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Geotechnical Investigation

Proposed Multi-Storey Building
1335 and 1339 Bank Street
Ottawa, Ontario

Prepared For

Boulet Construction

December 3, 2019

Report PG5044-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Boulet Construction to prepare the current geotechnical report for the proposed multi-storey building located at 1335 and 1339 Bank Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

❏ determine the subsurface soil and groundwater conditions by means of boreholes

❏ review available subsoil and groundwater information previously prepared by for the subject site.

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

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

Investigating the presence or potential presence of contamination on the subject property was carried out as a separate program and is reported under separate cover.

2.0 Proposed Development

It is understood that the proposed development will consist of a multi-storey building with two (2) levels of underground parking. It is expected that the proposed structure will occupy the entire boundary of the subject site.

It is further understood that the proposed development will be municipally serviced with water and sewer.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field portion of the geotechnical investigation was conducted on October 31, 2019. At that time, a total of three (3) boreholes were completed across the subject site to a maximum depth of 10.3 m to provide general coverage of the proposed development. The boreholes completed by Paterson were conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Relevant test holes completed by others as part of the previous subsoil and groundwater investigations have been included as part of the current geotechnical report. The approximate location of the test holes are presented on Drawing PG5044-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were generally drilled using a truck-mounted drill rig operated by a two-person crew, with the exception of boreholes BH 7 through BH 11, BH 101, and MW101 through MW104, which were advanced by others using a geoprobe. The drilling procedure consisted of drilling to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered by a 50 mm diameter split-spoon sampler, from the auger flights or a core barrel. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All rock core was classified on site and placed into core boxes. All samples were transported to our laboratory. The depths at which the split-spoon, auger and rock core samples were recovered from the boreholes are shown as SS, AU and RC, 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.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each core run of bedrock and are presented on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled rock core. The RQD value is the ratio, in percentage, of core length greater than 100 mm over the total core run length.

The 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.

Groundwater

Monitoring wells were installed in all three (3) boreholes during the current geotechnical investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❏ 1.5 to 3 m long slotted 32 mm diameter PVC screen sealed at strategic depths.
- ❏ 32 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 from 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

Sample Storage

The samples from the current geotechnical investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless 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 the top of spindle of the fire hydrant located at the southeast corner of the intersection of Bank Street and the Riverside Drive southbound lane. A geodetic elevation of 61.05 m was assigned to the TBM based on the drawing prepared by Farley, Smith & Denis Surveying Ltd. The location of the TBM, boreholes and the ground surface elevation at each borehole location completed during the current investigation are presented on Drawing PG5044-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

A total of 3 representative soil samples were submitted for grain size distribution analysis as part of the previous subsoil and groundwater investigations completed by others. The results are presented on the Grain Size Distribution sheets in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing. The sample was analyzed to determine the concentration of sulphate, chloride, resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of 2 contiguous properties identified as 1335 Bank Street and 1339 Banks Street, respectively. 1335 Bank Street is located at the southeast corner of Bank Street and the Riverside Drive southbound lane, and was formerly occupied by a retail fuel outlet and is currently used as an automotive service garage. The existing automotive garage consists of a one storey commercial slab-on-grade structure with two automotive bays and office space. 1335 Bank Street is bordered to the north by the Riverside Drive southbound lane followed by the Rideau River, to the west by Bank Street followed by vacant land, to the south by 1339 Bank Street, and to the east by a multi-storey office building.

1339 Bank Street is located at the northeast corner of the intersection of Bank Street and the Riverside Drive northbound lane. The site was originally developed with a commercial building in the 1920s and subsequently converted into an automotive service garage and retail fuel outlet before being redeveloped into a commercial restaurant. 1339 Bank Street is bordered to the west by Bank Street followed by commercial property, to the south by the Riverside Drive northbound lane followed by a multi-storey office building, to the east by an asphalt paved-parking area, and to the north by 1335 Bank Street.

1335 and 1339 Bank Street are approximately at grade with the adjacent roadways bordering the north, south and west property boundaries. The site is approximately 1 to 1.5 m above the existing grade of the neighbouring property to the east, which is supported by a concrete retaining wall along the east property boundary.

4.2 Subsurface Profile

Overburden

Generally, the soil conditions encountered at the test hole locations consist of a pavement structure overlying a fill material consisting of a mixture of silty sand with clay, gravel, cobbles and some fragments of shale, brick and asphalt.

An undisturbed silty clay/clayey silt and/or silty sand was encountered directly below the fill material at depths varying between 2 to 3.5 m below existing ground surface which in turn was overlying a thin deposit of compact to dense glacial till. The glacial till was observed to generally consist of a silty sand with gravel, cobbles and occasional boulders extending to the bedrock surface at approximate depths of 6.5 to 7 m.

Bedrock

Bedrock, consisting of a dark brown to black shale bedrock, was cored at each borehole location during the current investigation to a maximum depth of 10.5 m. The recovery values and RQD values for the bedrock cores were calculated during the current investigation. The recovery values varied between 54 to 100%, while the RQD values varied between 0 and 70%, generally increasing with depth. Based on these results, the bedrock quality varies from very poor to fair.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of dark brown to black shale with laminations of siltstone of the Billings Formation with overburden thickness varying between 5 and 10 m.

4.3 Groundwater

Groundwater levels were measured on November 29, 2019 in the monitoring wells installed during the current geotechnical field investigation. The measured groundwater level readings are presented in Table 1. It should be noted that surface water can become trapped within a backfilled boreholes that can lead to higher than typical groundwater level observations.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-19	59.75	3.92	55.83	November 29, 2019
BH 2-19	59.68	3.83	55.85	November 29, 2019
BH 3-19	59.64	6.70	52.94	November 29, 2019

Note:
The ground surface elevation at each borehole location was referenced to the top of spindle of the fire hydrant located at the southeast corner of the intersection of Bank Street and Riverside Drive southbound lane. A geodetic elevation of 61.05 m was assigned to the TBM based on the drawing prepared by Farley, Smith & Denis Surveying Ltd..

Based on our review of the historical monitoring wells installed at the subject site, general knowledge of the areas geology, experience with similar development projects in the immediate area in conjunction with the drawdown effect of the nearby Rideau River, it is expected that the long-term groundwater is located approximately 3 to 4 m below existing ground surface. However, it should be noted that perched water may be encountered within the upper fractured bedrock and at the bedrock/overburden interface which may lead to an initial high infiltration for building excavation.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It is expected that a raft foundation founded on the weathered bedrock will be used to provide foundation support for the proposed building. The raft slab will also form part of the water suppression system to manage and minimize groundwater infiltration for the purpose of preventing long term dewatering of the surrounding areas.

In order to further minimize groundwater infiltration, it is also recommended that the temporary shoring system for the excavation consist of a secant pile wall which is socketed into the bedrock. This is discussed further in Section 6.0.

Bedrock removal may be required to complete the lower portion of the excavation, dependent on the specific founding depths of the proposed building. This is discussed further in Section 5.2.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under the proposed building or other settlement sensitive structures. However, the site excavation is expected to occupy the majority of the site to a depth significantly below the existing grade, therefore, all topsoil and fill materials will be removed from within the perimeter of the proposed building and other settlement sensitive structures.

Further, existing foundation walls and other construction debris should be entirely removed from within the building perimeter.

Bedrock Removal

As noted above, bedrock removal may be required for the lower portion of the excavation dependent on the final founding depths of the proposed building.

Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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 conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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

Vibration Considerations

Construction operations could be 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 a cooperative environment with the residents.

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

Two parameters determine the permissible vibrations, 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). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill placed 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 approved prior to delivery to the site. The granular material should be placed in lifts no greater than 300 mm thick and compacted with suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD).

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

Pressure Relief Chamber

To prevent the long term dewatering of adjacent structures surrounding the site, at the founding level, a pressure relief chamber will be installed along with collection pipes within excavated or grinded trenches in the bedrock. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated in the lowest level within a utility room in close proximity to the proposed sump pits. Figure 2 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- manage any water infiltration along the bedrock surface during the excavation program.
- manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.
- regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- Once the building is completed, the pressure relief valve will be fully closed to prevent any further dewatering.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of **30 kPa** will be developed over the long term and should be incorporated in the design of the raft foundation and the foundation wall. Realistically, achieving a fully watertight is not always possible due to minor water infiltration and, therefore, a realistic long term hydrostatic pressure will be closer to 15 to 20 kPa.

5.3 Foundation Design

Bearing Resistance Values - Raft Foundation

It is expected that the proposed raft foundation will extend to the weathered bedrock surface to accommodate the 2 levels of underground parking.

A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **500 kPa** could be used. The loading conditions for the contact pressure are based on sustained loads, that are generally 100% dead load and 50% live load. The factored bearing resistance at ultimate limit states (ULS) is calculated to be **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **40 MPa/m** for a contact pressure of **500 kPa**. The design of the raft foundation should consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the above assumptions for the raft foundation, the proposed structure could be designed with the above parameters and a total and differential settlement of 10 and 5 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a weathered bedrock bearing medium when a plane extends a minimum of 2H:1V, from the bottom edge of the raft through the weathered bedrock or concrete.

5.4 Design for Earthquakes

A site specific shear wave velocity test was conducted by Geophysics GPR International Inc. on December 5, 2016. According to the results of the shear wave velocity test, the average shear wave velocity of the 30 m profile for foundations within 3 m of the bedrock surface was calculated to be greater than 1,500 m/s. Therefore, a seismic **Site Class A** is applicable for the proposed building founded directly on the bedrock surface as per Table 4.1.8.4.A of the OBC 2012. The results of the shear wave velocity test are provided in Appendix 1.

5.5 Basement Slab

It is expected that the lower basement slab will be placed over the raft foundation on a layer of clear stone or free draining granular backfill which will promote drainage to the sump pit. It is expected that the basement area will be mostly parking and that a concrete slab will be used. A rigid pavement structure is presented in Subsection 5.8. The thickness of the granular subfloor layer will be dependent on the proposed elevation of the lower basement slab. It is also expected that a sump pit will be incorporated in the design of the raft slab to drain any water which enters the granular layer via a breach in the raft slab or foundation wall waterproofing system.

The final basement floor slab and associated underfloor granular material should only be placed once the pressure relief chamber valve has been fully closed and no significant water infiltration is observed after hydrostatic pressure is applied.

