006-1
Dwg. No.: PG5006-1
Revision No.: 1

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