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Proposed Multi-Storey Redevelopment
1354 to 1376 Carling Avenue
Ottawa, Ontario

Prepared For

Holloway Lodging Corporation

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Report PG3736-1 Revision 2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Holloway Lodging Corporation to conduct a geotechnical investigation for the proposed multi-storey redevelopment to be located at 1354 to 1376 Carling Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the 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.

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 Project

Based on the current site plan, it is our understanding that the proposed redevelopment will consist of four multi-storey buildings with one basement level and/or one to two levels of underground parking along with at grade parking areas and access lanes. It is further understood that the existing buildings will be demolished as part of the proposed redevelopment.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechncial investigation was carried out on October 26, 27, 28, 30 and November 1, 2016. During that time, a total of thirteen (13) boreholes were advanced to a maximum depth of 10 m below existing ground surface. A supplemental geotechnical investigation was carried out on August 15 and 16, 2017 which consisted of five (5) boreholes (BH1-17 to BH5-17) advanced to a maximum depth of 13 m below existing ground surface. The borehole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the proposed development taking into consideration of existing site features and underground services. The locations of the boreholes are presented in Drawing PG3736-1 - Test Hole Location Plan included in Appendix 2.

The exterior boreholes were put down using a truck-mounted auger drill rig operated by a two person crew and the interior boreholes were put down using portable drilling operations. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights, using a 50 mm diameter split-spoon sampler or using 47.6 mm inside diameter coring equipment. All soil samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger, split spoon and rock core samples were recovered from the test holes are shown as, AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented 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.

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

Dynamic cone penetration testing (DCPT) was conducted at BH 2 and BH 4 during our field investigation. The DCPT consists of driving a steel rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment of penetration.

Rock coring was carried out in BH 1-17 to BH 4-17 of the August 2017 investigation to confirm depth to bedrock. 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.

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

Groundwater

51 mm diameter PVC groundwater monitoring wells were installed in BH 1, BH 2, BH 4, BH 9, BH 11 and BH13 and flexible polytubing was installed in BH 3, BH 5, BH 6, BH 10 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Flexible polytubing was also installed in each of the five boreholes drilled for the supplemental geotechnical investigation (BH 1-17 to BH 5-17).

Monitoring Well Installation

Typical monitoring well construction details (2016) are described below:

❏ 1.5 m long slotted 51 mm diameter PVC screen.
❏ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
❏ No.3 silica sand backfill within annular space around screen.
❏ Bentonite hole plug directly above PVC slotted screen to approximately 300 mm below the ground surface.
❏ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground utilities and existing site features. The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located at the southeast corner of the intersection of Archibald Street and Carling Avenue. A geodetic elevation of 75.14 m was provided for the TBM based on the site plan prepared by Annis, O'Sullivan, Vollebekk Ltd. The location of the TBM, boreholes and ground surface elevation at each borehole location are presented on Drawing PG3736-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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.

3.4 Analytical Testing

One soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Subsection 6.7 and shown in appendix 1.

4.0 Observations

4.1 Surface Conditions

The west portion of the site is currently occupied by a one to three storey hotel with a basement level. The central portion of the site is occupied by a multi-storey structure with a basement level. The east portion of the site is occupied by an underground parking structure. The remainder of the site consists of asphalt covered car parking, access lanes and landscaping areas.

The subject site is relatively flat and approximately at grade with adjacent roadways and neighbouring properties.

4.2 Subsurface Profile

Generally, the subsoil profile encountered at the borehole locations consists of an asphalt pavement structure overlying a fill layer followed by a native, very stiff silty clay deposit. A glacial till layer was encountered below the silty clay deposit. The fine soil matrix of the glacial till deposit was noted to consist of a grey silty clay to sandy silt with gravel, cobbles and boulders.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

Bedrock Depth

Practical refusal to DCPT/augering was encountered at depths varying between 6 to 10.1 m depth at BH 2, BH 3, BH 4, BH 5, BH 6, BH 8, BH 9, and BH 10. Local bedrock elevations were confirmed by rock coring at depths varying between 6.2 m and 9.2 m in the August 2017 supplemental investigation.

Available Bedrock Mapping

Based on available geological mapping, the subject site is located in an area where bedrock consists of limestone and dolomite (dolostone) of the Gull River Formation, and is expected at depths between 5 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and polytubing standpipes installed in the boreholes upon completion of the sampling program. The groundwater level readings are presented in Table 1. Long-term groundwater level can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 4 m depth.

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

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)		Recording Date
		Depth	Elevation	
* BH 1	75.28	4.25	71.03	November 7, 2016
* BH 2	75.13	3.84	71.29	November 7, 2016
BH 3	74.85	3.50	71.35	November 7, 2016
* BH 4	74.48	2.95	71.53	November 7, 2016
BH 5	74.22	3.60	70.62	November 7, 2016
BH 6	74.12	2.62	71.50	November 7, 2016
* BH 9	74.28	2.20	72.08	November 7, 2016
BH 10	74.11	Damaged	-	November 7, 2016
** BH 11	72.67	1.31	71.36	November 7, 2016
** BH 12	71.49	2.80	68.69	November 7, 2016
*** BH1-17	75.22	3.72	71.50	August 24, 2017
*** BH2-17	74.65	2.82	71.83	August 24, 2017
*** BH3-17	74.21	2.75	71.46	August 24, 2017
*** BH4-17	74.36	2.45	71.91	August 24, 2017
*** BH5-17	74.09	2.37	71.72	August 24, 2017

Notes: * Denotes exterior borehole instrumented with a monitoring well.
** Denotes interior borehole instrumented with a monitoring well.
*** Denotes exterior borehole instrumented with a flexible polytubing standpipe.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed redevelopment. Foundation options are dependent on the design building loading requirement and depth of the foundation. Several foundation options are listed below and discussed in the following sub-sections:

☐ Conventional pad and strip footings placed on an undisturbed stiff silty clay and/or compact glacial till bearing surface.

☐ Raft foundation.

☐ End bearing piled foundation.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and asphalt should be removed from within the perimeter of the proposed building and other settlement sensitive structures. Foundation walls, underground services, and other construction debris should be entirely removed from within the perimeter of the buildings. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's 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 composite drainage membrane.

5.3 Foundation Design

Several foundation options have been considered for the proposed multi-storey building which are dependent on the design loading requirements and foundation depth. The options are further discussed below.

Conventional Shallow Foundation

The bearing resistance values are provided on the assumption that the footings will be placed on bearing surfaces consisting of native undisturbed soil. The bearing surfaces should be free of fill, topsoil, surface water and deleterious materials, such as loose, frozen or disturbed soil prior to placing concrete.

Table 2 - Bearing Resistance Values at Limit States

Founding Layer	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Stiff Silty Clay (above elevation 71.0 m)	150	275
Stiff to Firm Silty Clay (below elevation 71.0 m)	100	180
Compact Glacial Till	200	350
Surface sounded bedrock	-	1,500

Note:
- ☐ SLS - Serviceability Limit States
- ☐ ULS - Ultimate Limit States
- ☐ A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The SLS values are based on a total settlement of 25 mm, and a differential settlement of 20 mm between adjacent footings, both founded on a similar bearing medium.

For areas where bedrock is encountered below design underside of footing level, lean concrete (min. 15 MPa) in-filled trenches could be used to extend footings to an approved bedrock surface. Near vertical, zero entry trenches extending at least 150 mm wider than the proposed footing face should be extended through the glacial till to a bedrock bearing surface. The bearing surface should be inspected by Paterson personnel and in-filled with a lean concrete to design underside of footing level.

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 a stiff silty clay or glacial till bearing medium 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 or engineered fill of the same or higher capacity as the soil.

Raft Foundation

Alternatively, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **200 kPa** can be used for design purposes. 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. The contact pressure provided considers the stress relief associated with the soil removal associated with one underground parking level. The factored bearing resistance (contact pressure) at ULS can be taken as **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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.

Modulus of Subgrade Reaction

Typical values of subgrade modulus for a compact glacial till and stiff silty clay are provided in Table 3.

Table 3 - Modulus of Subgrade Reaction

Soil Type	Modulus of Soil Reaction (MPa/m)
Stiff Silty Clay	5
Compact Glacial Till	30

Piled Foundation

Consideration could be given to transferring the foundation loads through overlying soils to bedrock at depth. This can be achieved with piles driven to end-bearing on or near the bedrock. A suitable pile type would be concrete-filled steel pipe piles driven closed ended and bearing on bedrock.

The axial capacity at serviceability limits of selected end-bearing concrete-filled steel pipe piles are summarized in Table 4.

Table 4 - Summary of Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
193	9.5	730	880	8	24
193	14.5	980	1180	9	34
245	9	940	1130	10	28
245	13	1300	1560	10	42

It should be noted that 193 mm O.D. concrete-filled steel pipe piles will be more susceptible to misalignment and pile toe damage when advancing through boulders or dense materials and/or encountering sloping bedrock.

As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to reduce potential damage to the pile tip during driving.

Provision should be made for restriking all of the piles at least once, 48 hours after the initial driving, to confirm the design set and/or the permanence of the set, and to check for upward displacement due to driving adjacent piles. It is recommended that a pile load test or dynamic monitoring and capacity testing be carried out at an early stage during the piling operations to verify the transferred energy from the pile driving equipment and determine the load carrying capacity of the piles. The recommended number of tests at each building is dependent on the number of piles and pile sizes; as a guideline a minimum of 2 tests per pile size should be carried out. It is also recommended that the tested pile locations be spread out across the proposed building footprints.

The post construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

If piles are to be left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the piles. Further guidelines can be provided on these measures at the time of construction, if required.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave testing was located towards the southern half of the site, as presented in Drawing PG3736-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an east-west orientation along the south property boundary of the site. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and 3, 4.5 and 30 m away from the first and last geophone.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

The glacial till and bedrock velocities were interpreted to be 414 and 2,525 m/s, respectively. It is understood that the currently proposed design includes one level of underground parking. However, consideration is being given to including a second level of underground parking. We have completed an analysis for both scenarios and the details are presented below.

Vs30 - One Underground Parking Level

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012. If the building is founded on glacial till, approximately 5 m above the bedrock surface, the following equation applies:

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

$$V_{s30} = -\frac{30m}{\sum\left(\frac{5m}{414m/s} + \frac{25m}{2{,}525m/s}\right)}$$

$$V_{s30} = 1{,}364m/s$$

Vs30 - Two Underground Parking Levels

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012. If the building is founded on glacial till, approximately 2 to 3 m above the bedrock surface, the following equation applies:

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

$$V_{s30} = -\frac{30m}{\sum\left(\frac{3m}{414m/s} + \frac{27m}{2{,}525m/s}\right)}$$

$$V_{s30} = 1{,}672m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for foundations placed on glacial till, and 5 m above the bedrock surface, is 1,364 m/s. Therefore, a **Site Class C** is applicable for design of the proposed building. Alternatively, if the footings are extended to the bedrock surface or within 3 m of the bedrock surface, the Vs_{30} would be 1,672 m/s, and a **Site Class A** would apply.