In consideration of the groundwater conditions encountered at the subject site, an underfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lower basement floor slab (discussed in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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.

Where undrained conditions are anticipated (i.e. below the groundwater level), 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 Rock Anchor Design

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.

The anchor should 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 40 and 50 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

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. 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 **0.575 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Parameters

Parameters used to calculate grouted rock anchor lengths are provided in Table 2.

Table 2 - 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 Shale Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Shale	60 MPa
Unit weight - Submerged Bedrock	15 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 are provided in Table 3. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2	0.8	2.8	450
	2.6	1	3.6	600
	3.2	1.2	4.4	750
	4.5	2	6.5	1000
125	1.6	0.6	2.2	600
	2	1	3	750
	2.6	1.4	4	1000
	3.2	1.8	5	1250

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.

5.8 Pavement Structure

For design purposes, the pavement structures presented in the following tables are recommended, where required.

Table 4 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

Table 5 - Recommended Flexible Pavement Structure - Lower Parking Level

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

Table 6 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation is recommended and the membrane is to be installed from 2 m below finished grade, down the foundation walls to the underside of raft elevation. It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation. The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is further recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the raft interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

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

Underfloor Drainage

Underfloor drainage will be required to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, we recommend that 150 mm in diameter perforated pipes be placed at 6 m centres underlying the basement floor slab. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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

Pressure Relief Chamber

The purpose of the pressure relief chamber will be to control the groundwater infiltration and hydrostatic pressure created by fully or partially tanking the basement level. To avoid uplift on the raft foundation slab prior to having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during the construction program.

During the construction program, the valve of the pressure relief chamber can be gradually closed as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

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, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided.

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.

It is expected that the foundations will generally not require protection against frost action due to the founding depth. However, unheated structures, such as the access ramp, may require insulation against the deleterious effect of frost action.

6.3 Excavation Side Slopes

The side slopes of the excavations should either be cut back at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the proposed building to the site boundaries, it is anticipated that a temporary shoring system will be required to support the excavation.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils and weathered bedrock. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

The temporary shoring system is recommended to consist of a secant pile wall. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. Given the depth of bedrock at the subject site, the system can be anchored or braced. 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, if required, by means of rock bolts.

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

Table 7 - Soil Parameters

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
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil/bedrock subgrade. 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 (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 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

Groundwater Control for Building Construction

Given the depth of the proposed excavation below the groundwater level and the predominantly sandy soils encountered overlying the bedrock, groundwater infiltration into the excavation is anticipated to be moderate to high. It is therefore recommended that the shoring system consist of a secant pile wall which is socketed into the bedrock in order to act as a cofferdam.

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

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

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater breaching the waterproofing system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (less than 5,000 L/day) with higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane has been recommended to lessen the effects of water infiltration. Any minor dewatering of the site will be minimal and will be within the glacial till and/or bedrock layer. Therefore, no adverse effects to the surrounding buildings or properties are expected with the lowering of the groundwater in this area.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The excavations may be completed in proximity of existing structures which could 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 which 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.

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.

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.

6.8 Slope Stability Recommendations

A review of the existing slopes located to the north of the subject site, along the banks of the Rideau River, was completed in March 2018 by others and is provided in Appendix 1. In summary, the slopes are comprised mostly of granular materials and have inclinations of 2.5H:1V to 3H:1V. The slopes are considered stable, from a geotechnical perspective, with a global factor of safety greater than 1.5 under static conditions. Further, the proposed development at the subject site will not reduce the stability or factor of safety of these slopes.

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.

- ❏ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ❏ Observe and approve the installation of the pressure relief chamber and associated piping.
- ❏ Review proposed waterproofing and foundation drainage design and requirements.
- ❏ 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 and follow-up 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 recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Boulet Construction 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.

Richard Groniger, C. Tech.

Scott S. Dennis, P.Eng.

Report Distribution

- ❏ Boulet Construction (e-mail copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

GRAIN SIZE DISTRIBUTION ANALYSIS BY OTHERS

ANALYTICAL TESTING RESULTS

SHEAR WAVE VELOCITY TEST RESULTS BY OTHERS

SLOPE STABILITY REVIEW BY OTHERS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1335 Bank Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the northern corner of Bank Street and Riverside Drive. Geodetic elevation = 59.434m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 31

FILE NO. PG5044

HOLE NO. BH 1-19

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	59.75
Asphaltic concrete	0.08	AU	1				
FILL: Brown silty sand	0.30						
FILL: Brown sand, some silt and gravel, trace brick	3.05	SS	2	38	5	1	58.75
		SS	3	42	3	2	57.75
		SS	4	0	2	3	56.75
FILL: Dark grey silty clay	3.81	SS	5	79	2		
Loose, brown **SILTY SAND-GRAVEL**	5.33	SS	6	62	4	4	55.75
		SS	7	62	6	5	54.75
Loose, grey **SILTY SAND,** occasional gravel	6.30	SS	8	100	5	6	53.75
GLACIAL TILL: Dense, brown silty sand with clay, gravel and shale fragments	6.65	SS	9	59	37		
BEDROCK: Poor to fair quality, black shale - 100mm mud seam at 6.8m depth	10.26	RC	1	100	0		
		RC	2	100	0	7	52.75
		RC	3	100	40	8	51.75
						9	50.75
		RC	4	100	52	10	49.75
End of Borehole (GWL @ 3.92m - Nov. 29, 2019)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Monitoring Well Construction

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
Prop. Multi-Storey Building - 1335 Bank Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the northern corner of Bank Street and Riverside Drive. Geodetic elevation = 59.434m.

REMARKS

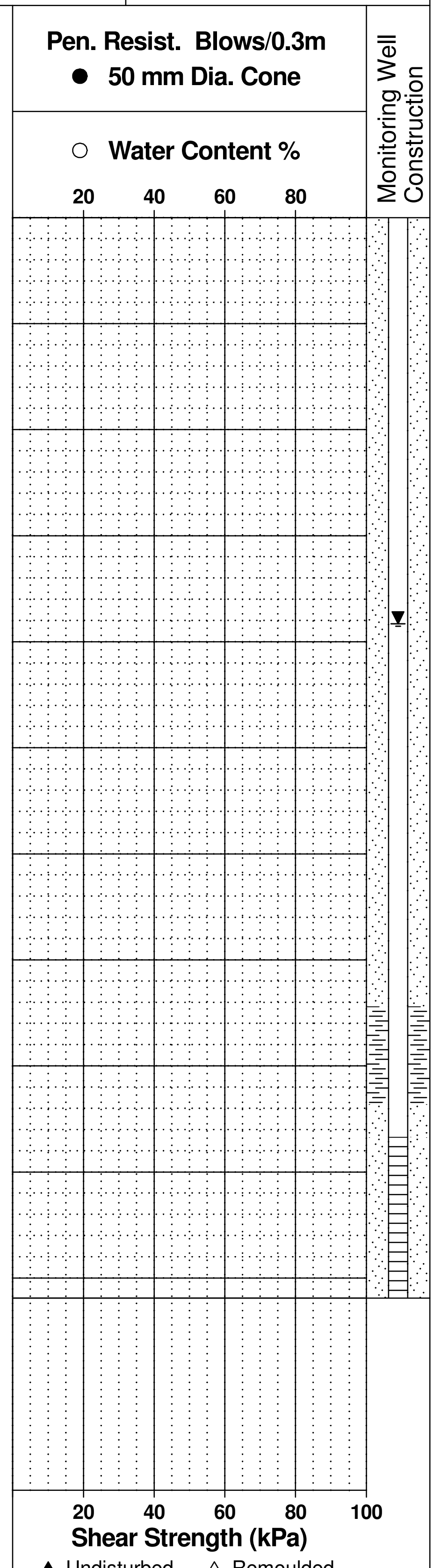
BORINGS BY CME 55 Power Auger **DATE** 2019 November 1

FILE NO. PG5044

HOLE NO. BH 2-19

SOIL DESCRIPTION	Layer depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand and gravel	3.05	SS	2	67	7
		SS	3	21	5
		SS	4	33	2
Dark grey **SILTY CLAY/CLAYEY SILT**	3.81	SS	5	71	3
Grey **SILTY CLAY**	5.33	SS	6	100	3
		SS	7	92	2
Compact to dense, brown **SILTY SAND,** some gravel	7.47	SS	8	21	20
		SS	9	100	11
		SS	3	62	33
BEDROCK: Poor to fair quality, black shale	10.19	RC	1	100	38
		RC	2	95	59
End of Borehole					
(GWL @ 3.83m - Nov. 29, 2019)					

DEPTH (m)	ELEV. (m)
0	59.68
1	58.68
2	57.68
3	56.68
4	55.68
5	54.68
6	53.68
7	52.68
8	51.68
9	50.68
10	49.68



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1335 Bank Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located on the northern corner of Bank Street and Riverside Drive. Geodetic elevation = 59.434m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 1

FILE NO. PG5044

HOLE NO. BH 3-19

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	59.64
TOPSOIL	0.15						
FILL: Brown sand with gravel, some silt	0.76	SS	1	58	10		
FILL: Brown silty sand, some gravel	1.52	SS	2	50	10	1	58.64
Compact, dark brown **SILTY SAND,** some clay, trace gravel	2.39	SS	3	71	11	2	57.64
Dense to compact, brown **SAND** with silt, occasional gravel	4.27	SS	4	42	31	3	56.64
		SS	5	54	8		
		SS	6	62	14	4	55.64
Compact to very dense, brown **SILTY FINE SAND**	5.33	SS	7	88	23	5	54.64
Very dense, grey **SILTY SAND-GRAVEL**	6.17	SS	8	83	64	6	53.64
BOULDERS	6.53	SS	9	67	50+		
BEDROCK: Poor to fair quality, black shale	10.21	RC	1	100	0	7	52.64
		RC	2	100	44	8	51.64
		RC	3	92	70	9	50.64
						10	49.64
End of Borehole							

(GWL @ 6.70m - Nov. 29, 2019)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Monitoring Well Construction

