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Geotechnical Investigation
Proposed Mixed-Use Office Building
807 - 825 Montreal Road
Ottawa, Ontario

Prepared For

Darwin Group Ottawa

July 7, 2015

Report: PG3530-1

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Table of Contents

Page

APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Darwin Group Ottawa to conduct a geotechnical investigation for the proposed mixed-use office building to be located at 807-825 Montreal Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 PROPOSED PROJECT

It is understood that the proposed development will consist of a nine (9) storey mixed-use office building with 2 levels of underground parking. Associated access lanes and landscaped areas are further anticipated.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on June 4, 2015. At that time, three (3) boreholes were advanced to a maximum depth of 4.8 m. A previous investigation was also carried out by Paterson within the subject site on August 18, 2010 with a total of six (6) boreholes. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG3530-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down 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. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

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

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.

Rock samples were recovered from all the current boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

A flexible polyethylene standpipe was installed in BH 1 and BH 2 to permit the monitoring of the groundwater level subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of a nail in a telephone pole located along the north side of Montreal Road, just south of the subject site. A geodetic elevation of 96.14 m was provided for the TBM. The location of the TBM and boreholes, as well as, the ground surface elevation of each borehole are presented on Drawing PG3530-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

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 analysed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the soil. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is mainly grass covered with mature trees along the north and east portions of the site. Generally, the site topography slopes gradually down to the south and west. It should be noted that the subject site was formerly occupied by two (2) residential buildings which were demolished prior to the time of this investigation.

4.2 Subsurface Profile

The subsurface profile at the borehole locations consists of topsoil overlying a brown silty fine sand with gravel with some organics followed by a limestone bedrock. Practical auger refusal was encountered on inferred bedrock surface at depths varying between 0.5 and 1.2 m. Specific details of the soil profile at each borehole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Diamond drilling was carried out at the borehole locations to determine the nature of the bedrock to assess its quality. The bedrock consists of grey limestone. The recovery values and RQD values for the bedrock cores were calculated. The recovery values varied between 72 and 100% while the RQD values ranged between 18 and 83%. Based on these results, the quality of the bedrock ranges from very poor to good.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone of the Bobcaygeon Formation. The overburden drift thickness is estimated between ground surface and 3 m depth.

4.3 Groundwater

Groundwater level readings were taken at the borehole locations completed as part of our current investigation on June 23, 2015 and are presented in Table 1. Our groundwater measurements are presented in the Soil Profile and Test Data sheets. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher water levels than noted during the investigation.

Groundwater is subject to seasonal fluctuations and therefore, groundwater could vary at the time of construction.

Table 1 - Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater		Date
		Depth (m)	Elevation (m)	
Groundwater Levels - PG3530 Investigation				
BH 1-15	95.05	1.91	93.14	June 23, 2015
BH 2-15	95.92	2.82	93.10	June 23, 2015
BH 3-15	97.73	Dry	n/a	June 23, 2015
Groundwater Levels - PG2203 Investigation				
BH 1	98.66	Dry	n/a	August 25, 2010
BH 4	95.50	Dry	n/a	August 25, 2010

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is expected that the proposed building will be founded directly on a clean, surface sounded bedrock surface.

Bedrock removal will be required to complete the basement level of the proposed multi-storey building. Hoe ramming could be used where only small quantities of bedrock need to be removed. Line drilling and controlled blasting could be used where large quantities of bedrock need to be removed. The blasting operations should be planned and carried out under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Existing foundation walls and other construction debris should be entirely removed from within the proposed building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

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

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm per second during the blasting program to reduce the risks of damage to the existing structures.

Blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

5.3 Foundation Design

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A bearing resistance value at SLS of **3,000 kPa** and a factored bearing resistance value at ULS of **5,000 kPa** could be used if the bedrock is free of seams, fractures and voids within 1.5 m below the bedrock surface. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level along the footing alignments. The drill hole inspection should be carried out by the geotechnical consultant.

Footings bearing on surface sounded bedrock and designed using the above mentioned bearing pressures will be subjected to negligible post-construction total and differential settlements.

5.4 Design for Earthquakes

The proposed building can be designed using a seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). The soils underlying the site are not susceptible to liquefaction. A higher site class, such as Class B or Class A would likely be applicable for the subject site. However, the higher site classes (Class A or B) would have to be determined based on site-specific shear wave velocity testing.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the bedrock surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

The upper 200 mm of sub-slab fill is recommended to consist 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 a minimum of 98% of the SPMDD.