5.5 Basement Floor Slab

The native soil or approved fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone for the basement floor slab used for finished space. In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

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, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) 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 when using the effective unit weight.

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. The 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 maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted 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

Car only parking and heavy truck parking areas, and access lanes may be required at this site. The proposed pavement structures are presented in Tables 5 and 6.

Table 5 - 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 6 - 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
450	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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for all the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer. A waterproofing system should be provided to the elevator pit (pit bottom and walls).

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. A composite drainage system should be applied to the exterior of the building foundation walls in order to minimize the risk of groundwater infiltration from the backfill materials.

Alternatively, where foundation walls are to be formed against a temporary shoring system, the following is recommended. A composite drainage system should be fastened to the shoring face to allow for a blind sided foundation wall pour. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. An interior perimeter drainage consisting of a minimum 150 mm diameter perforated, corrugated PVC pipe be placed along the interior side of the exterior footing. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. 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, pile caps and grade beams of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m 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.

Other exterior unheated footings, pile caps or grade beams, 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

At this site, temporary shoring may be required to complete the required excavations. However, it is recommended that where sufficient room is available open cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

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.

Temporary Shoring

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation 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.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 7 - 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 γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ 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.

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 pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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.

Permit to Take Water

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

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of 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 MOECC review of the PTTW application.

Impacts on Neighbouring Properties

Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

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.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. 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 results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should 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 geotechnical 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 Holloway Lodging Corporation or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Nathan F. S. Christie, P.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept. 18-2017
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Report Distribution

- ☐ Holloway Lodging Corporation (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of the intersection of Archibald St. & Carling Avenue. Geodetic elevation = 75.14m, as

REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY CME 55 Power Auger

DATE August 15, 2017

FILE NO. PG3736

HOLE NO. BH 1-17

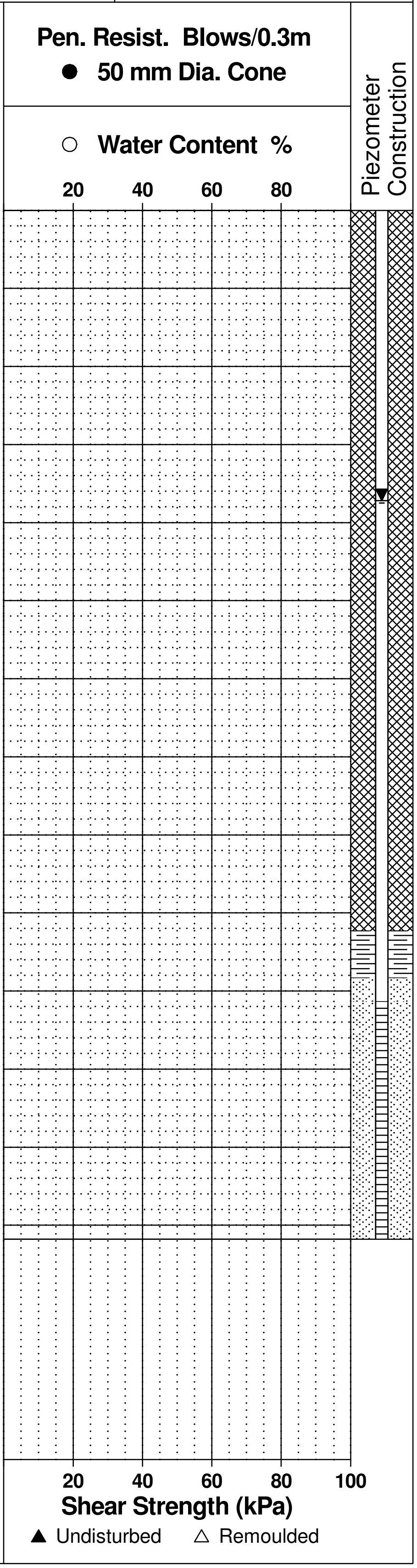
SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.09
FILL: Crushed stone with silt and sand	0.33
FILL: Brown silty sand, some clay, trace organics	1.65
Compact, brown **SILTY SAND**	1.98
Brown to grey **SILTY CLAY,** trace sand	3.81
GLACIAL TILL: Grey silty clay, some sand, trace gravel, possible cobbles and boulders	9.22
BEDROCK: Grey limestone interbedded with shale	13.18

End of Borehole

(GWL @ 3.72m - August 24, 2017)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	12
SS	3	100	2
SS	4	33	P
SS	5	54	9
RC	1	33	
RC	2	71	34
RC	3	100	67
RC	4	100	93

DEPTH (m)	ELEV. (m)
0	75.22
1	74.22
2	73.22
3	72.22
4	71.22
5	70.22
6	69.22
7	68.22
8	67.22
9	66.22
10	65.22
11	64.22
12	63.22
13	62.22



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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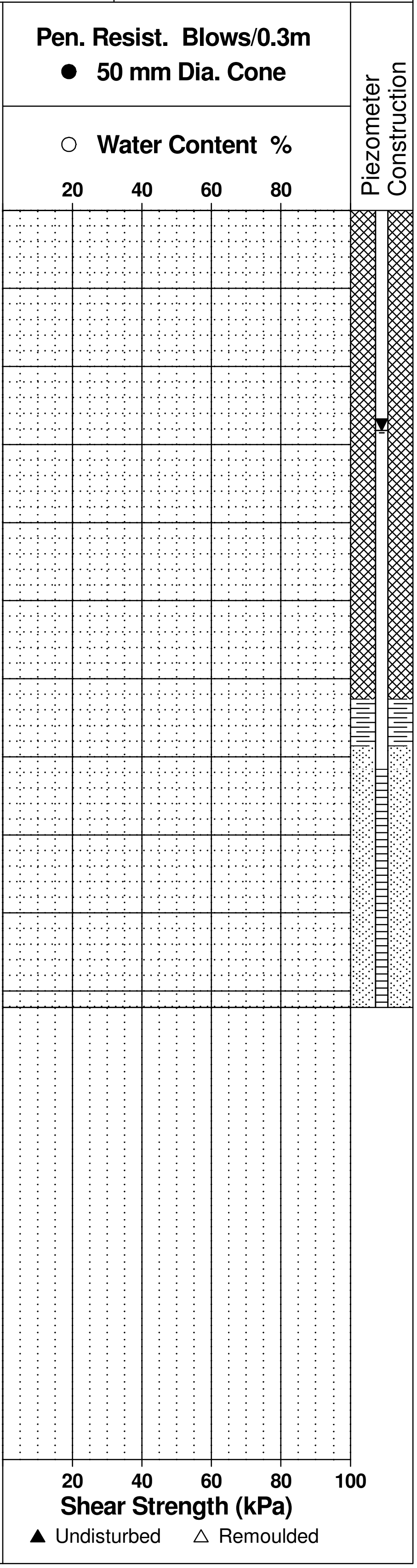
BORINGS BY CME 55 Power Auger

DATE August 15, 2017

FILE NO. **PG3736**

HOLE NO. **BH 2-17**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.65
Asphaltic concrete 0.06							
FILL: Crushed stone with silt and sand 0.20 1.07		AU	1				
						1	73.65
FILL: Brown silty sand, trace gravel, cobbles and boulders							
Brown **SILTY CLAY,** trace sand 2.29		SS	2	50	4	2	72.65
						3	71.65
GLACIAL TILL: Grey silty clay, trace gravel, cobbles, possible boulders		SS	3	54	9		
						4	70.65
		SS	4	46	11		
						5	69.65
6.20						6	68.65
BEDROCK: Grey limestone interbedded with shale		RC	1	89	37		
						7	67.65
		RC	2	100	77	8	66.65
						9	65.65
		RC	3	100	90		
10.21						10	64.65
End of Borehole							
(GWL @ 2.82m - August 24, 2017)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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BORINGS BY CME 55 Power Auger

DATE August 15, 2017

FILE NO. PG3736

HOLE NO. BH 3-17

SOIL DESCRIPTION

GROUND SURFACE

Asphaltic concrete 0.08
FILL: Crushed stone with silt and 0.25
sand 0.53
FILL: Brown silty sand 0.59
Concrete 0.91
FILL: Brown silty clay, trace sand, 1.07
gravel and cobbles 1.22
FILL: Brown silty sand
FILL: Brown silty clay, trace sand
and gravel
Stiff to firm, brown **SILTY CLAY,**
trace sand 4.11

- grey by 3.0m depth
GLACIAL TILL: Grey silty clay,
some sand, trace gravel, cobbles, 5.60
possible boulders

GLACIAL TILL: Grey silty sand,
some gravel, cobbles, trace boulders
9.14

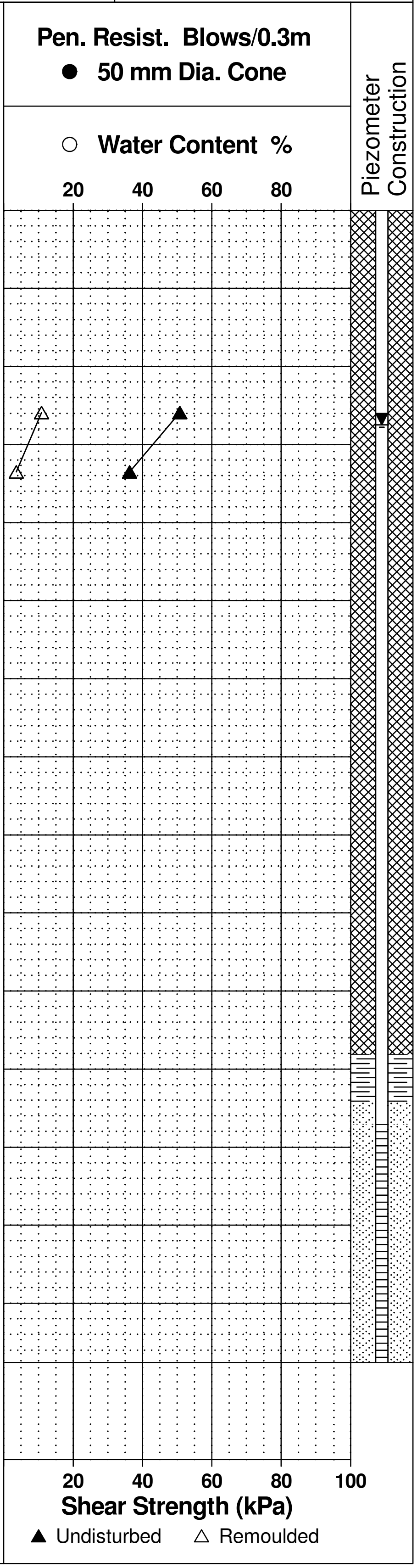
BEDROCK: Grey limestone
interbedded with shale
14.76