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

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

SOIL PROFILE & TEST DATA

Phase I-II-Environmental Site Assessment
1339 Bank Street
Ottawa, Ontario

DATUM TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

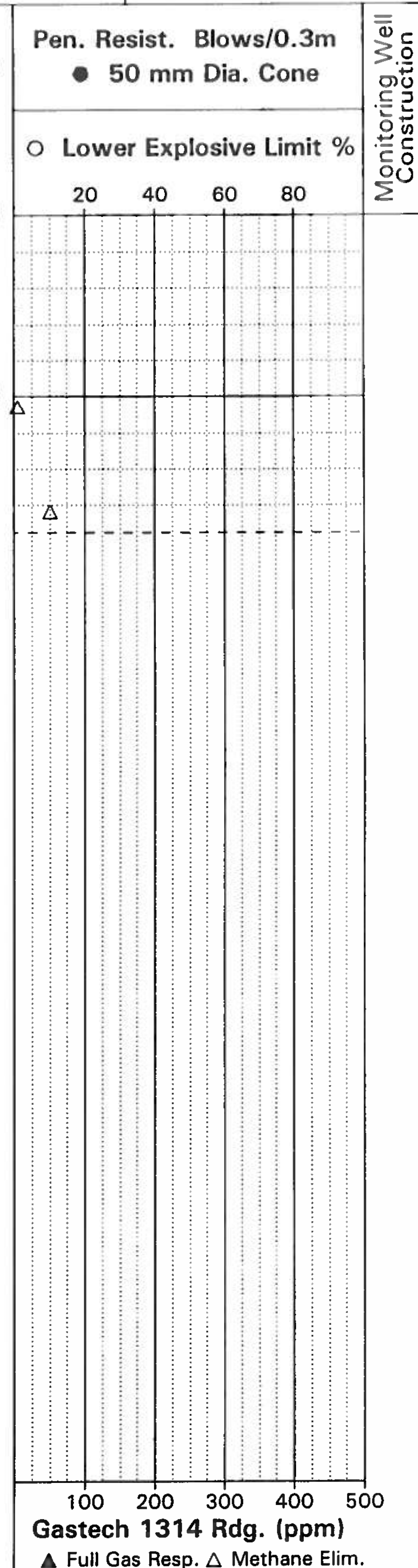
BORINGS BY CME 55 Power Auger

DATE 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.83
Asphaltic concrete 0.05							
FILL: Crushed stone 0.46							
FILL: Brown silty sand with gravel		SS	1	21	33	1	98.83
1.75		SS	2	33	50+		
End of Borehole							
Practical refusal to augering @ 1.75m depth							



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

SOIL PROFILE & TEST DATA

Phase I-II-Environmental Site Assessment
1339 Bank Street
Ottawa, Ontario

DATUM: TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

BORINGS BY: CME 55 Power Auger

DATE: 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0	99.96		
Asphaltic concrete (0.08)									
FILL: Crushed stone (0.46)									
FILL: Brown silty sand with gravel, trace clay		SS	1	75	14	1	98.96		
		SS	2	50	9	2	97.96		
(2.90)		SS	3	75	7				
Brown SILTY CLAY with sand seams						3	96.96		
		SS	4	75	6				
- grey by 4.0m depth		SS	5	67	4	4	95.96		
(4.72)									
Grey SILTY CLAY with sand and occasional gravel		SS	6	75	11	5	94.96		
(5.18)									
End of Borehole									

Lower Explosive Limit %: 20 40 60 80

Gastech 1314 Rdg. (ppm): 100 200 300 400 500

▲ Full Gas Resp. △ Methane Elim.

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

SOIL PROFILE & TEST DATA

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1339 Bank Street
Ottawa, Ontario

DATUM TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

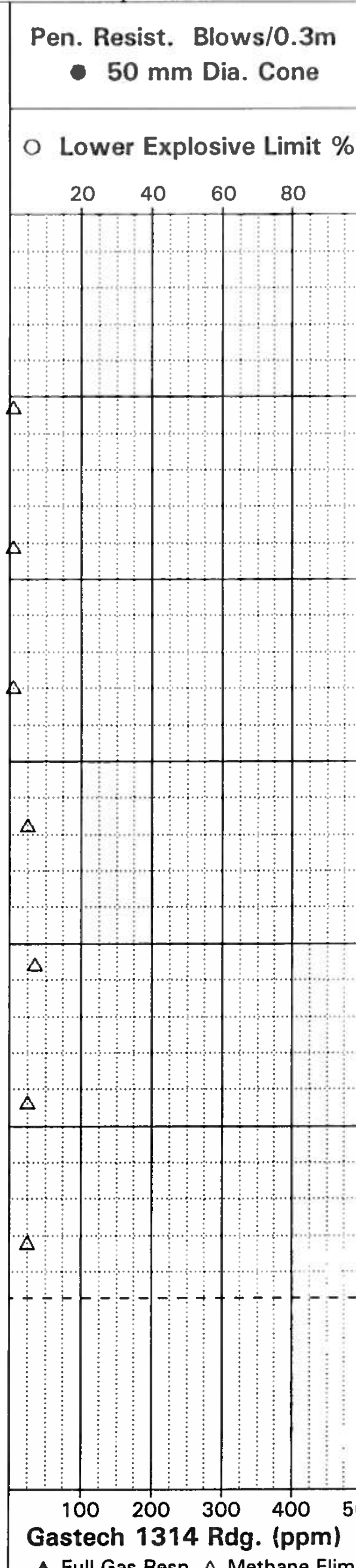
BORINGS BY CME 55 Power Auger

DATE 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.80
Asphaltic concrete	0.08						
FILL: Crushed stone	0.46						
FILL: Brown silty sand		SS	1	4	1	1	98.80
		SS	2	12	2	2	97.80
	3.05	SS	3	12	7	3	96.80
Black **PEAT**	3.66	SS	4	50	7		
Grey-brown SILTY CLAY, some sand seams, trace organic matter	4.57	SS	5	75	5	4	95.80
Compact, brown SILTY **SAND** with gravel, trace sea shells		SS	6	58	19	5	94.80
	5.94	SS	7	75	19		
End of Borehole							



Monitoring Well Construction

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

SOIL PROFILE & TEST DATA

Phase I-II-Environmental Site Assessment
1339 Bank Street
Ottawa, Ontario

DATUM TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

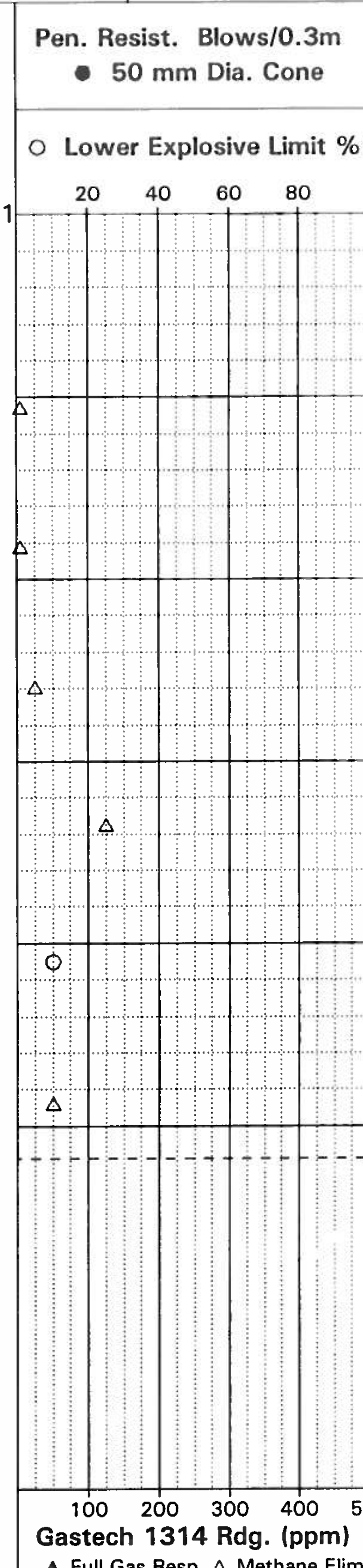
DATE 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.43				
FILL: Brown silty sand with gravel, brick and coal fragments	2.13	SS	1	50	13
		SS	2	12	4
Brown SILTY CLAY with sand seams and black staining - grey by 3.0m depth	4.27	SS	3	33	4
		SS	4	100	2
		SS	5	100	3
Very loose, grey SANDY SILT with black staining, trace clay	5.18	SS	6	25	3
End of Borehole					

DEPTH (m)	ELEV. (m)
0	100.11
1	99.11
2	98.11
3	97.11
4	96.11
5	95.11



Monitoring Well Construction

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

SOIL PROFILE & TEST DATA

Phase I-II-Environmental Site Assessment
1339 Bank Street
Ottawa, Ontario

DATUM: TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

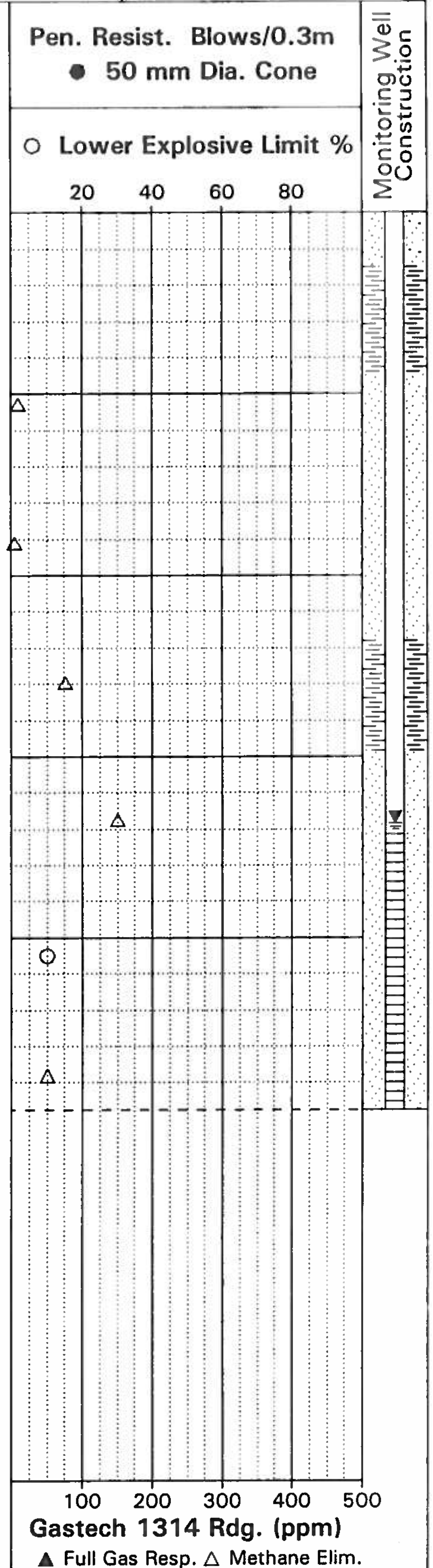
BORINGS BY: CME 55 Power Auger

DATE: 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.98
Asphaltic concrete	0.05						
FILL: Crushed stone	0.46						
FILL: Brown silty sand with gravel, brick and asphalt pieces	2.29	SS	1	17	5	1	98.98
		SS	2	25	4	2	97.98
Grey SILTY CLAY with occasional black staining	4.95	SS	3	42	4	3	96.98
		SS	4	33	5	4	95.98
		SS	5	33	4		
		SS	6	20	50+		
End of Borehole							
(GWL @ 3.37m-Dec. 7/07)							



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

SOIL PROFILE & TEST DATA

Phase I-II-Environmental Site Assessment
1339 Bank Street
Ottawa, Ontario

DATUM TBM - Finished floor level @ entrance of building, assumed elevation = 100.00m.