5.6 Basement Wall

The basement walls are understood to be poured against a composite drainage system, which will be placed against the exposed bedrock face. Lateral earth pressures are expected to be negligible for the majority of the basement wall height due to the anticipated method of construction.

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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 bulk (drained) unit weight of 20 kN/m^3.

The foundation is expected be provided with a perimeter drainage system and/or waterproofing system; therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure for all subsurface units below the watertable when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using 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)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. Note that surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

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. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

5.7 Pavement Structure

Recommended Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

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

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material’s SPMDD using suitable vibratory equipment.

5.8 Rock Anchor Design

Overview of Anchor Features

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 a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

The anchor be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of shale ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 4.

Table 4 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Limestone	80 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 5. The factored tensile resistance values given in Table 5 are based on a single anchor with no group influence effects.

Table 5 - Recommended Rock Anchor Lengths

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.25	0.5	1.75	250
	2.25	0.5	2.75	500
	3.5	1.1	4.6	1000
	4.2	2	6.2	1500
125	2	0.5	2.5	500
	3	1	4	1000
	3.5	1.5	5	1500

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structure. Insufficient room is expected to be available for exterior backfill. To limit the volume of groundwater handled by the building's sump pit, the following water infiltration system is recommended:

- ☐ A waterproofing membrane should be applied to the prepared vertical bedrock surface from top of bedrock to below grade to the founding elevation. The membrane will serve as a water infiltration suppression system.

- ☐ Composite drainage layer, such as Miradrain G100N or equivalent, will be placed against the waterproofing membrane from the surface to the proposed founding elevation.

Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommend to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, Paterson recommends a 150 mm diameter perforated PVC pipe be placed at 6 m centres across the building's footprint. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of 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 excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

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

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

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

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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 reduce the potential 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.

6.5 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.

A temporary MOE permit to take water (PTTW) may be required if more than 50,000 L/day is expected to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

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

Precaution must be taken where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The analytical testing results are presented in Table 6 along with industry standards for the applicable threshold values. These results are indicative that Type 10 Portland cement (Type GU, or normal cement) would be appropriate for this site.

Table 6 - Corrosion Potential

Parameter	Laboratory Results	Threshold	Commentary
Chloride	37 µg/g	Chloride content less than 400 mg/g	Negligible concern
pH	7.53	pH value less than 5.0	Neutral Soil
Resistivity	63 ohm.m	Resistivity greater than 1,500 ohm.cm	Moderate Corrosion Potential
Sulphate	54 µg/g	Sulphate value greater than 1 mg/g	Negligible Concern

7.0 RECOMMENDATIONS

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

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

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

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

❏ Observation of all subgrades prior to backfilling.

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

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

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

8.0 STATEMENT OF LIMITATIONS

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

A 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 immediate notification 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 Darwin Group Ottawa or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Office Building - 807-825 Montreal Rd.
Ottawa, Ontario

DATUM TBM - Nail in telephone pole located in front of subject site. Geodetic elevation = 96.14m.