End of Borehole

(GWL @ 2.75m - August 24, 2017)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	46	6
SS	4	62	P
SS	5	100	P
SS	6	58	13
SS	7	50	32
SS	8	0	28
RC	1	100	100
RC	2	70	62
RC	3	81	59
RC	4	100	61

DEPTH (m)	ELEV. (m)
0	74.21
1	73.21
2	72.21
3	71.21
4	70.21
5	69.21
6	68.21
7	67.21
8	66.21
9	65.21
10	64.21
11	63.21
12	62.21
13	61.21
14	60.21



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SOIL PROFILE AND TEST DATA

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Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

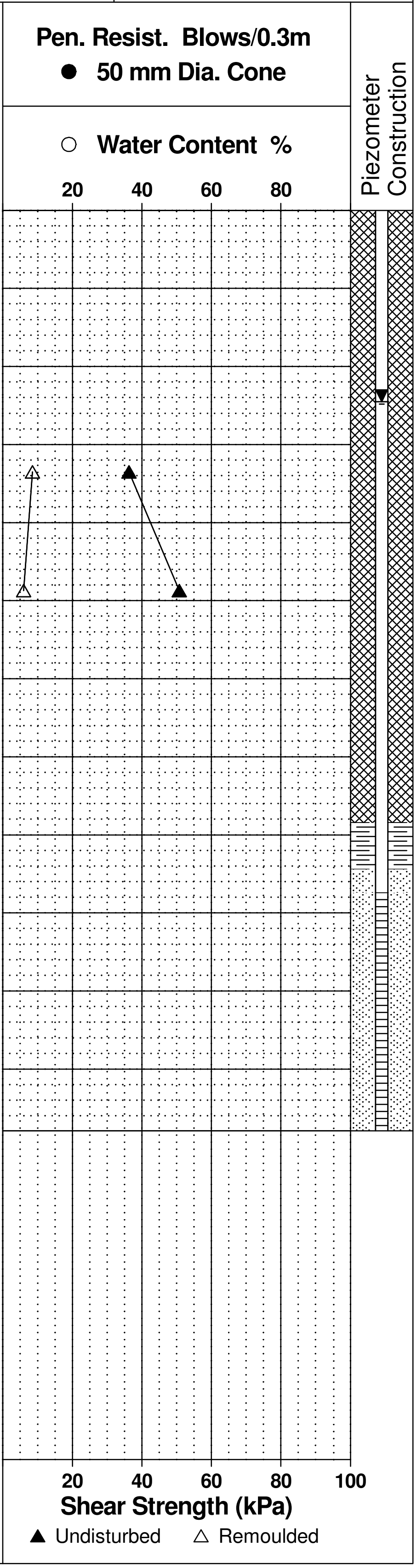
BORINGS BY CME 55 Power Auger

DATE August 16, 2017

FILE NO. PG3736

HOLE NO. BH 4-17

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.06	AU	1			0	74.36
FILL: Crushed stone with silt sand 0.18	AU	2				
sand 0.76						
FILL: Brown silty sand, trace gravel 1.07 and cobbles	SS	3	33	12	1	73.36
FILL: Brown silty clay, trace sand and organics	SS	4	83	9	2	72.36
Firm to stiff, brown **SILTY CLAY,** trace sand	SS	5	4	4		
					3	71.36
- grey by 3.0m depth	SS	6	50	P		
					4	70.36
	SS	7	100	P	5	69.36
5.64						
					6	68.36
GLACIAL TILL: Grey silty clay, some sand, gravel, trace cobbles and boulders	SS	8	86	59		
					7	67.36
7.49						
	RC	1	82	36	8	66.36
					9	65.36
BEDROCK: Grey limestone interbedded with shale	RC	2	100	86		
					10	64.36
	RC	3	98	67	11	63.36
11.79						
End of Borehole						
(GWL @ 2.45m - August 24, 2017)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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FILE NO. PG3736

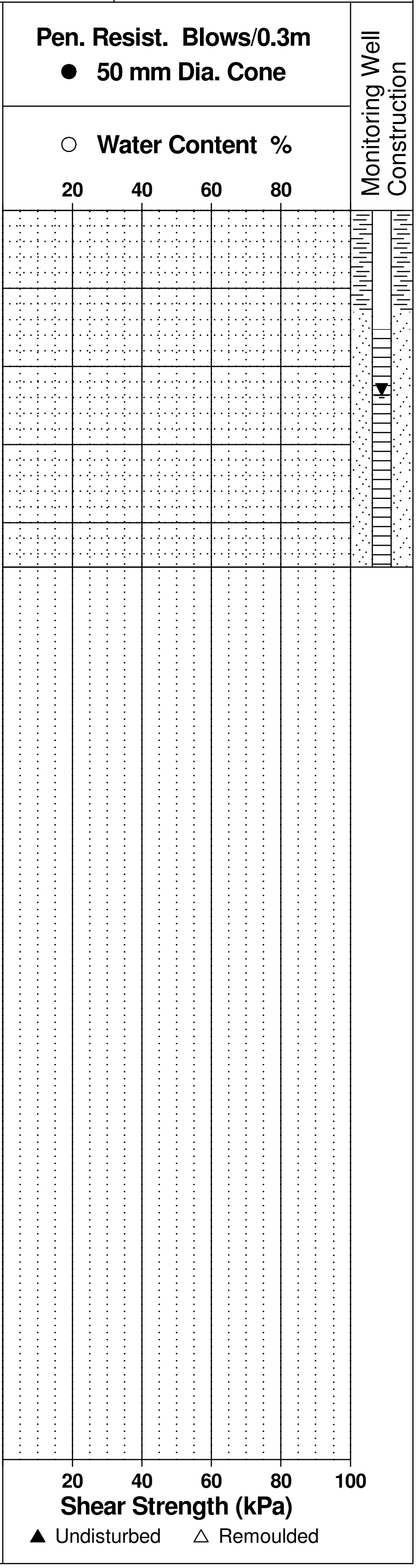
HOLE NO. BH 5-17

BORINGS BY CME 55 Power Auger

DATE August 16, 2017

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06	AU	1		
FILL: Crushed stone with silt and sand	0.30	AU	2		
FILL: Brown silty sand, trace clay and construction debris	2.44	AU AU	3 4		
Grey **SILTY CLAY,** trace sand	4.57	AU AU	5 6		
End of Borehole (GWL @ 2.37m - August 24, 2017)					

DEPTH (m)	ELEV. (m)
0	74.09
1	73.09
2	72.09
3	71.09
4	70.09



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BORINGS BY CME 55 Power Auger

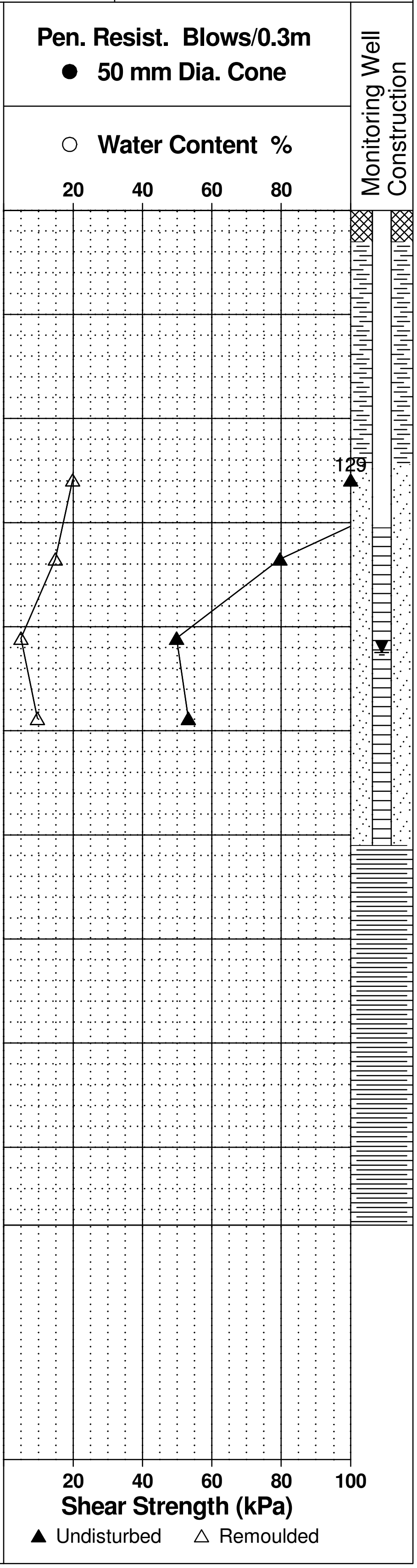
DATE October 27, 2016

FILE NO. PG3736

HOLE NO. BH 1

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Topsoil and organics	0.10	AU	1		
FILL: Brown silty fine sand with gravel, topsoil, trace clay - asphalt by 1.8m depth		SS	2	33	6
	2.20	SS	3	21	13
FILL: Brown silty clay with sand, some gravel, trace asphalt	2.80	SS	4	33	5
Very stiff to stiff, brown **SILTY CLAY** - stiff to firm and grey by 3.7m depth		SS	5	75	3
- trace sand and gravel by 4.3m depth		SS	6	83	1
	5.26	SS	7	75	2
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, gravel, trace cobbles		SS	8	50	4
		SS	9	67	11
	7.16	SS	10	67	23
GLACIAL TILL: Compact, grey sandy silt with clay, gravel, cobbles, trace boulders		SS	11	58	30
		SS	12	58	19
	9.75	SS	13	75	31
End of Borehole (GWL @ 4.25m-Nov. 7, 2016)					