REMARKS

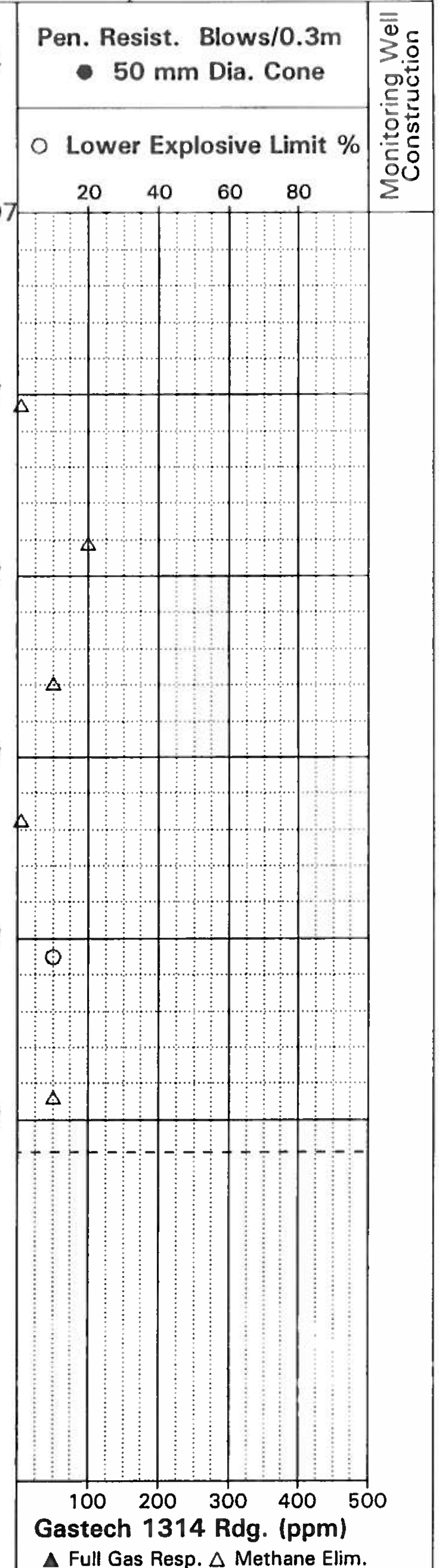
BORINGS BY CME 55 Power Auger

DATE 4 DEC 07

FILE NO. PE1283

HOLE NO. BH 6

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.07
Asphaltic concrete	0.08						
FILL: Crushed stone	0.40						
FILL: Brown silty sand		SS	1	42	5	1	99.07
	2.13	SS	2	12	2	2	98.07
FILL: Brown silty sand with gravel and wood and occasional coal pieces		SS	3	33	4	3	97.07
	3.50	SS	4	4	4		
Grey SILTY CLAY with sand seams and occasional black staining	4.57	SS	5	83	5	4	96.07
Compact, grey SANDY SILT	5.18	SS	6	17	11	5	95.07
End of Borehole							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the 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

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.

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

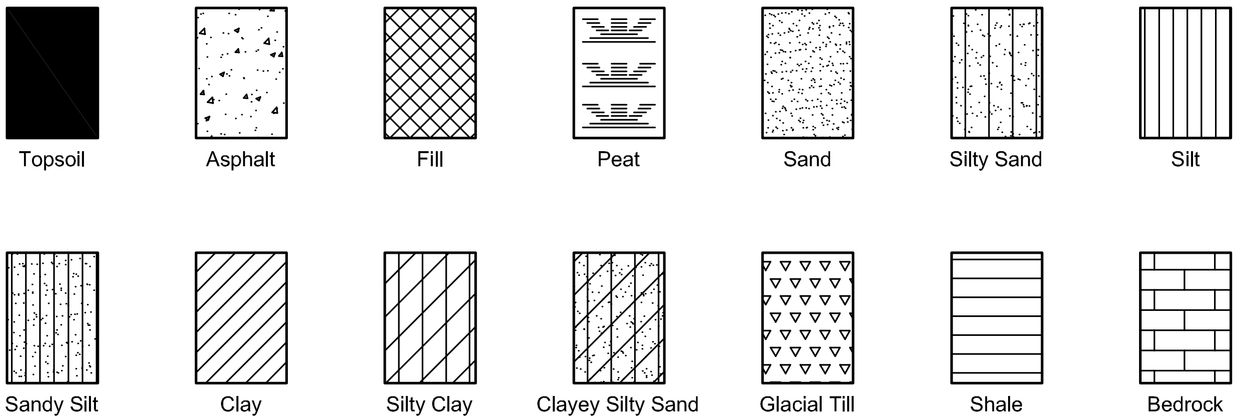
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

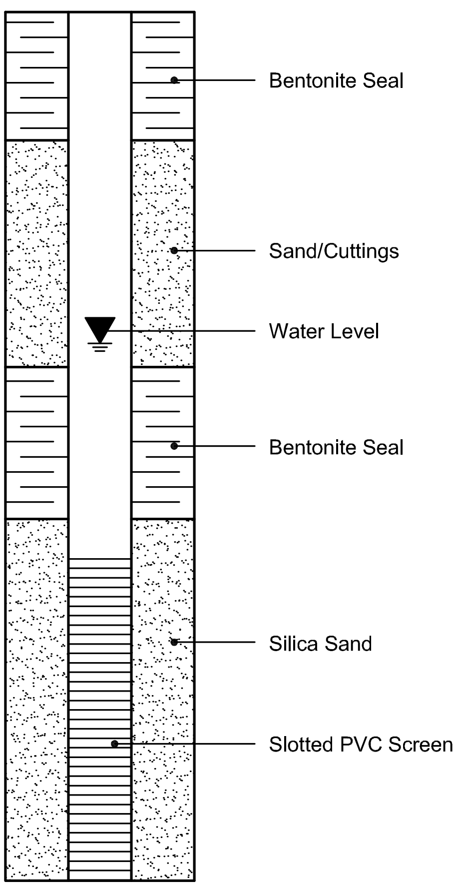
SYMBOLS AND TERMS (continued)

STRATA PLOT

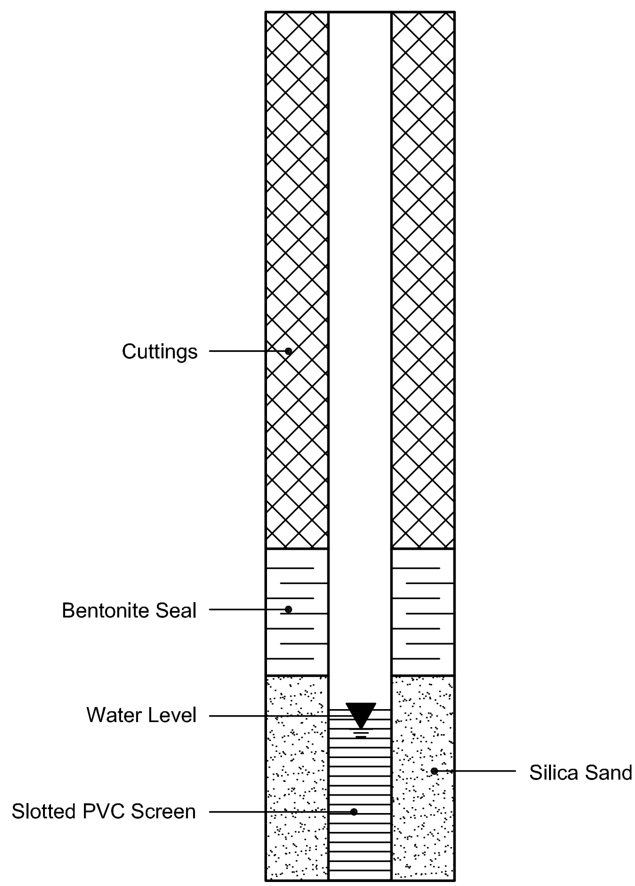


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole MW1A/B (2013)

Project No: OTT-00235328-AO

Figure No. 3

Page. 1 of 2

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

Location: 1335 Bank Street, Ottawa Ontario

Date Drilled: December 24, 2013 and September 19, 2016

Drill Type: CME

Datum: Geodetic

Logged by: TG

Checked by: MM

Legend	Symbol	Legend	Symbol
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Depth (m)	Geodetic (m)	SOIL DESCRIPTION	SPT N Value	Combustible Vapour Reading (ppm)	Natural Moisture Content
0	60.13 / 60.1	**ASPHALT** ~ 50 mm			
		FILL Silty sand, some gravel, wood chips, black, moist, no PHC odours, (compact)			
1			15	30	×
2			19	20	×
2.5			12	50	×
3	57.33 / 57.1	**SILTY CLAY AND SAND** Sand layers, black, wet, PHC odour and sheen, (firm)	5	200	×
4			12	300	×
4.5	55.6	**SAND** Layered silty fine sand and coarse sand, occasional gravel, some clay pockets, brown to grey, (compact to dense)			
5			38	140	×
		PHC odours to a depth of 5.3 m	20	50	×
6			38	30	×
6.3	53.8	**GRAVELLY SAND TILL** Some silt, brown, no PHC odours, (dense)			
6.7	53.4	**SHALE BEDROCK** BILLINGS FORMATION Black shale with calcareous siltstone interbeds, poor to good quality			
10					