REMARKS

BORINGS BY CME 55 Power Auger

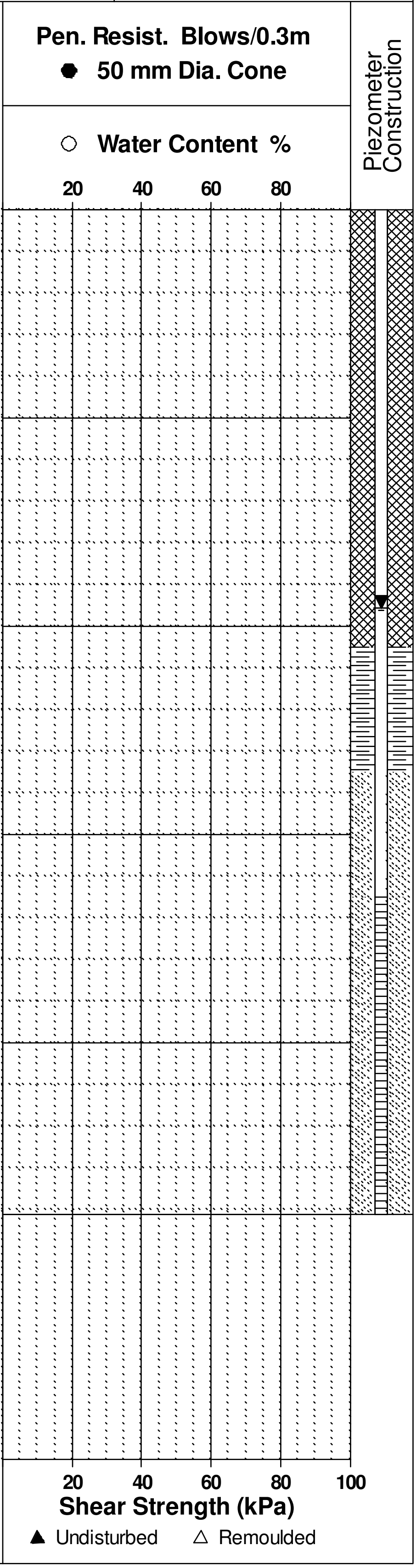
DATE June 4, 2015

FILE NO. **PG3530**

HOLE NO. **BH 1-15**

SOIL DESCRIPTION		SAMPLE			
	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.08				
Brown **SILTY SAND,** trace organics and gravel	1.04	SS	1	67	50+
BEDROCK: Grey limestone interbedded with shale	4.82	RC	1	100	58
		RC	2	100	18
		RC	3	100	67
End of Borehole (GWL @ 1.91m-June 23, 2015)					

DEPTH (m)	ELEV. (m)
0	95.05
1	94.05
2	93.05
3	92.05
4	91.05



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Office Building - 807-825 Montreal Rd.
Ottawa, Ontario

DATUM TBM - Nail in telephone pole located in front of subject site. Geodetic elevation = 96.14m.

REMARKS

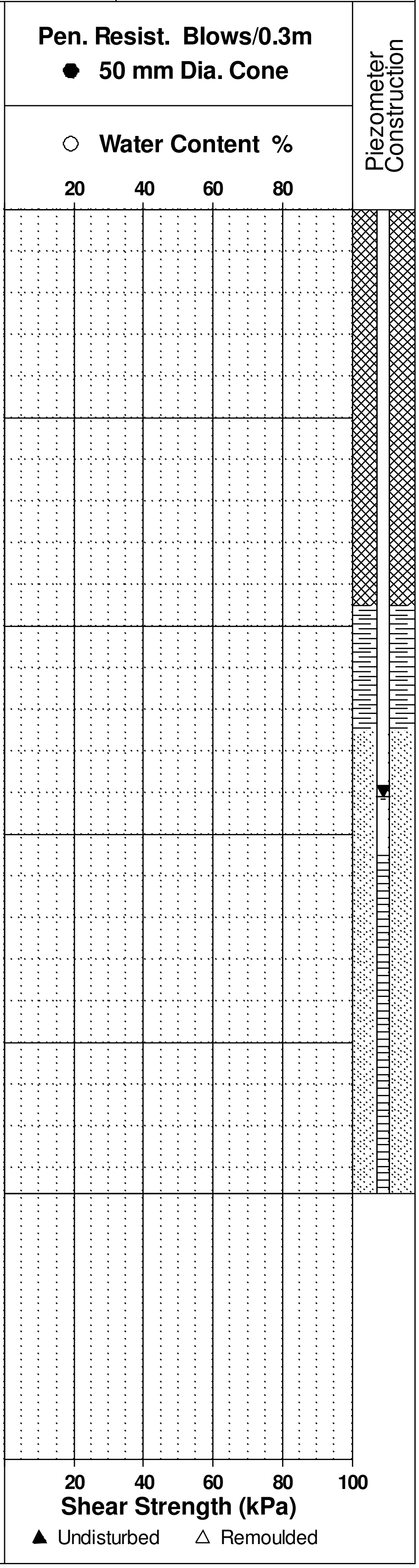
BORINGS BY CME 55 Power Auger

DATE June 4, 2015

FILE NO. PG3530

HOLE NO. BH 2-15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.92
Compact, brown **SILTY SAND,** trace gravel and organics		SS	1	38	11		
		SS	2	46	17	1	94.92
1.22							
BEDROCK: Grey limestone interbedded with shale		RC	1	72	21		
						2	93.92
		RC	2	100	67		
						3	92.92
		RC	3	95	53		
						4	91.92
4.72							
End of Borehole (GWL @ 2.82m-June 23, 2015)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Office Building - 807-825 Montreal Rd.
Ottawa, Ontario