DEPTH (m)	ELEV. (m)
0	75.28
1	74.28
2	73.28
3	72.28
4	71.28
5	70.28
6	69.28
7	68.28
8	67.28
9	66.28



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REMARKS

BORINGS BY CME 55 Power Auger

DATE October 27, 2016

FILE NO. PG3736

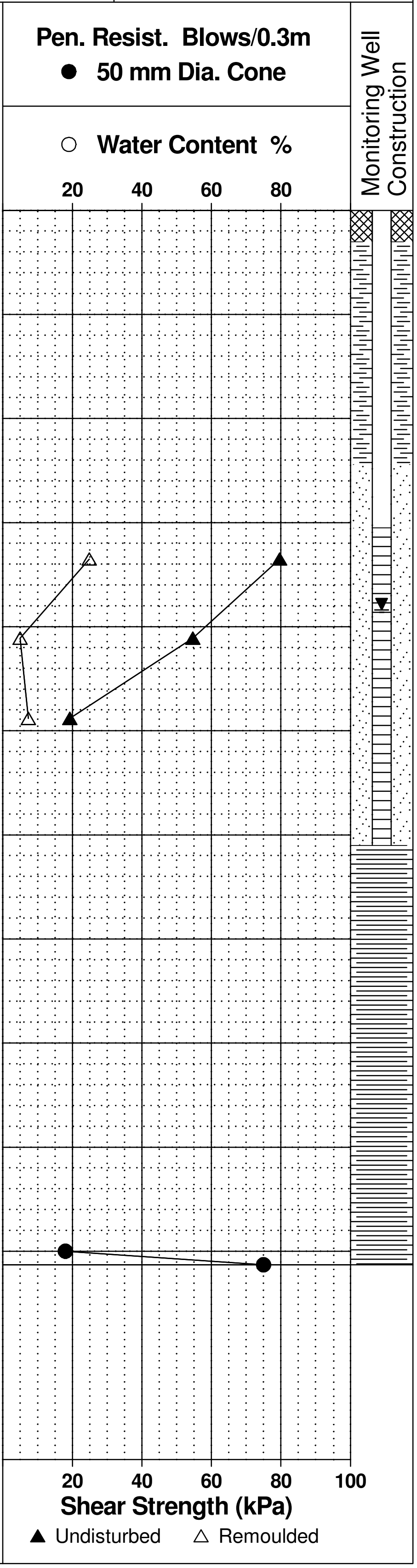
HOLE NO. BH 2

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty fine sand with gravel, trace clay and asphalt	0.60	AU	1		
FILL: Brown silty clay, trace sand, gravel and topsoil		SS	2	29	8
		SS	3	42	6
	2.59	SS	4	67	7
Very stiff to stiff, brown **SILTY CLAY**		SS	5	25	4
- stiff to firm and grey by 4.0m depth		SS	6	79	3
	4.98	SS	7	100	2
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand and gravel		SS	8	58	5
		SS	9	58	6
	7.32	SS	10	67	5
GLACIAL TILL: Compact, grey sandy silt with clay, gravel, cobbles, trace boulders		SS	11	42	17
		SS	12	33	18
	9.75	SS	13	33	32
Dynamic Cone Penetration Test commenced at 9.75m depth.	10.13				
End of Borehole					

Practical DCPT refusal at 10.13m depth

(GWL @ 3.84m-Nov. 7, 2016)

DEPTH (m)	ELEV. (m)
0	75.13
1	74.13
2	73.13
3	72.13
4	71.13
5	70.13
6	69.13
7	68.13
8	67.13
9	66.13
10	65.13



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REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY CME 55 Power Auger

DATE October 28, 2016

FILE NO. PG3736

HOLE NO. BH 3

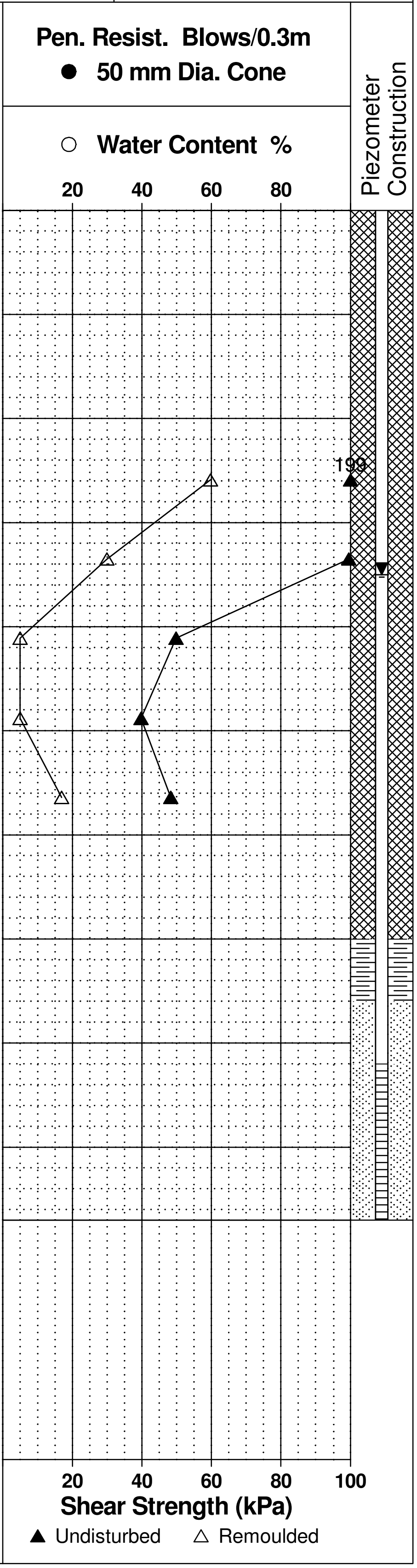
SOIL DESCRIPTION GROUND SURFACE	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
FILL: 50mm Asphaltic concrete over red-brown silty fine sand with crushed stone and gravel, trace brick	AU	1		
1.22	SS	2	33	7
FILL: Brown silty clay, trace sand and gravel - some sand, brick and topsoil by 1.5m depth 2.13	SS	3	75	9
Very stiff to stiff, brown **SILTY CLAY** - firm by 3.0m depth - grey by 3.8m depth	SS	4	67	5
	SS	5	83	2
	SS	6	83	2
5.49	SS	7	92	2
GLACIAL TILL: Stiff, grey silty clay to clayey silt, some sand, trace gravel 5.70	SS	8	83	4
GLACIAL TILL: Loose, grey sandy silt with clay, gravel, trace cobbles and boulders 6.80	SS	9	92	10
GLACIAL TILL: Compact to dense, grey silty fine sand with gravel, some cobbles and boulders - some running sand from 6.9 to 8.1m depth	SS	10	73	50+
	SS	11	83	40
	SS	12	56	50+
9.70	SS	13	55	57

DEPTH (m)	ELEV. (m)
0	74.85
1	73.85
2	72.85
3	71.85
4	70.85
5	69.85
6	68.85
7	67.85
8	66.85
9	65.85

End of Borehole

Practical refusal to augering at 9.70m depth

(GWL @ 3.50m-Nov. 7, 2016)



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REMARKS

BORINGS BY CME 55 Power Auger

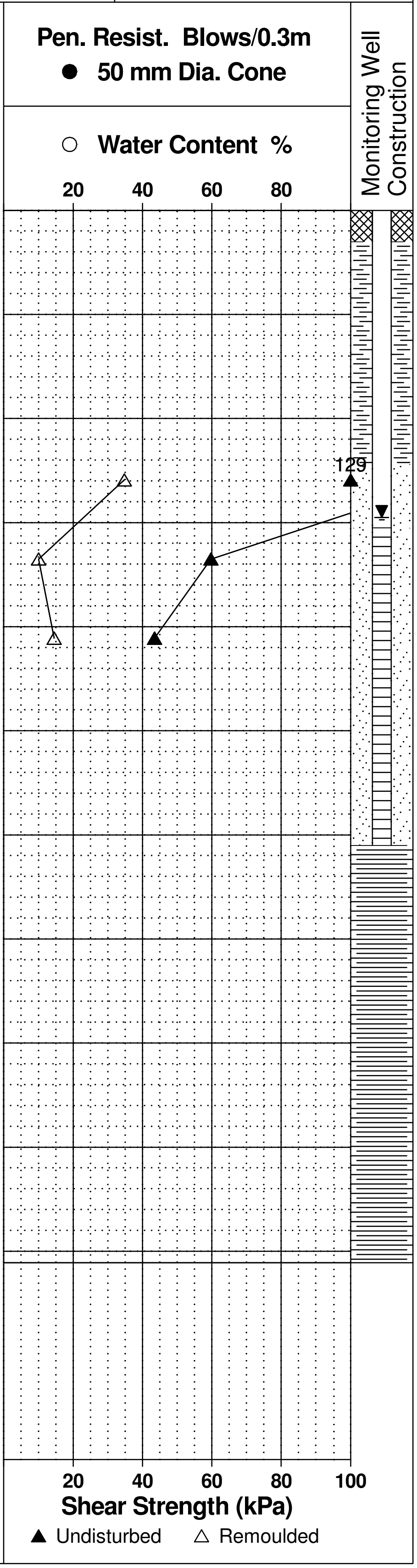
DATE October 28, 2016

FILE NO. PG3736

HOLE NO. BH 4

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: 50mm Asphaltic concrete over brown silty fine sand with crushed stone and gravel 0.76	AU	1		
FILL: Red-brown sand, some asphalt 1.12	SS	2	67	3
FILL: Brown silty clay, some sand, topsoil, trace gravel 1.62	SS	3	67	10
Very stiff to stiff, brown **SILTY CLAY**	SS	4	75	3
- firm and grey by 3.8m depth	SS	5	33	2
- trace sand by 4.3m depth 4.57	SS	6	83	1
GLACIAL TILL: Stiff, grey silty clay to clayey silt, some sand, trace gravel	SS	7	33	8
6.20	SS	8	67	21
GLACIAL TILL: Compact, grey silty fine to medium sand with gravel, trace cobbles	SS	9	46	24
	SS	10	100	43
- some running sand from 6.2 to 9.75m depth	SS	11	75	33
	SS	12	83	30
9.75	SS	13		
Dynamic Cone Penetration Test commenced at 9.75m depth. 10.11				
End of Borehole				
Practical DCPT refusal at 10.11m depth				
(GWL @ 2.95m-Nov. 7, 2016)				

DEPTH (m)	ELEV. (m)
0	74.48
1	73.48
2	72.48
3	71.48
4	70.48
5	69.48
6	68.48
7	67.48
8	66.48
9	65.48
10	64.48



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of the intersection of Archibald St. & Carling Avenue. Geodetic elevation = 75.14m, as
REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY CME 55 Power Auger **DATE** October 31, 2016

FILE NO. PG3736

HOLE NO. BH 5

SOIL DESCRIPTION

GROUND SURFACE

FILL: 50mm Asphaltic concrete over red-brown silty fine sand with crushed stone and gravel 0.60

FILL: Brown silty sand with clay, gravel, some topsoil, trace wood 1.68

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

- firm and grey by 3.0m depth

5.03

GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, some gravel, trace cobbles 6.25