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Overburden and bedrock monitoring wells with a 38mm diameter casing were installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Dec 28, 2013	2.7	
Jan 16, 2014	2.6	
Jan 29, 2014	2.6	
Sep 27, 2016	3.0	
Oct 27, 2016	2.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.01 - 7.19	100	57
2	7.19 - 8.61	80	0
3	8.61 - 10.14	97	87
4	10.14 - 10.66	100	97

Log of Borehole MW1A/B (2013)

Project No: OTT-00235328-AO

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

Figure No. 3

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m³
		SHALE BEDROCK BILLINGS FORMATION Black shale with calcareous siltstone interbeds, poor to good quality *(continued)*	50.13	10				
				11				
			48.5					
		BOREHOLE TERMINATED AT 11.66 m						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Overburden and bedrock monitoring wells with a 38mm diameter casing were installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Dec 28, 2013	2.7	
Jan 16, 2014	2.6	
Jan 29, 2014	2.6	
Sep 27, 2016	3.0	
Oct 27, 2016	2.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.01 - 7.19	100	57
2	7.19 - 8.61	80	0
3	8.61 - 10.14	97	87
4	10.14 - 10.66	100	97

Log of Borehole MW2A/B (2013)

Project No: OTT-00235328-AO

Figure No. 4

Page. 1 of 2

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

Location: 1335 Bank Street, Ottawa Ontario

Date Drilled: 12/24/13

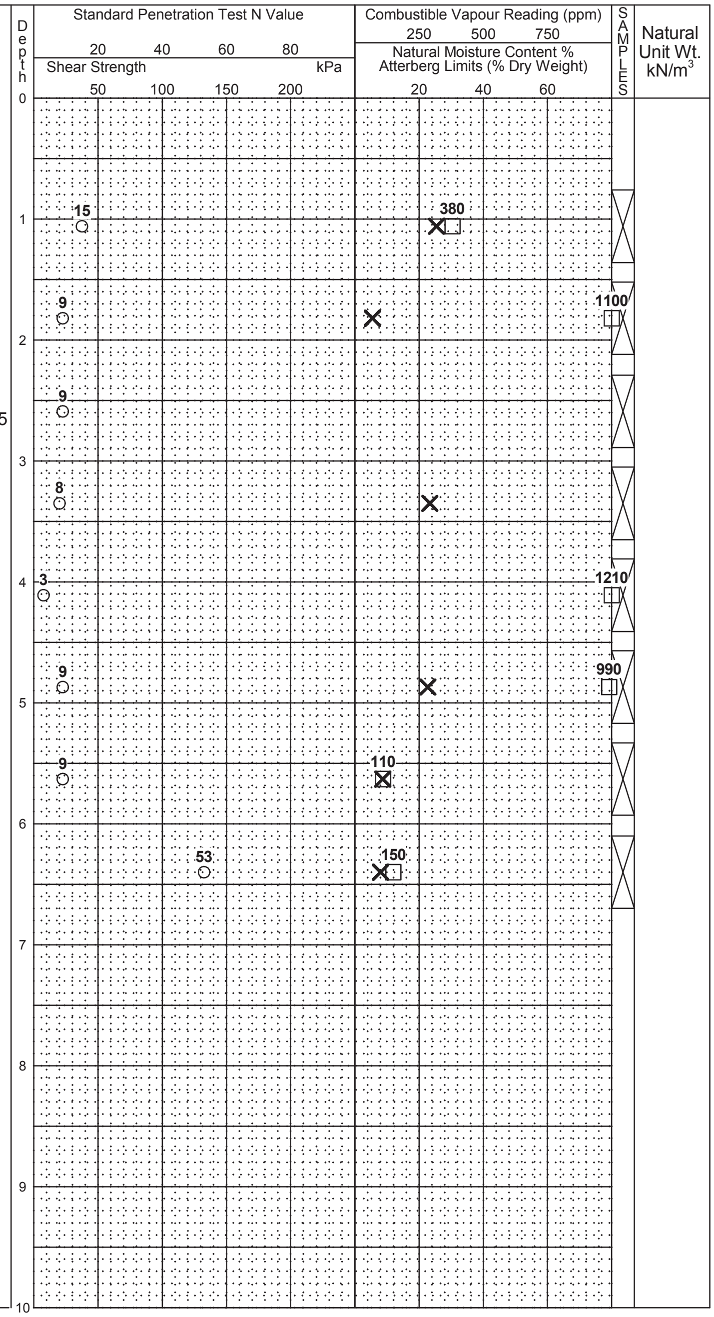
Drill Type: CME

Datum: Geodetic

Logged by: TG Checked by: MM

Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢—○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

SOIL DESCRIPTION	Geodetic m
	60.15
ASPHALT ~ 50 mm	60.1
FILL Silty sand fill, some gravel, wood chips, dark brown, moist, (compact to loose) PHC odour start at 2.2 m depth	57.45
	56.8
SILTY CLAY Sandy, some gravel, grey, wet, PHC odour and sheen, (firm to soft)	56.4
SAND Layered silty fine sand and coarse sand, some silty clay pockets or seams, grey (very loose to loose) PHC odour and sheen to a depth of 5.2 m	54.1
SILTY SAND TILL Some gravel, some shale pieces, grey, no PHC odours, (dense)	52.7
SHALE BEDROCK BILLINGS FORMATION Black shale with calcareous siltstone interbeds, poor to good quality	



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NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Overburden and bedrock monitoring wells with a 38mm diameter casing were installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Dec 28, 2013	2.6	
Jan 16, 2014	2.5	
Jan 29, 2014	2.5	
Sep 27, 2016	2.8	
Oct 27, 2016	2.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.65 - 8.48	54	37
2	8.48 - 10	98	63
3	10 - 11.53	98	85

Log of Borehole MW2A/B (2013)

Project No: OTT-00235328-AO

Figure No. 4

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m³
		SHALE BEDROCK BILLINGS FORMATION Black shale with calcareous siltstone interbeds, poor to good quality *(continued)*	50.15	10				
				11				
			48.6					
		BOREHOLE TERMINATED AT 11.5 m						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Overburden and bedrock monitoring wells with a 38mm diameter casing were installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Dec 28, 2013	2.6	
Jan 16, 2014	2.5	
Jan 29, 2014	2.5	
Sep 27, 2016	2.8	
Oct 27, 2016	2.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.65 - 8.48	54	37
2	8.48 - 10	98	63
3	10 - 11.53	98	85

Log of Borehole MW3A/B (2013)

Project No: OTT-00235328-AO

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

Location: 1335 Bank Street, Ottawa Ontario

Figure No. 5

Page. 1 of 2

Date Drilled: 12/23/13

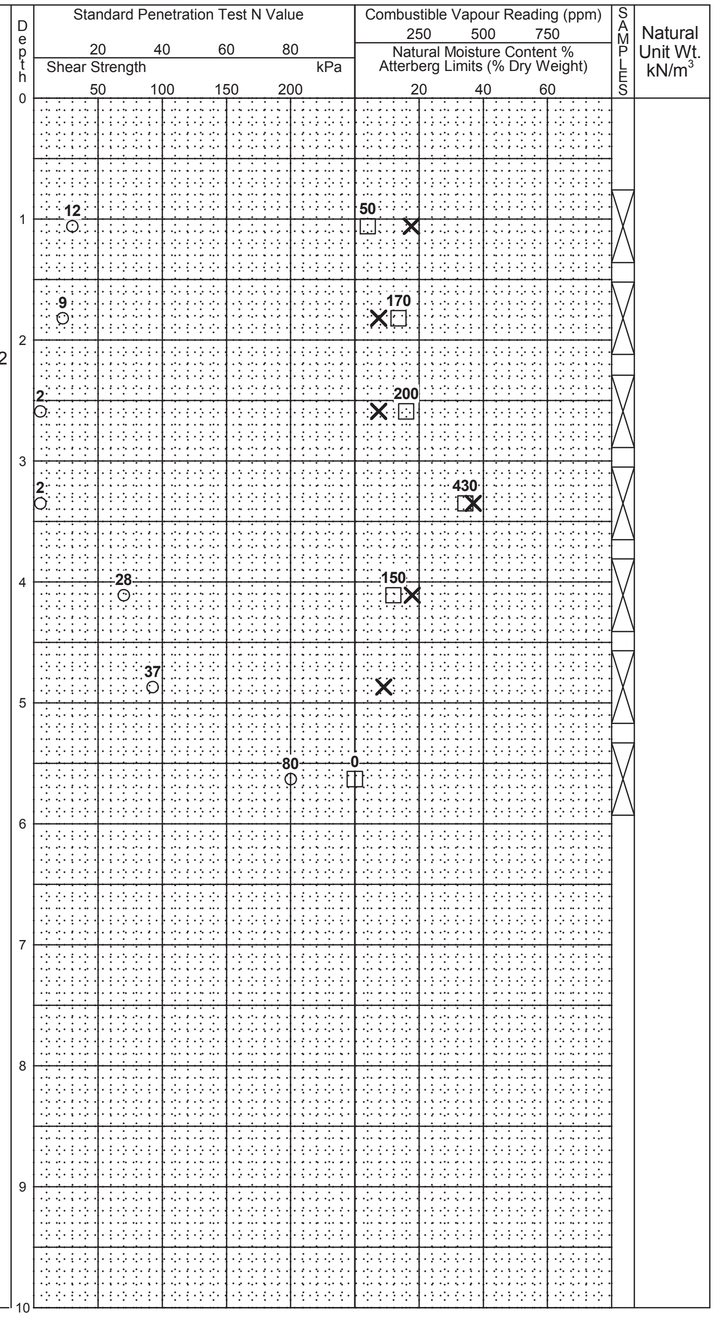
Drill Type: CME

Datum: Geodetic

Logged by: TG Checked by: MM

Legend		Legend	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample		Natural Moisture Content	X
SPT (N) Value	○	Atterberg Limits	
Dynamic Cone Test		Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
			59.92	0
		ASPHALT ~ 50 mm	59.9	
		FILL Silty sand, some gravel, red brick pieces, dark brown, moist, no PHC odour, (compact to loose)	57.72	1, 2
		SILTY CLAY AND SAND Sand layers, grey, wet, PHC odour, (soft)	57.5	3
		SAND Layered fine and coarse sand, some silty clay pockets, grey, PHC odour, (compact)	56.1	4
		SILTY SAND TILL Some gravel, shale pieces, grey, (dense to very dense) PHC odour to a depth of 5.3 m	54.9	5, 6
		SHALE BEDROCK BILLINGS FORMATION Black shale with calcareous siltstone interbeds, very poor to fair quality	53.3	7, 8, 9, 10