DATUM TBM - Nail in telephone pole located in front of subject site. Geodetic elevation = 96.14m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 4, 2015

FILE NO. PG3530

HOLE NO. BH 3-15

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			20 40 60 80	
GROUND SURFACE							0	97.73		
TOPSOIL	0.05									
Compact, dark brown **SILTY SAND,** trace organics and gravel	0.46		SS	1		50+				
			RC	1	94	76	1	96.73		
BEDROCK: Grey limestone interbedded with shale							2	95.73		
			RC	2	100	83				
	3.28						3	94.73		
End of Borehole (BH dry - June 23, 2015)										

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

FILE NO. PG2203

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

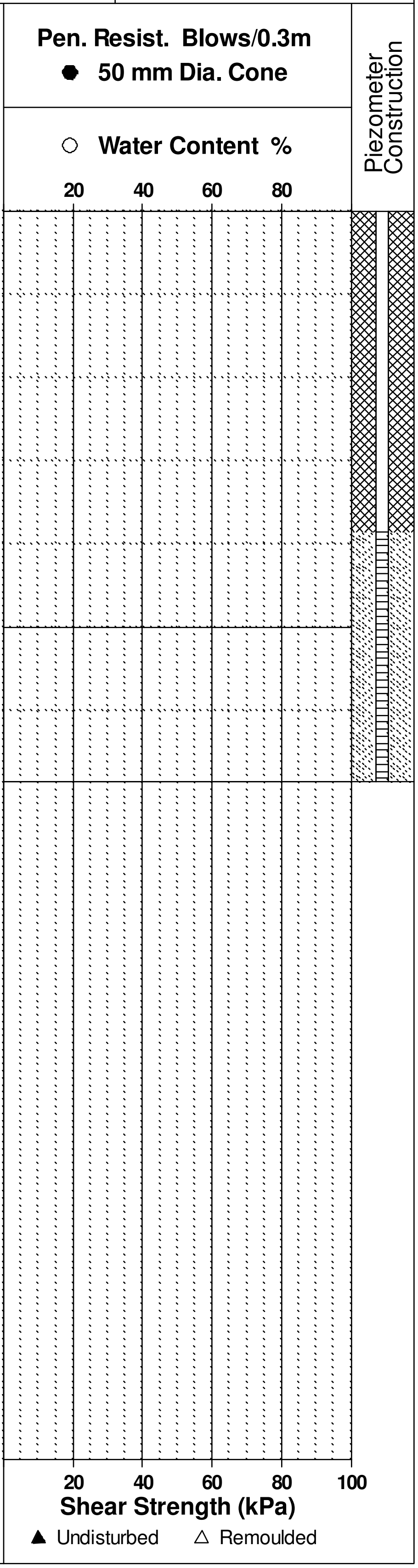
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.66
TOPSOIL 0.13							
Brown **SILTY SAND,** trace organics and gravel in upper 600mm		AU	1				
1.07		SS	2	0	35	1	97.66
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 1.37							

End of Borehole

Practical refusal to augering @ 1.37m depth

(BH dry upon completion)

(Piezometer destroyed - Aug. 25/10)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.10						0	98.25		
Brown **SILTY SAND,** trace organics and gravel 0.71		AU	1						
End of Borehole Practical refusal to augering @ 0.71m depth (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	97.44		
TOPSOIL 0.25									
Brown **SILTY SAND,** trace organics and gravel 0.60		AU	1						
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 1.01		SS	2	0	50+	1	96.44		
End of Borehole Practical refusal to augering @ 1.01m depth (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