GLACIAL TILL: Compact, grey sandy silt with gravel, trace cobbles and boulders

- some running sand from 6.25 to 6.7m depth

7.77

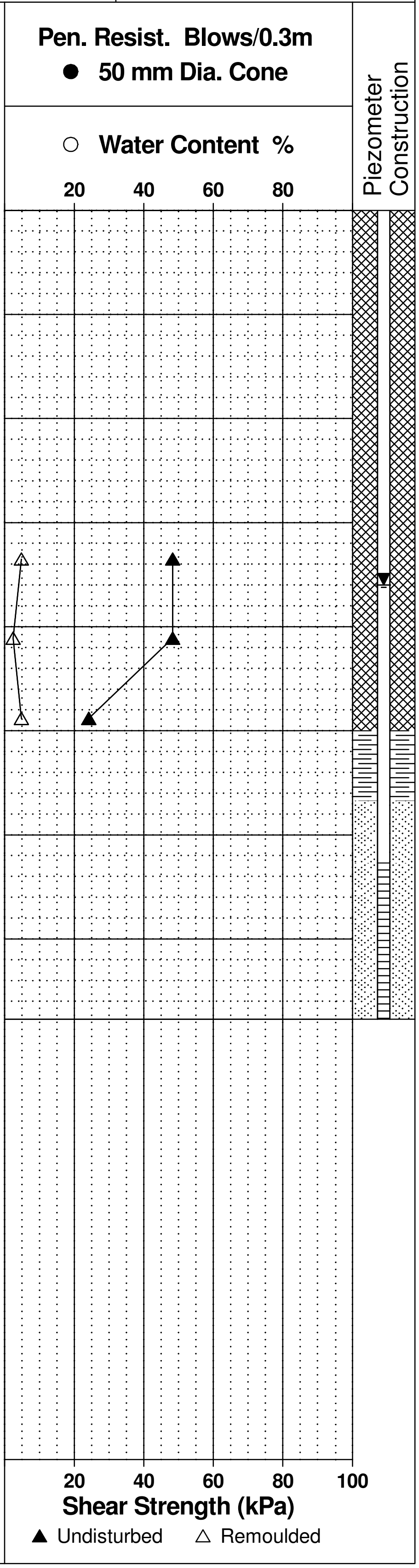
End of Borehole

Practical refusal to augering at 7.77m depth

(GWL @ 3.60m-Nov. 7, 2016)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	7
SS	3	58	7
SS	4	67	2
SS	5	100	2
SS	6	100	1
SS	7	50	6
SS	8	33	37
SS	9	54	40
SS	10	100	50+

DEPTH (m)	ELEV. (m)
0	74.22
1	73.22
2	72.22
3	71.22
4	70.22
5	69.22
6	68.22
7	67.22



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of the intersection of Archibald St. & Carling Avenue. Geodetic elevation = 75.14m, as per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 26, 2016

FILE NO. PG3736

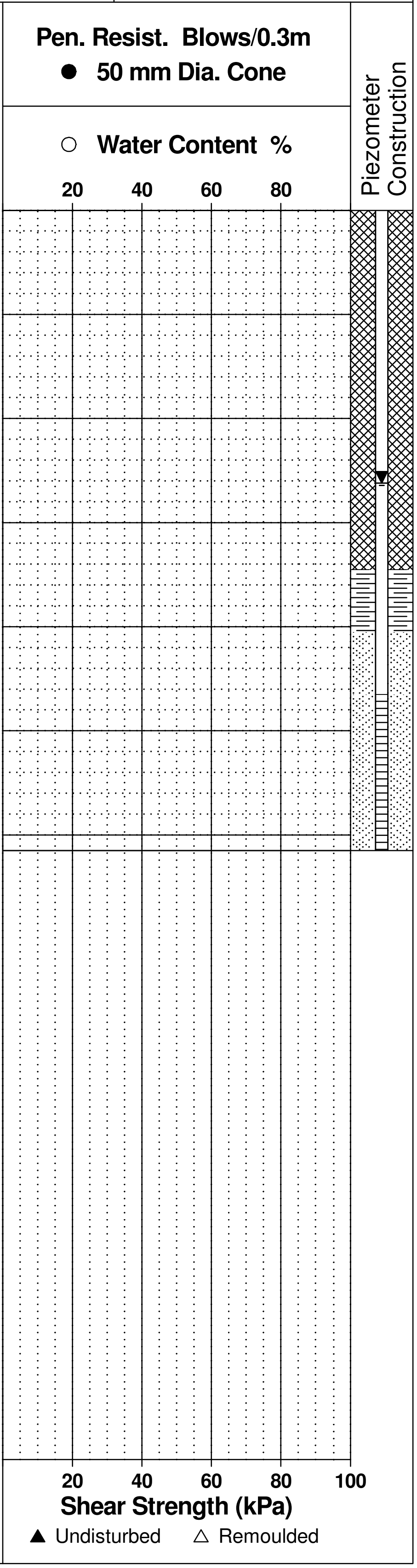
HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.12
FILL: 50mm Asphaltic concrete over brow silty fine to medium sand with crushed stone and gravel	AU	1				
1.07	SS	2	22	50+	1	73.12
FILL: Brown silty clay with sand, gravel and cobbles	SS	3	67	8		
2.21					2	72.12
GLACIAL TILL: Stiff, brown silty clay with sand, gravel, trace cobbles	SS	4	21	4		
	SS	5	29	5	3	71.12
- grey by 3.7m depth	SS	6	83	7	4	70.12
4.72	SS	7	71	28	5	69.12
GLACIAL TILL: Compact to dense, grey sandy silt to silty sand with gravel, cobbles, trace boulders	SS	8	76	50+		
6.15	SS	9	50	50+	6	68.12

End of Borehole

Practical refusal to augering at 6.15m depth

(GWL @ 2.62m-Nov. 7, 2016)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE October 26, 2016

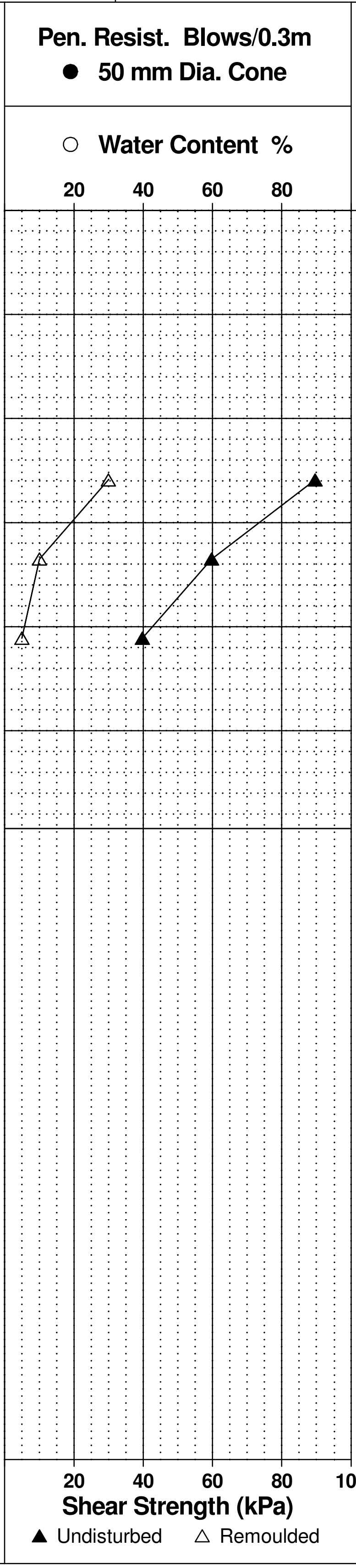
FILE NO. PG3736

HOLE NO. BH 7

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: 50mm Asphaltic concrete over brown silty fine sand with crushed stone, gravel, trace cobbles	0.69
FILL: Brown to green silty clay, trace topsoil, sand and gravel - some topsoil by 1.5m depth	1.83
TOPSOIL	2.13
Stiff, brown **SILTY CLAY** - firm and grey by 3.35m depth	4.11
GLACIAL TILL: Firm to stiff, grey silty clay to clayey silt with gravel	5.18
GLACIAL TILL: Compact, sandy silt to silty fine sand with gravel, cobbles, trace boulders	5.94
End of Borehole Practical refusal to augering at 5.94m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	67	8
SS	3	67	4
SS	4	42	2
SS	5	100	W
SS	6	100	W
SS	7	71	6
SS	8	50	23

DEPTH (m)	ELEV. (m)
0	74.71
1	73.71
2	72.71
3	71.71
4	70.71
5	69.71



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY CME 55 Power Auger

DATE October 26, 2016

FILE NO. PG3736

HOLE NO. BH 8

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: 50mm Asphaltic concrete over brown silty fine sand, some crushed stone and gravel — 0.76	AU	1		
FILL: Brown silty clay with sand, gravel, trace wood, cobbles and topsoil — 1.83	SS	2	50	11
	SS	3	54	9
Very stiff to stiff, brown **SILTY CLAY**	SS	4	54	5
- stiff to firm and grey by 3.2m depth	SS	5	67	3
— 4.70	SS	6	92	2
GLACIAL TILL: Stiff, grey silty clay to clayey silt, some gravel	SS	7	42	2
	SS	8	17	6
— 6.78	SS	9	17	6
GLACIAL TILL: Compact to dense, grey sandy silt to silty fine sand with gravel, cobbles, trace boulders — 7.32	SS	10	35	50+

End of Borehole

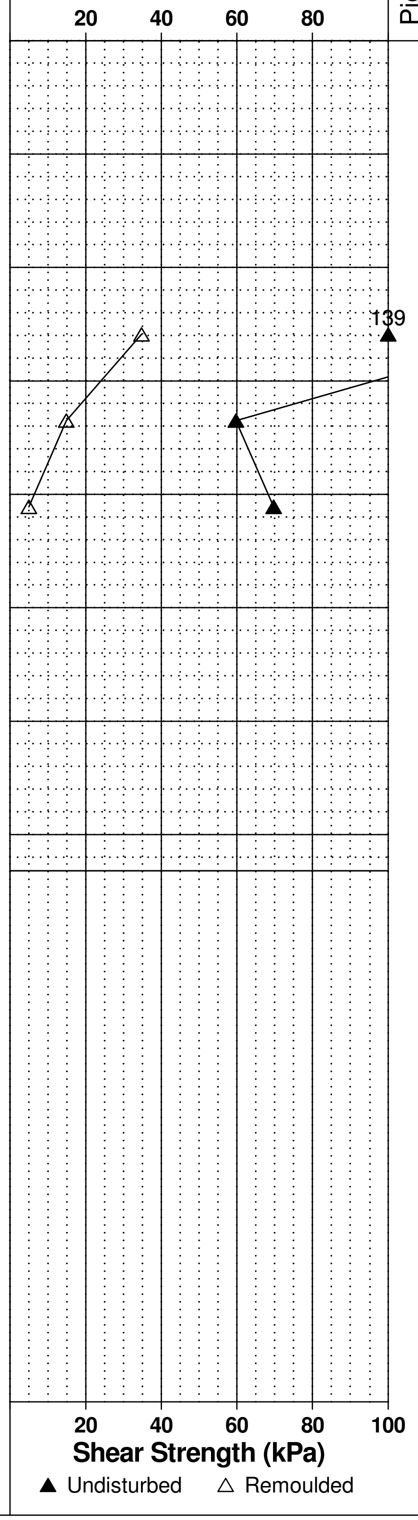
Practical refusal to augering at 7.32m depth