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NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A monitoring well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Jan 13, 2014	2.1	
Jan 15, 2014	2.0	
Jan 28, 2014	2.2	
Sep 27, 2016	2.2	
Oct 27, 2016	2.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.1 - 7.19	84	22
2	7.19 - 8.71	72	52
3	8.71 - 10.23	100	67

Log of Borehole MW3A/B (2013)

Project No: OTT-00235328-AO

Figure No. 5

Project: Geotechnical Investigation. Proposed 16 Storey Mixed Use Building

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m^3
			49.92	10				
			49.7					
		BOREHOLE TERMINATED AT 10.2 m						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A monitoring well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00235328-AO

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Jan 13, 2014	2.1	
Jan 15, 2014	2.0	
Jan 28, 2014	2.2	
Sep 27, 2016	2.2	
Oct 27, 2016	2.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.1 - 7.19	84	22
2	7.19 - 8.71	72	52
3	8.71 - 10.23	100	67

PINCHIN ENVIRONMENTAL

Log of Borehole: BH-7

Project #: 58781.001 **Logged By:** MK

Project: Supplemental Phase II ESA

Client: Cara Operations Limited

Location: 1339 Bank Street, Ottawa, ON

Drill Date: October 29, 2010 **Project Manager:** FG

SUBSURFACE PROFILE

Description	Depth (m)	Monitoring Well Details
Ground Surface	0	NO MONITORING WELL INSTALLED
ASPHALT	0.15	
SANDY GRAVEL Brown, moist, trace clay throughout	3.51	
CLAY Grey, moist, PHC like odour	5.33	
End of Borehole Refusal		

SAMPLE

Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
1	40	NA	BH-7, SS-1	0.7	
2	40	NA	BH-7. SS-2	1	
3	40	NA	BH-7, SS-3	1	
4	40	NA	BH-7, SS-4	1.5	
5	60	NA	BH-7, SS-5	2	
6	60	NA	BH-7, SS-6	32	
7	100	NA	BH-7, SS-7	155	
8	100	NA	BH-7, SS-8	211	
9	60	NA	BH-7, SS-9	39	PHC BTEX

Depth scale: ft 0–20; m 0–6

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc.

Drilling Method: Geo-Probe

Well Casing Size: 5.1 cm

Top of Casing Elevation: NA

Groundwater Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH-8

PINCHIN ENVIRONMENTAL

Project #: 58781.001 **Logged By:** MK

Project: Supplemental Phase II ESA

Client: Cara Operations Limited

Location: 1339 Bank Street, Ottawa, ON

Drill Date: October 29, 2010 **Project Manager:** FG

SUBSURFACE PROFILE					SAMPLE					
Depth (ft / m)	Symbol	Description	Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0 ft / 0 m		Ground Surface		NO MONITORING WELL INSTALLED						
		ASPHALT	0.15							
		SANDY GRAVEL Brown, moist, PHC like odour, trace clay through out			1	50	NA	BH-8, SS-1	0.5	
					2	50	NA	BH-8, SS-2	1	
					3	80	NA	BH-8, SS-3	3.4	
					4	80	NA	BH-8, SS-4	2.6	
					5	60	NA	BH-8, SS-5	4	
					6	60	NA	BH-8, SS-6	5	
			3.77							
		CLAY Grey, moist, PHC like odour			7	80	NA	BH-8, SS-7	4.8	
		Turning wet	4.88		8	80	NA	BH-8, SS-8	130	PHC BTEX
		End of Borehole Refusal								

Depth scale: 0–20 ft; 0–6 m.

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc.

Drilling Method: Geo-Probe

Well Casing Size: 5.1 cm

Top of Casing Elevation: NA

Groundwater Elevation: NA

Sheet: 1 of 1

PINCHIN ENVIRONMENTAL

Log of Borehole: BH-9

Project #: 58781.001
Logged By: MK
Project: Supplemental Phase II ESA
Client: Cara Operations Limited
Location: 1339 Bank Street, Ottawa, ON
Drill Date: October 29, 2010
Project Manager: FG

SUBSURFACE PROFILE / SAMPLE

Depth	Description	Depth (m)	Monitoring Well Details
ft 0 / m 0	Ground Surface		
	ASPHALT	0.15	NO MONITORING WELL INSTALLED
	SANDY GRAVEL Brown, moist, PHC like odour, trace clay throughout	2.86	
	CLAY Grey, moist, PHC like odour	4.57	
	End of Borehole Refusal		

Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
1	40	NA	BH-9, SS-1	1.2	
2	40	NA	BH-9, SS-2	1.7	
3	40	NA	BH-9, SS-3	2	
4	40	NA	BH-9, SS-4	3	
5	60	NA	BH-9, SS-5	3.5	
6	60	NA	BH-9, SS-6	3.8	
7	100	NA	BH-9, SS-7	4.1	
8	100	NA	BH-9, SS-8	168	PHC BTEX

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc.
Drilling Method: Geo-Probe
Well Casing Size: 5.1 cm
Top of Casing Elevation: NA
Groundwater Elevation: NA
Sheet: 1 of 1

PINCHIN ENVIRONMENTAL

Log of Borehole: BH-10

Project #: 58781.001 **Logged By:** MK

Project: Supplemental Phase II ESA

Client: Cara Operations Limited

Location: 1339 Bank Street, Ottawa, ON

Drill Date: October 29, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Depth (ft)	Depth (m)	Description	Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	0	Ground Surface		NO MONITORING WELL INSTALLED						
		ASPHALT	0.15							
1–2		**SANDY GRAVEL** Brown, moist, trace clay throughout			1	40	NA	BH-10, SS-1	1.6	
3–4	1				2	40	NA	BH-10, SS-2	2.6	
5–6					3	40	NA	BH-10, SS-3	2.4	
7–8	2				4	40	NA	BH-10, SS-4	2.3	
9–10	3				5	60	NA	BH-10, SS-5	3	
11–12			3.77		6	60	NA	BH-10, SS-6	7	
13–14	4	**CLAY** Grey, moist, PHC like odour			7	100	NA	BH-10, SS-7	65	
15–16		Turning wet	4.88		8	100	NA	BH-10, SS-8	268	PHC BTEX
17–20	5–6	End of Borehole Refusal								

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc. **Top of Casing Elevation:** NA

Drilling Method: Geo-Probe **Groundwater Elevation:** NA

Well Casing Size: 5.1 cm **Sheet:** 1 of 1

Log of Borehole: BH10-7

PINCHIN ENVIRONMENTAL

Project #: 58781.001 **Logged By:** MK
Project: October 29, 2010
Client: Cara Operations Limited
Location: 1339 Bank Street, Ottawa, ON
Drill Date: October 29, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Depth (ft)	Depth (m)	Description	Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	0	Ground Surface								
		ASPHALT	0.15							
1		**SANDY GRAVEL** Brown, moist, trace gravel throughout			1	30	NA	BH-10, SS-1	0.8	
3	1				2	30	NA	BH-10, SS-2	1	
5					3	60	NA	BH-10, SS-3	2.1	
7	2	Black staining			4	60	NA	BH-10, SS-4	2	
9	3				5	60	NA	BH-10, SS-5	3.4	
11					6	60	NA	BH-10, SS-6	5	
12			3.75							
13	4	**CLAY** Grey, wet, PHC like odour		Note: Due to cave in monitoring well could only be installed at 3.96 mbgs	7	90	NA	BH-10, SS-7	42	
15			4.88		8	90	NA	BH-10, SS-8	200	PHC BTEX
16	5	End of Borehole Refusal								
20	6	Note: Groundwater level measured at 3.38 mbgs								

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc.
Drilling Method: Geo-Probe
Well Casing Size: 5.1 cm
Top of Casing Elevation: 103.891
Groundwater Elevation: 103.821
Sheet: 1 of 1

PINCHIN ENVIRONMENTAL

Log of Borehole: BH-11

Project #: 58781.001 **Logged By:** MK

Project: Supplemental Phase II ESA

Client: Cara Operations Limited

Location: 1339 Bank Street, Ottawa, ON

Drill Date: October 29, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Depth (ft)	Depth (m)	Description	Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	0	Ground Surface		NO MONITORING WELL INSTALLED						
0–0.5	0–0.15	**ASPHALT**	0.15							
0.5–10.7	0.15–3.27	**SANDY GRAVEL** Brown, moist, PHC like odour, trace clay throughout	3.27		1	20	NA	BH-11, SS-1	1	
					2	20	NA	BH-11, SS-2	1.6	
					3	50	NA	BH-11, SS-3	2.1	
					4	50	NA	BH-11, SS-4	3	
					5	70	NA	BH-11, SS-5	2.7	
10.7–15	3.27–4.57	**CLAY** Grey, moist, PHC like odour			6	70	NA	BH-11, SS-6	7	
		PHC like odour			7	100	NA	BH-11, SS-7	1305	
			4.57		8	100	NA	BH-11, SS-8	1504	PHC BTEX
15	4.57	End of Borehole Refusal								

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Contractor: Strata Soil Sampling Inc.

Drilling Method: Geo-Probe

Well Casing Size: 5.1 cm

Top of Casing Elevation: NA

Groundwater Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH101

Project #: 58781 **Logged By:** SB

Project: Phase II Environmental Site Assessment

Client: Cara Operations Limited

Location: 1339 Bank Street, Ottawa, Ontario

Drill Date: May 25, 2010 **Project Manager:** FG

SUBSURFACE PROFILE					SAMPLE					
Depth (ft / m)	Symbol	Description	Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0 / 0		Ground Surface	0.00	No Monitoring Well Installed						
0–6.5 ft / 0–1.98 m	⁙	***Sand Fill*** Brown, trace asphalt, brick and gravel, damp.	1.98		1	50	NA	BH101-SS1	4.2	
6.5–13 ft / 1.98–3.96 m	▨	***Clay*** Brown, damp. -Becoming grey. -Grey, black staining, PHC-like odours.	3.96		2	50	NA	BH101-SS2	2.3	
								BH101-SS3	1.2	
					3	40	NA	BH101-SS4	240	PHCs VOCs
		End of Borehole Refusal at 3.96 mbgs.								