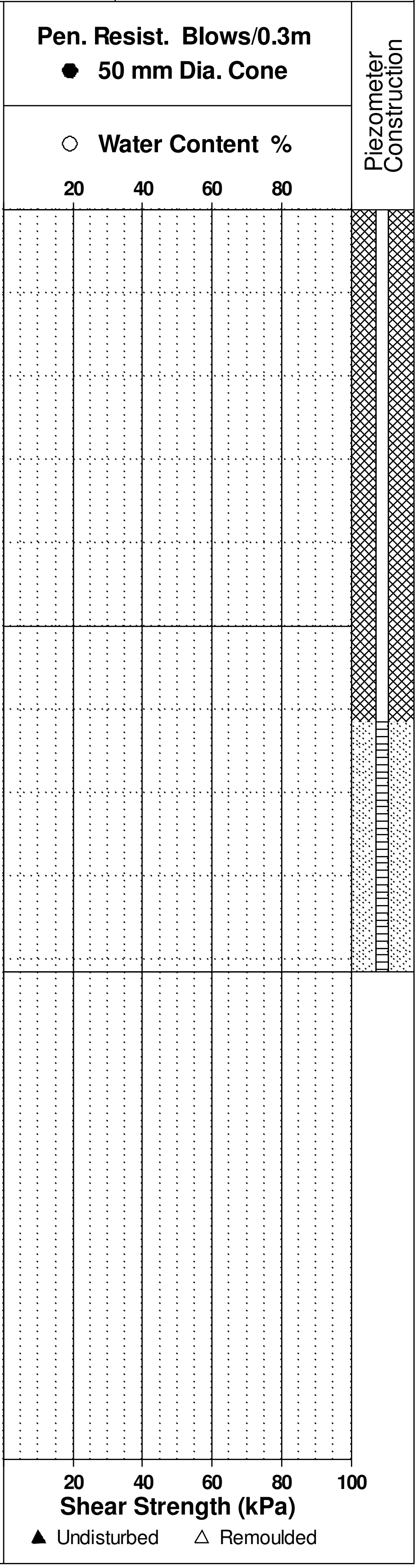
BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.50
TOPSOIL 0.15							
FILL: Brown silty sand with topsoil, gravel and brick - organics noted within upper 600mm of fill layer		AU	1				
		SS	2	50	22	1	94.50
1.37							
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders		SS	3	0	50+		
1.83							
End of Borehole Practical refusal to augering @ 1.83m depth (BH dry upon completion) (Piezometer dry - Aug. 25/10)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	95.96		
TOPSOIL 0.20									
Brown **SILTY SAND,** gravel - trace organics noted in upper 600mm		AU	1						
0.91		SS	2	0	50+				
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders						1	94.96		
1.50									
End of Borehole Practical refusal to augering @ 1.50m depth (BH dry upon completion)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	96.01		
Asphaltic concrete 0.06									
FILL: Brown silty sand with crushed stone 0.68		AU	1						
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 0.97		SS	2	0	50+				
End of Borehole Practical refusal to augering @ 0.97m depth (BH dry upon completion)									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. PH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	98.35		
OVERBURDEN									
1.01						1	97.35		
End of Probehole Practical refusal to augering @ 1.01m depth									

Water Content % scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Seniors Residence - 817 Montreal Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant near the southeast corner of subject site. Geodetic elevation = 97.304m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 18, 2010

FILE NO. PG2203

HOLE NO. PH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	98.15		
OVERBURDEN	▨	AU	1						
						1	97.15		
1.07 End of Probehole Practical refusal to augering @ 1.07m depth									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, 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.

Relative Density	‘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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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)

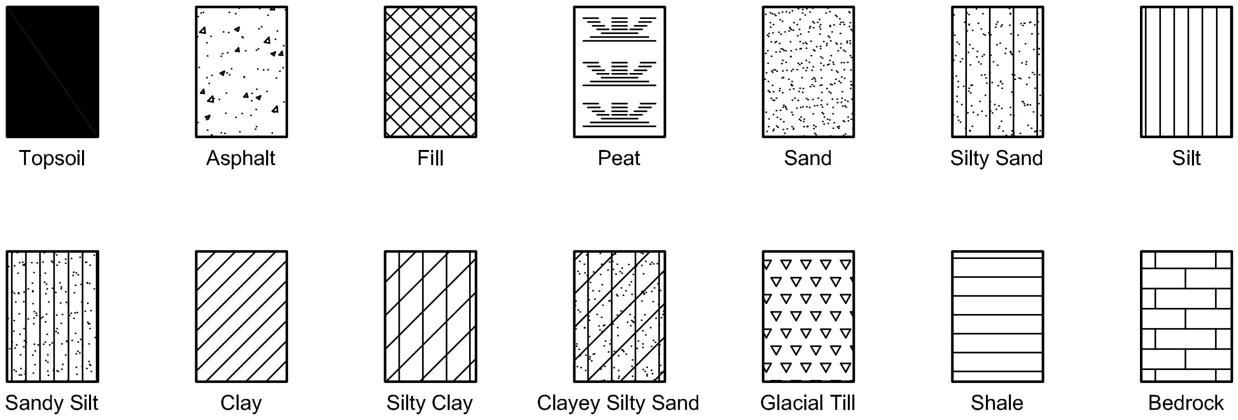
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