DEPTH (m)	ELEV. (m)
0	74.62
1	73.62
2	72.62
3	71.62
4	70.62
5	69.62
6	68.62
7	67.62

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

○ Water Content %

Piezometer Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY CME 55 Power Auger

DATE October 27, 2016

FILE NO. PG3736

HOLE NO. BH 9

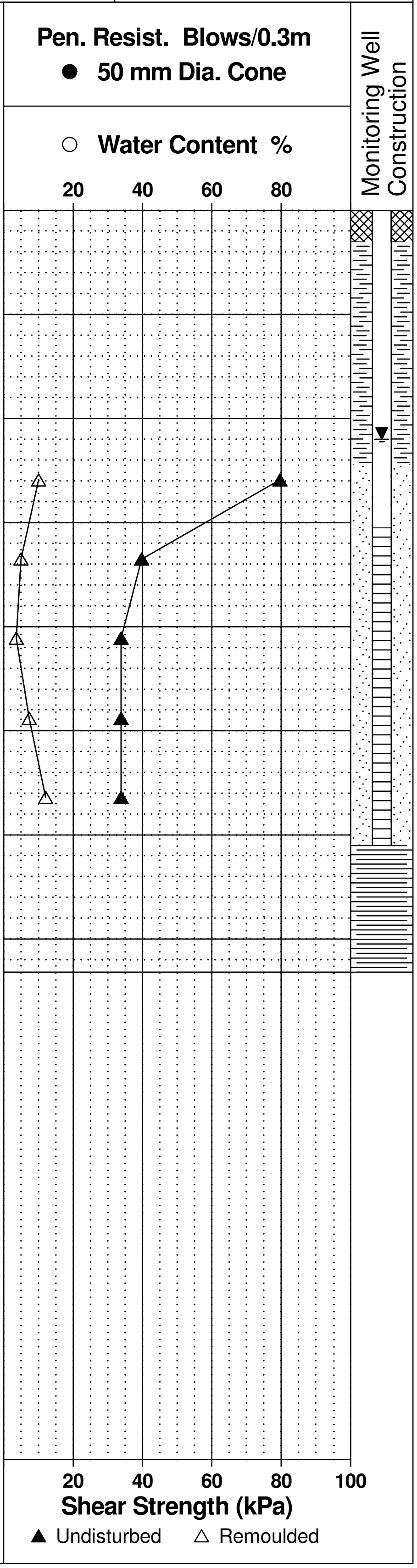
SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: 50mm Asphaltic concrete over brown silty fine sand with crushed stone and gravel 0.60	AU	1		
FILL: Brown silty clay with sand, trace gravel and asphalt 0.97	SS	2	42	9
Very stiff to stiff, brown **SILTY CLAY** - firm and grey by 3.0m depth	SS	3	67	9
	SS	4	88	3
	SS	5	100	W
	SS	6	100	W
	SS	7	100	2
6.10	SS	8	67	
Loose to compact, grey fine to medium **SAND,** some gravel	SS	9	58	4
7.32	SS	10	83	36

DEPTH (m)	ELEV. (m)
0	74.28
1	73.28
2	72.28
3	71.28
4	70.28
5	69.28
6	68.28
7	67.28

End of Borehole

Practical refusal to augering at 7.32m depth

(GWL @ 2.20m-Nov. 7, 2016)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

DATUM / **REMARKS** TBM - Top spindle of fire hydrant located at the southeast corner of the intersection of Archibald St. & Carling Avenue. Geodetic elevation = 75.14m, as per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG3736

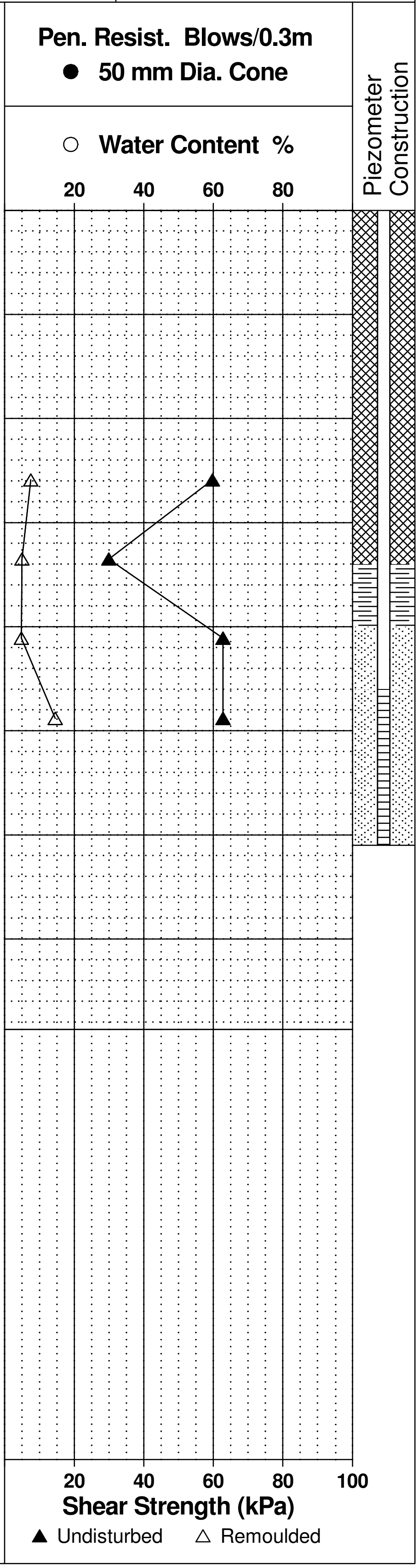
HOLE NO. BH10

BORINGS BY CME 55 Power Auger

DATE October 28, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: 50mm Asphaltic concrete over crushed stone with silty sand and clay — 0.90	AU	1		
FILL: Brown silty sand with clay, some brick	SS	2	50	6
— 2.18	SS	3	42	5
Very stiff to stiff, brown **SILTY CLAY** - firm to stiff and grey by 3.0m depth	SS	4	58	1
	SS	5	92	W
	SS	6	92	1
— 5.08	SS	7	67	24
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, gravel, trace cobbles and boulders — 5.26	SS	8	42	12
GLACIAL TILL: Compact, grey sandy silt with clay, gravel, cobbles and boulders — 6.02	SS	9	58	12
GLACIAL TILL: Compact, grey silty fine to medium sand with gravel, cobbles, trace clay and boulders	SS	10	58	20
- some running sand — 7.87	SS	11	80	50+
End of Borehole Practical refusal to augering at 7.87m depth				

DEPTH (m)	ELEV. (m)
0	74.11
1	73.11
2	72.11
3	71.11
4	70.11
5	69.11
6	68.11
7	67.11



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SOIL PROFILE AND TEST DATA

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Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS

BORINGS BY Portable Drill

DATE October 31, 2016

FILE NO. PG3736

HOLE NO. BH11

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)	Monitoring Well Construction
GROUND SURFACE						0	72.67		
FILL: 50mm Asphaltic concrete over crushed stone		AU	1						
0.76						1	71.67		▼
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, gravel, cobbles, trace boulders		SS	2						
		SS	3			2	70.67		
		SS	4			3	69.67		
		SS	5						
		SS	6			4	68.67		
4.27									
End of Borehole (GWL @ 1.31m-Nov. 7, 2016)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of the intersection of Archibald St. & Carling Avenue. Geodetic elevation = 75.14m, as per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

FILE NO. PG3736

HOLE NO. BH12

BORINGS BY Portable Drill

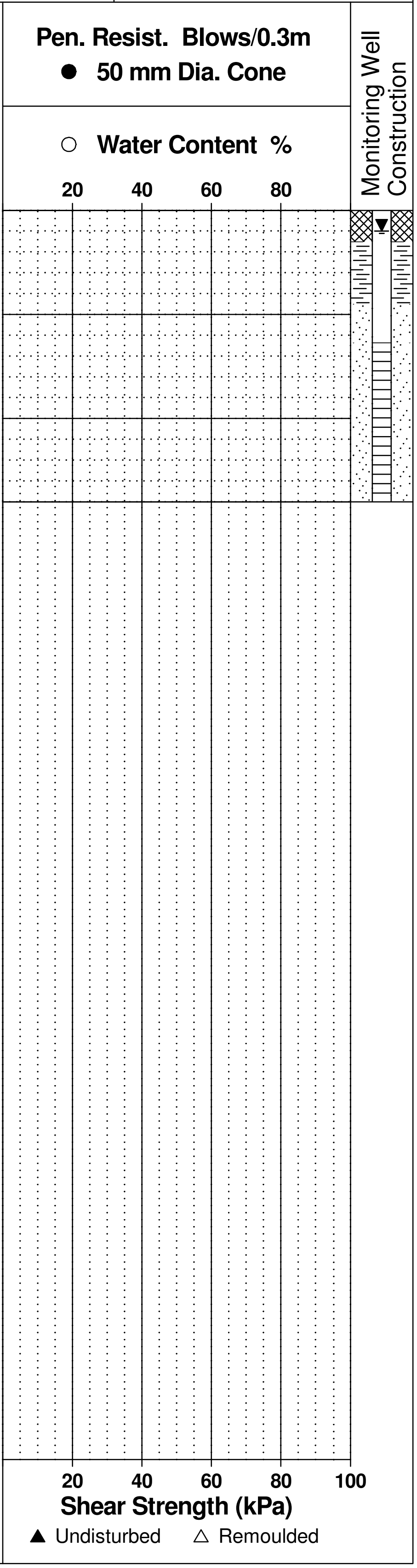
DATE November 1, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.49
Concrete slab 0.10						
FILL: Gravel, cobbles with sandy silt and clay 0.60	AU	1				
0.97						
Grey **SILTY CLAY**	SS	2	100		1	70.49
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, trace gravel and cobbles	SS	3	7			
	SS	4	83		2	69.49
2.80	SS	5				

End of Borehole

Practical split spoon refusal at 2.80m depth

(GWL @ 0.20m-Nov. 7, 2016)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Multi-Storey Redevelopment - 1354-1376 Carling Ave.
Ottawa, Ontario

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REMARKS per plan prepared by Annis, O'Sullivan, Vollebekk Ltd.