Depth scale: 0–20 ft; 0–6 m.

Contractor: Strata Soil Sampling Inc.

Drilling Method: Direct push

Well Casing Size: NA

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW101

Project #: 58781 **Logged By:**

Project: Phase II Environmental Site Assessment

Client: Cara Operations Limited

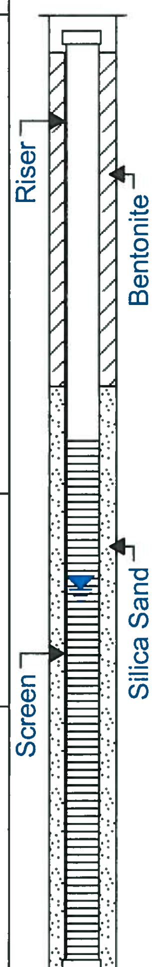
Location: 1339 Bank Street, Ottawa, Ontario

Drill Date: May 25, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Depth (ft / m)	Description	Depth (m)	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0 / 0	Ground Surface	0.00						
0–9 ft / 0–2.74 m	***Sand and Gravel Fill*** Brown, traces of asphalt, brick, concrete, damp.	2.74	1	35	NA	MW101-SS1	5	
			2	35	NA	MW101-SS2	12	
9–13 ft / 2.74–3.96 m	***Silty Clay*** Brown, some sand and gravel fill, black staining, PHC-like odours, wet.	3.96	3	40	NA	MW101-SS3	195	PHCs VOCs
13–18 ft / 3.96–5.49 m	***Sandy Silty Clay*** Brown, trace gravel, black staining, PHC-like odours, wet.	5.49	4	20	NA	MW101-SS4	125	
	End of Borehole Refusal at 5.49 mbgs.							

Monitoring Well Details:



Groundwater level at 3.30 m below top of casing as measured on May 25, 2010.

Contractor: Strata Soil Sampling Inc.

Drilling Method: Direct push

Well Casing Size: 32 mm

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW102

PINCHIN ENVIRONMENTAL

Project #: 58781 **Logged By:** SB

Project: Phase II Environmental Site Assessment

Client: Cara Operations Limited

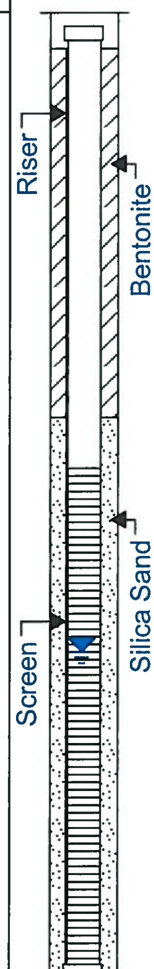
Location: 1339 Bank Street, Ottawa, Ontario

Drill Date: May 25, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Depth (ft / m)	Symbol	Description	Depth (m)	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0 / 0		Ground Surface	0.00						
0–19 ft / 0–5.79 m	Sand	**Sand** Light brown, trace gravel, brick, concrete, asphalt, damp.		1	30	NA	MW102-SS1	1.8	
				2	30	NA	MW102-SS2	0.7	
		-Varsol-like odours.							
				3	65	NA	MW102-SS3	8.8	PHCs VOCs
		-Grey discolouration, PHC-like odours, wet to 5.79 mbgs.					MW102-SS4	5.2	
		-Trace silt.	5.79						
		End of Borehole							

Monitoring Well Details:



Groundwater level at 3.85 m below top of casing as measured on May 25, 2010.

Contractor: Strata Soil Sampling Inc.

Drilling Method: Direct push

Well Casing Size: 32 mm

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW103

Project #: 58781 **Logged By:** SB

Project: Phase II Environmental Site Assessment

Client: Cara Operations Limited

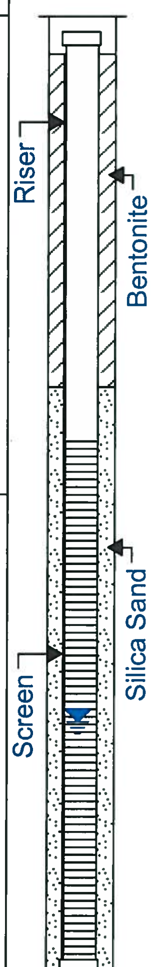
Location: 1339 Bank Street, Ottawa, Ontario

Drill Date: May 25, 2010 **Project Manager:** FG

SUBSURFACE PROFILE

Depth (m)	Description
0.00	Ground Surface
0.00 – 2.74	***Sandy Clay*** Brown, trace gravel, trace asphalt, moist, PHC-like odours in top 0.15 m. -Trace brick fragments.
2.74 – 5.49	***Clay*** Dark brown, moist. -Grey. -PHC-like odours to 4.27 mbgs. -Trace sand.
5.49	End of Borehole. Refusal at 5.49 mbgs.

Monitoring Well Details



Groundwater level at 4.05 m below top of casing as measured on May 25, 2010.

SAMPLE

Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
1	45	NA	MW103-SS1	212	
2	40	NA	MW103-SS2	7.5	
			MW103-SS3	3.3	
3	85	NA	MW103-SS4	1.6	
			MW103-SS5	220	PHCs, VOCs
4	60	NA	MW103-SS6	9.1	
			MW103-SS7	1.4	

Contractor: Strata Soil Sampling Inc.

Drilling Method: Direct push

Well Casing Size: 32 mm

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW104

Project #: 58781 **Logged By:** SB

Project: Phase II Environmental Site Assessment

Client: Cara Operations Limited

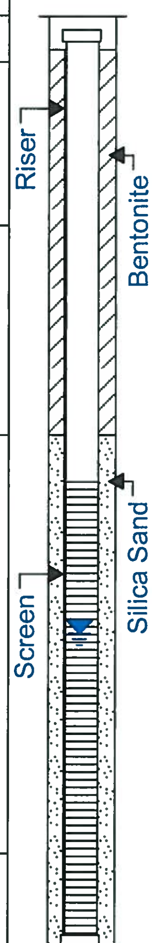
Location: 1339 Bank Street, Ottawa, Ontario

Drill Date: May 25, 2010 **Project Manager:** FG

SUBSURFACE PROFILE / SAMPLE

Description	Depth (m)	Sampler #	Recovery (%)	N-Value	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
Ground Surface	0.00						
Topsoil	0.30						
Sandy Silt Brown, trace clay, moist.	1.37	1	40	NA	MW104-SS1	6.5	
Sandy Clay Brown, trace gravel, damp.	2.74	2	20	NA	MW104-SS2	9.5	
Sandy Silt Brown, damp. -Wet.	5.49	3	30	NA	MW104-SS3	12.5	PHCs VOCs
		4	25	NA	MW104-SS4	4.6	
Clay Light grey, trace gravel, moist.	6.10	5	20	NA	MW104-SS5	1.2	
End of Borehole							

Monitoring Well Details:



Groundwater level at 4.05 m below top of casing as measured on May 25, 2010.

Contractor: Strata Soil Sampling Inc.

Drilling Method: Direct push

Well Casing Size: 32 mm

Pinchin Environmental Ltd.
2470 Milltower Court
Mississauga, ON L5N 7W5

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole BH 01 (1999)

Project No: OTT-000235328-AO- (Prev MA13771)

Figure No. 6

Project: Phase II Environmental Site Assessment

Page. 1 of 1

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Date Drilled: 'July 12, 1999

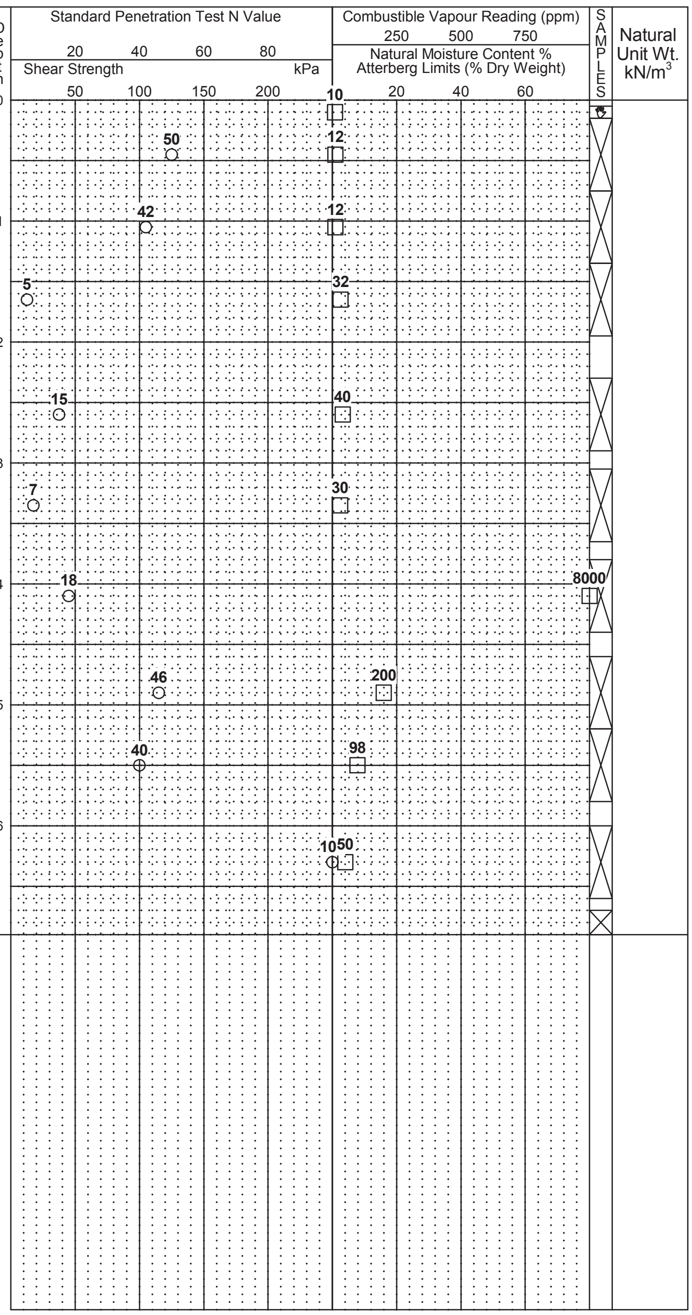
Drill Type: CME 55 Truck mount

Datum: Geodetic

Logged by: _________ Checked by: _________

Legend			
Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample		Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	
Dynamic Cone Test		Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
			59.9	0
		ASPHALTIC CONCRETE~50 mm	59.9	
		GRANULAR FILL Grey, moist, (loose)	59.6	
		SAND FILL Medium to coarse grained, some gravel, occasional asphalt, wood and brick pieces in the upper levels, brown, moist, (compact to loose)		1, 2, 3
		SILTY CLAY TO CLAY AND SILT Black staining, grey, wet, (very stiff)	56.1	
		SILTY SAND Fine to medium grained, some gravel, black staining in upper levels, occasional cobbles and boulders, grey, wet, (dense)	55.9 / 55.7	4, 5
		SHALE TILL TO WEATHERED SHALE BEDROCK Occasional sandy seems, grey, wet, (dense)	54.6	6
			53.0	
		Refusal to Augering at 6.9 m depth		