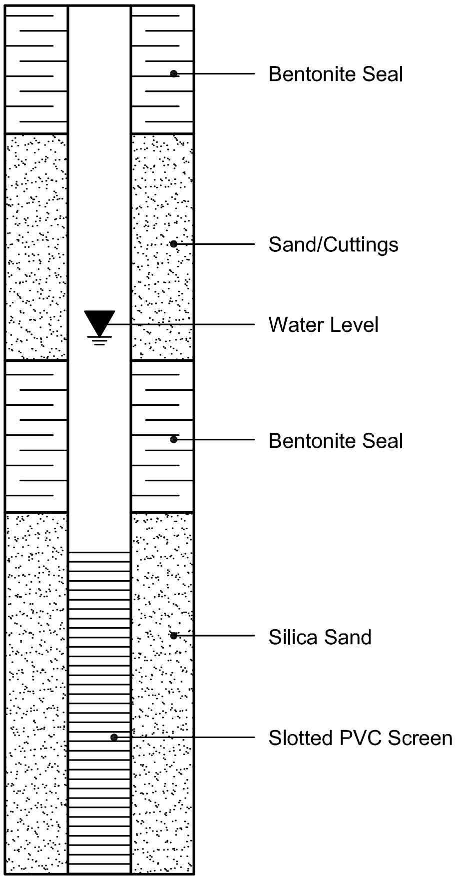
SYMBOLS AND TERMS (continued)

STRATA PLOT

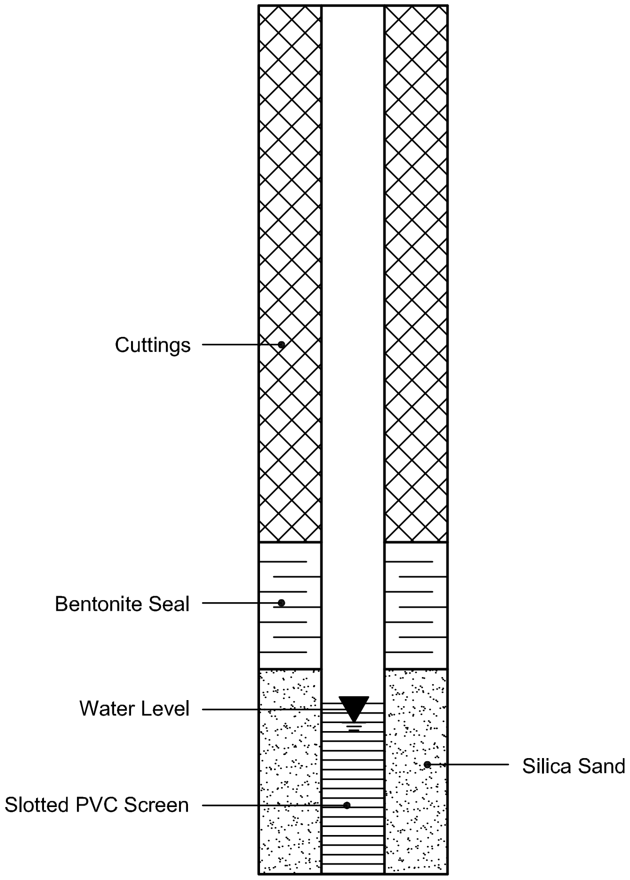


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Order #: 1035149

Certificate of Analysis

Report Date: 30-Aug-2010
Order Date:25-Aug-2010

Client: **Paterson Group Consulting Engineers**
Client PO: 9541 Project Description: PG2203

	Client ID:	BH5-SS2	-	-	-
	Sample Date:	18-Jul-10	-	-	-
	Sample ID:	1035149-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	96.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.53	-	-	-
Resistivity	0.10 Ohm.m	63.0	-	-	-
Anions					
Chloride	5 ug/g dry	37	-	-	-
Sulphate	5 ug/g dry	54	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3530-1 - TEST HOLE LOCATION PLAN