BORINGS BY Portable Drill

DATE November 1, 2016

FILE NO. PG3736

HOLE NO. BH13

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	71.03		
Concrete slab — 0.10									
FILL: Crushed stone — 0.60		AU	1						
GLACIAL TILL: Stiff, grey silty clay to clayey silt with sand, trace gravel and cobbles		SS	2	50		1	70.03		
		SS	3	58					
		SS	4	100		2	69.03		
2.74		SS	5	100					
End of Borehole Practical split spoon refusal at 2.74m depth									

20 40 60 80

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)

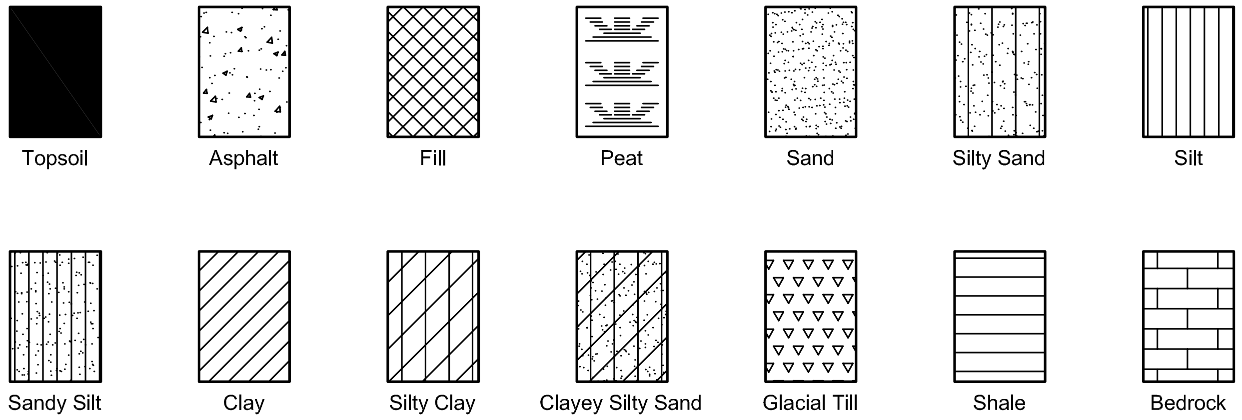
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

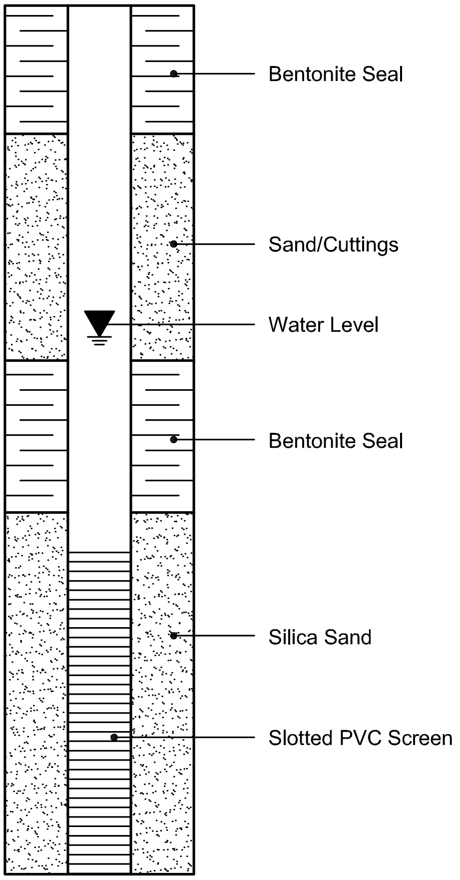
SYMBOLS AND TERMS (continued)

STRATA PLOT

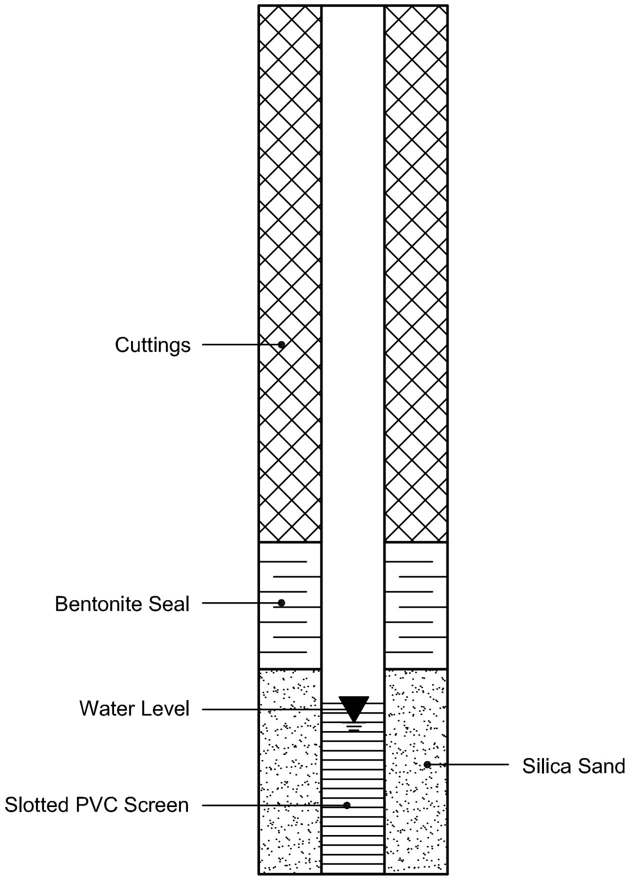


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21322

Report Date: 30-Nov-2016

Order Date: 25-Nov-2016

Project Description: PG3736

	Client ID:	BH3-SS4	-	-	-
	Sample Date:	28-Oct-16	-	-	-
	Sample ID:	1648457-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.61	-	-	-
Resistivity	0.10 Ohm.m	8.99	-	-	-
Anions					
Chloride	5 ug/g dry	700	-	-	-
Sulphate	5 ug/g dry	423 [1]	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG3736-1 - TEST HOLE LOCATION PLAN

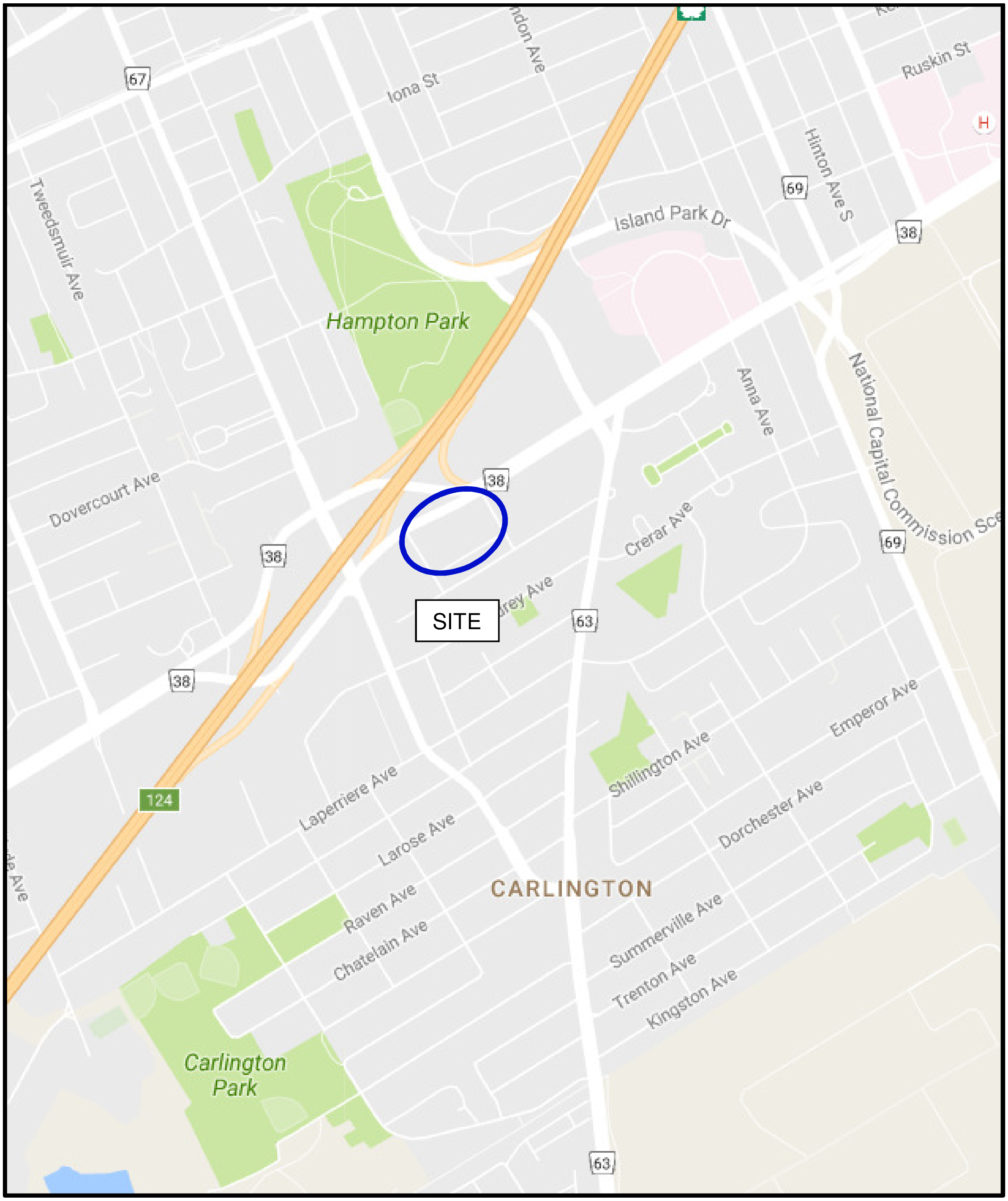


FIGURE 1

KEY PLAN

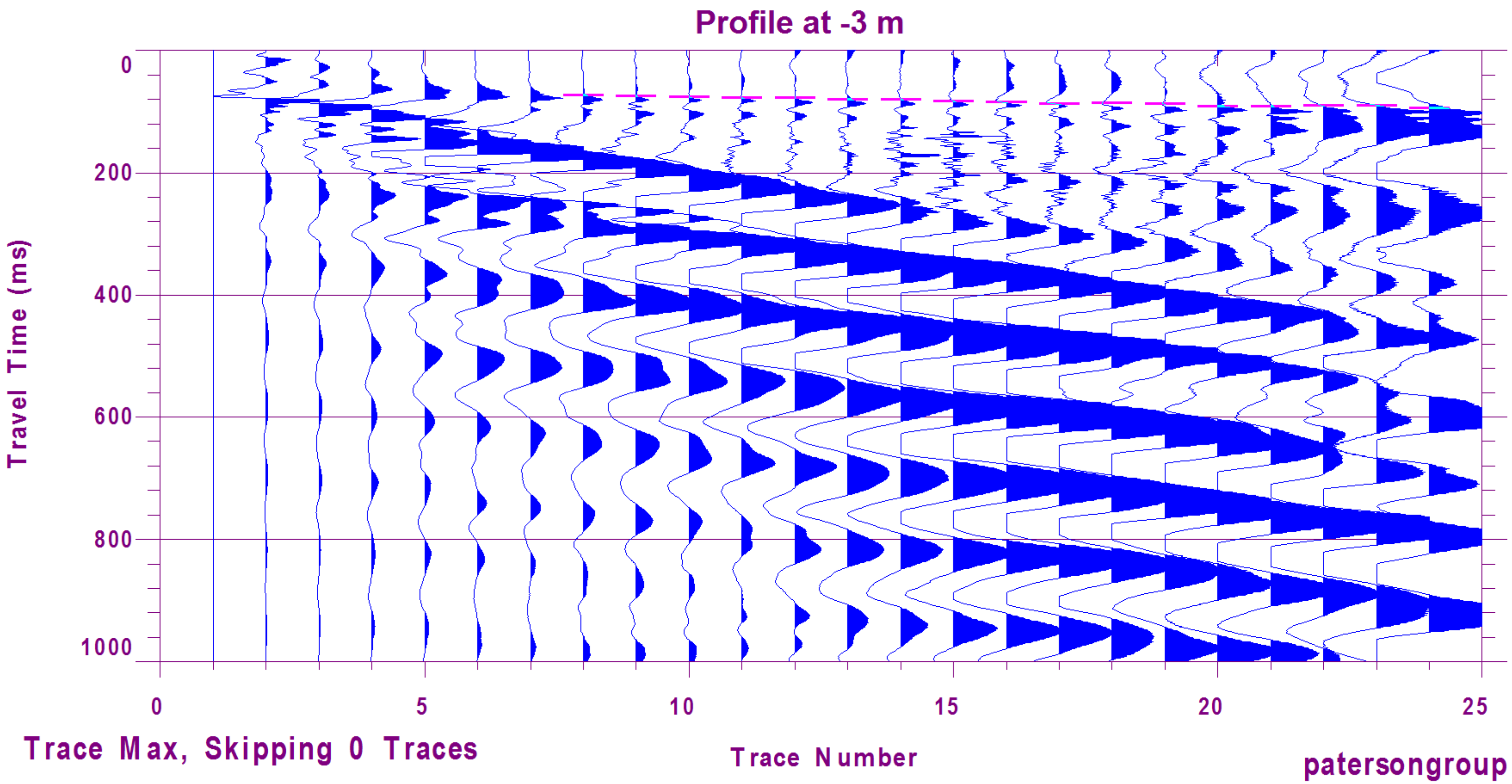


Figure 2 – Shear Wave Velocity Profile at Shot Location -3 m

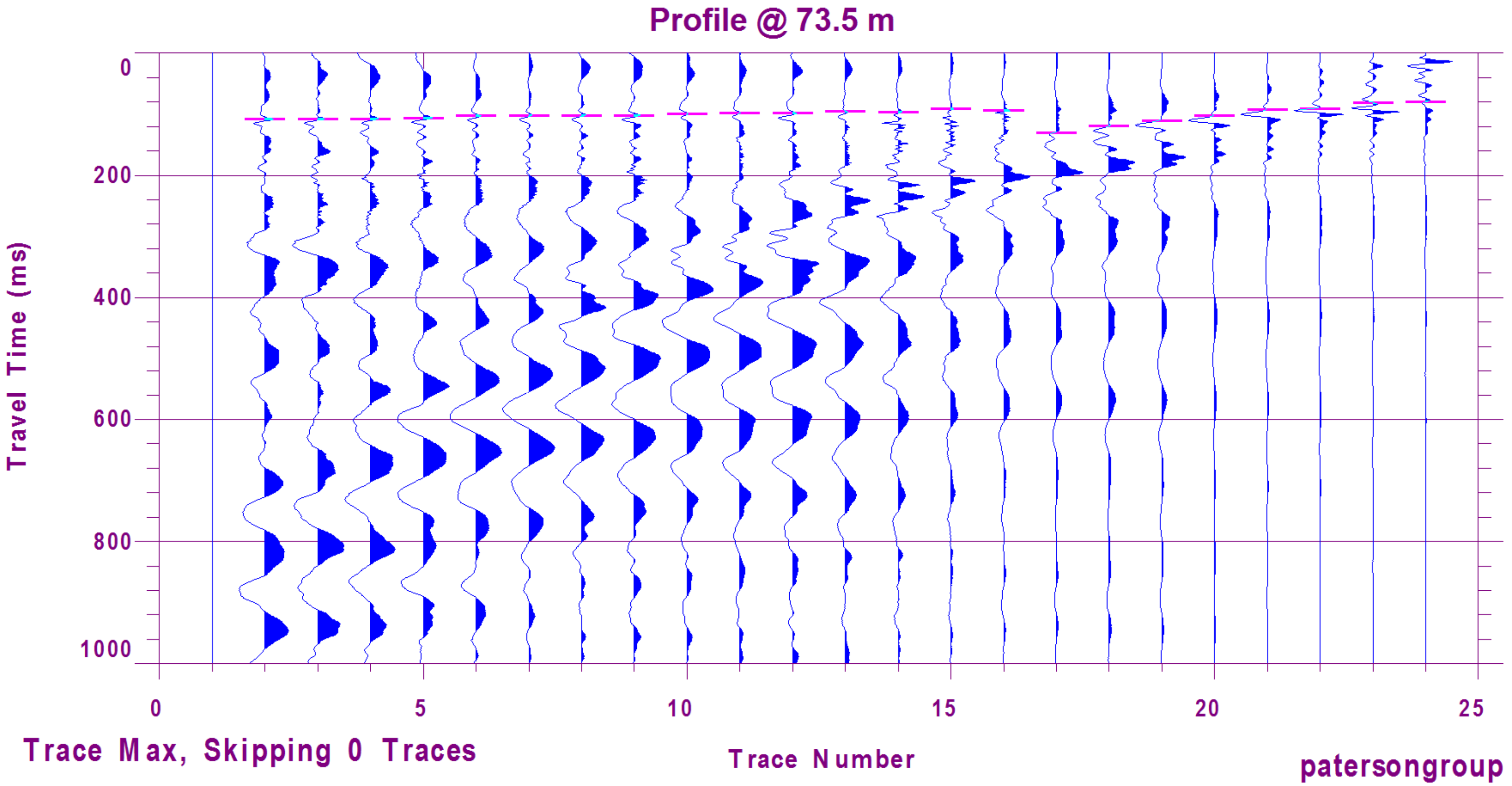
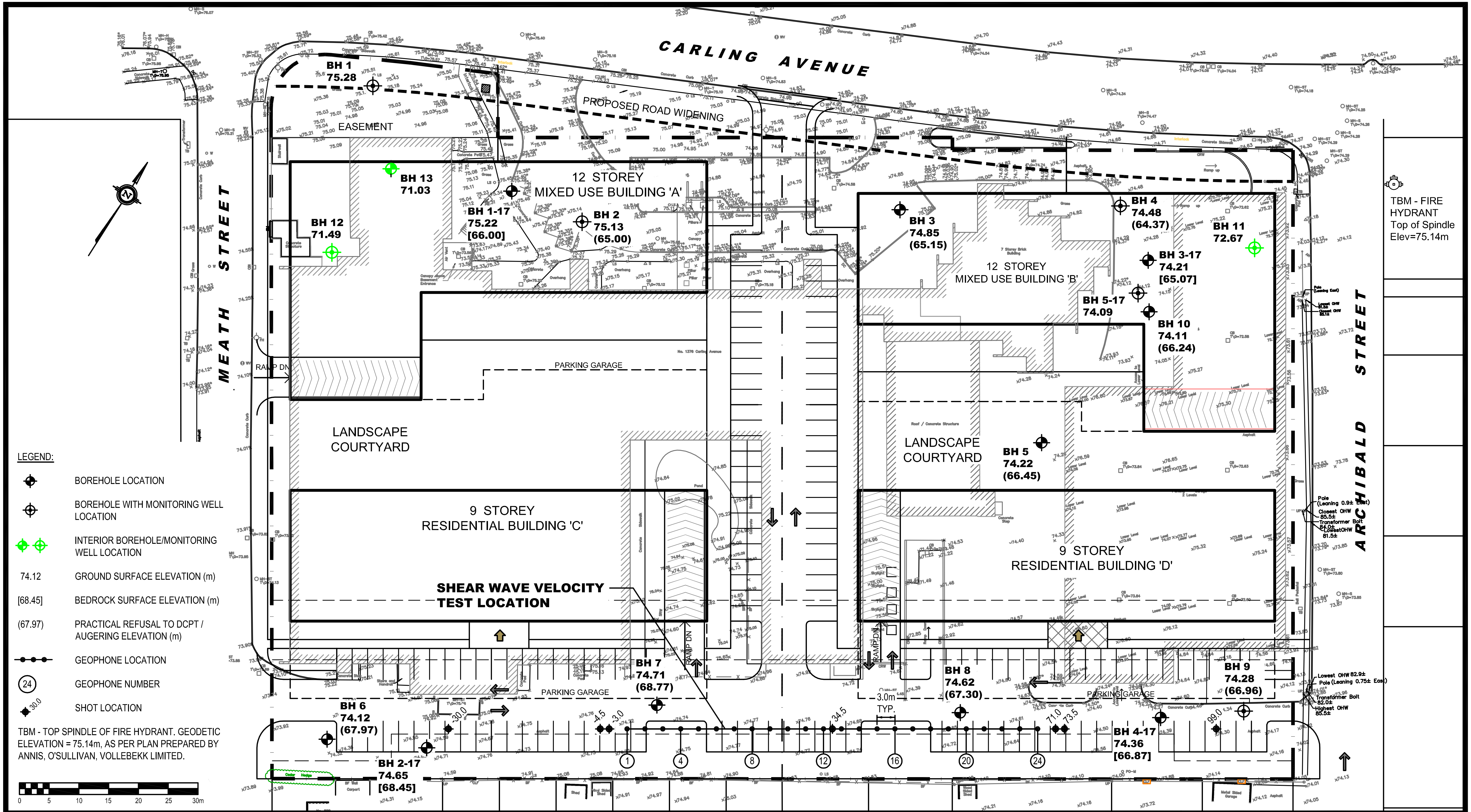


Figure 3 – Shear Wave Velocity Profile at Shot Location 73.5 m



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖ ⌖	INTERIOR BOREHOLE/MONITORING WELL LOCATION
74.12	GROUND SURFACE ELEVATION (m)
[68.45]	BEDROCK SURFACE ELEVATION (m)
(67.97)	PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
●—●—●	GEOPHONE LOCATION
㉔	GEOPHONE NUMBER
◆ 30.0	SHOT LOCATION

TBM - TOP SPINDLE OF FIRE HYDRANT. GEODETIC ELEVATION = 75.14m, AS PER PLAN PREPARED BY ANNIS, O'SULLIVAN, VOLLEBEKK LIMITED.

patersongroup
consulting engineers

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL
1	NEW BOREHOLES AND SEISMIC ARRAY SURVEY ADDED	29/08/2017	DJG

HOLLOWAY LODGING CORP.
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY REDEVELOPMENT - 1354 TO 1376 CARLING AVE.
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 11/2016
Drawn by: MPG
Report No.: PG3736-2
Checked by: RG
Dwg. No.: PG3736-1
Approved by: DJG
Revision No.: 1

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