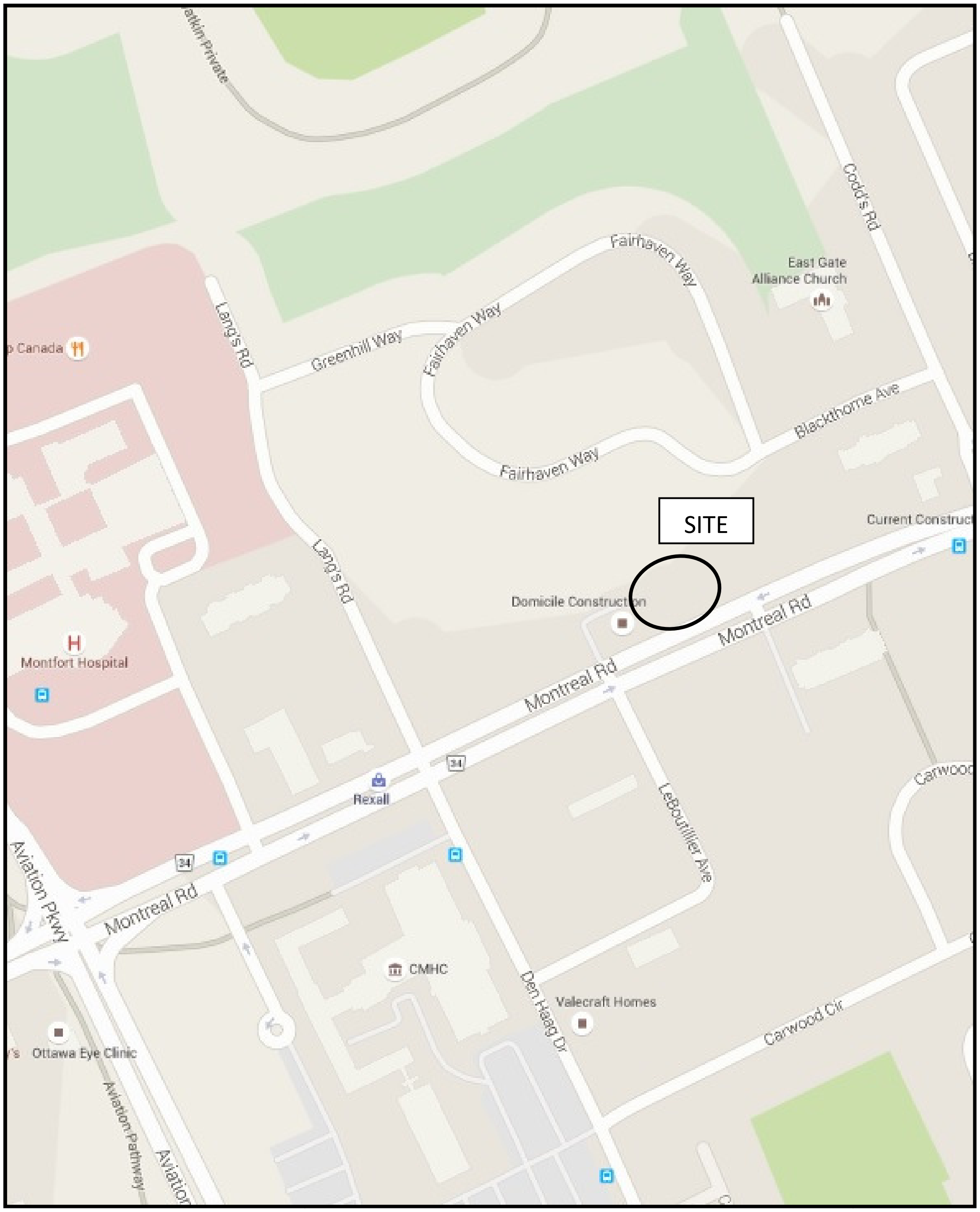
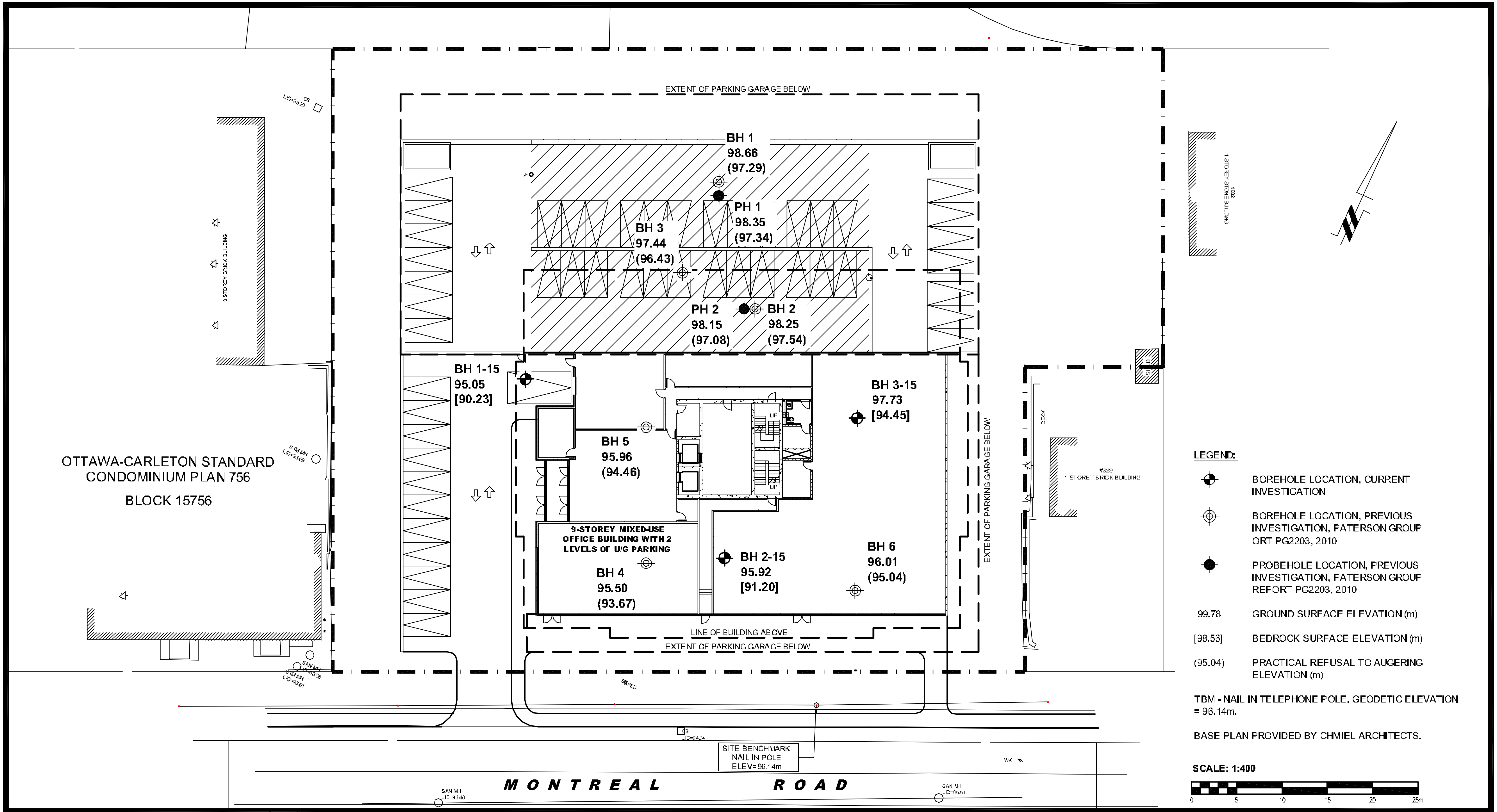


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION, CURRENT INVESTIGATION
⌖	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP ORT PG2203, 2010
●	PROBEHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG2203, 2010
99.78	GROUND SURFACE ELEVATION (m)
[98.56]	BEDROCK SURFACE ELEVATION (m)
(95.04)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - NAIL IN TELEPHONE POLE. GEODETIC ELEVATION = 96.14m.

BASE PLAN PROVIDED BY CHMIEL ARCHITECTS.

SCALE: 1:400

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consulting engineers

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NO.	REVISIONS	DATE	INITIAL

DARWIN GROUP OTTAWA

GEOTECHNICAL INVESTIGATION
PROP. MIXED-USED OFFICE BUILDING - 807-825 MONTREAL ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 06/2015
Drawn by: MPG	Report No.: PG3530-1
Checked by: FA	Drawing No.: PG3530-1
Approved by: DJG	Revision No.:

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