Additional Notes
1. A strong Hydrocarbon odour was detected in the soil samples retrieved from below 3.8 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 50 mm monitoring well was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.6	
3 days	4.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

LOG OF BOREHOLE BH LOGS 1999- 235328 - CUTTS MOTOR.GPJ TROW OTTAWA.GDT 7/13/17

Log of Borehole BH 02 (1999)

exp.

Project No: OTT-000235328-AO- (Prev MA13771)

Figure No. 7

Project: Phase II Environmental Site Assessment

Page. 1 of 1

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Date Drilled: 'July 12, 1999

Drill Type: CME 55 Truck mount

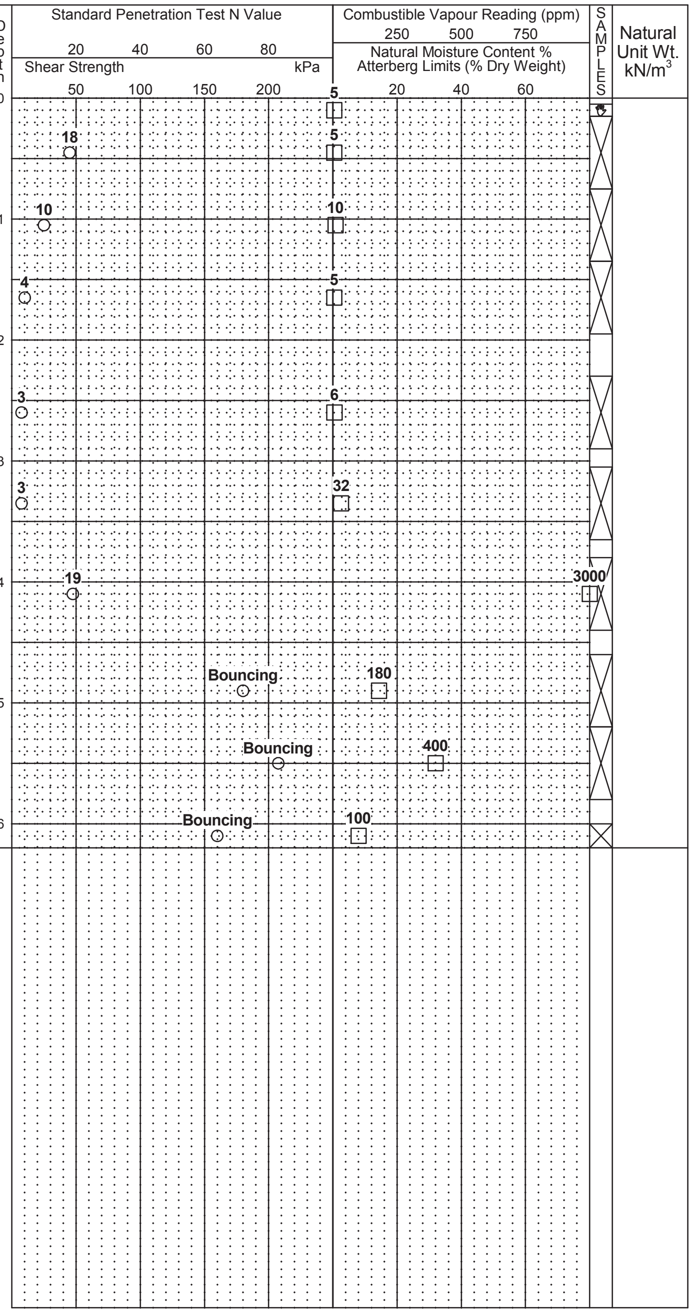
Datum: Geodetic

Logged by: _________ Checked by:_________

Symbol	Legend	Symbol	Legend
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	X
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

SOIL DESCRIPTION	Geodetic m	Depth
	60	0
ASPHALTIC CONCRETE~50 mm	60.0	
GRANULAR FILL Grey, moist, (loose)	59.5	
SAND FILL Medium to coarse grained, brown, moist, (loose to very loose)		1
	56.7	2, 3
SILTY CLAY Some sand, grey, wet, (soft)	56.2	
SILTY SAND Fine to medium grained, some gravel, black staining in upper levels, occasional cobbles and boulders, grey, wet, (dense)	56.1	4, 5
SHALE TILL TO WEATHERED SHALE BEDROCK Occasional sandy seems, grey, very wet, (dense)	54.7	6
	53.8	

Refusal to Augering at 6.2 m depth



Additional Notes

1. A Hydrocarbon odour was detected in the soil samples retrieved from below 3.8 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 50 mm monitoring well was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.6	
4 days	3.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 03 (1999)

Project No: OTT-000235328-AO- (Prev MA13771)

Figure No. 8

Project: Phase II Environmental Site Assessment

Page. 1 of 1

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Date Drilled: 'July 12, 1999

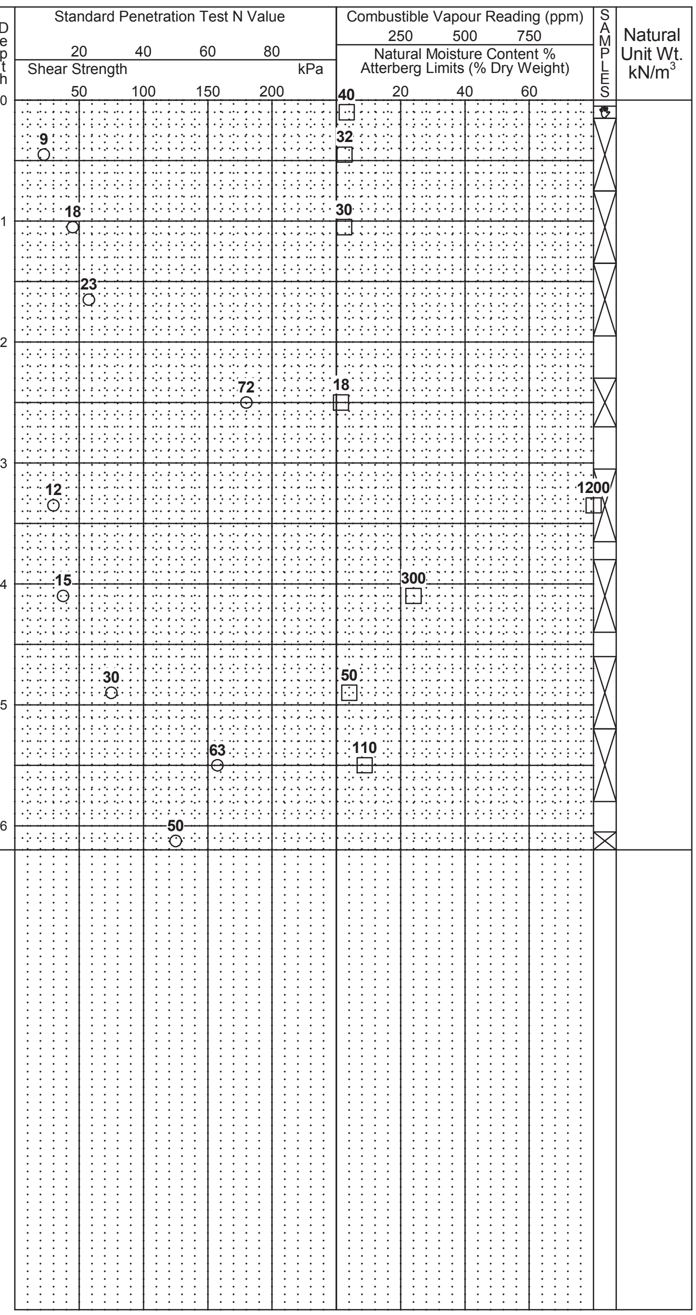
Drill Type: CME 55 Truck mount

Datum: Geodetic

Logged by: ________ Checked by: ________

Legend		Legend	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

SOIL DESCRIPTION	Geodetic (m)
	60.3
ASPHALTIC CONCRETE~50 mm	60.3
GRANULAR FILL Grey, moist, (loose)	60.0
SAND FILL Medium to coarse grained, some coarse gravel, occasional brick fragments, brown, moist, (compact)	57.3
SILTY CLAY Some sand, grey, moist, (firm)	56.5
SILTY SAND TOP SAND Fine to medium grained, some gravel, grey, very wet, (compact)	56.4 / 55.0
SILTY SAND TILL Fine to medium grained, grey, very wet, (compact to dense)	54.1
Refusal to Augering at 6.2 m depth	



Additional Notes
1. A Hydrocarbon odour was detected in the soil samples retrieved from below 3.0 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13 mm slotted standpipe was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	
4 days	3.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 04 (1999)

Project No: OTT-000235328-AO- (Prev MA13771)

Figure No. 9

Page. 1 of 1

Project: Phase II Environmental Site Assessment

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Date Drilled: 'July 13, 1999

Drill Type: CME 55 Truck mount

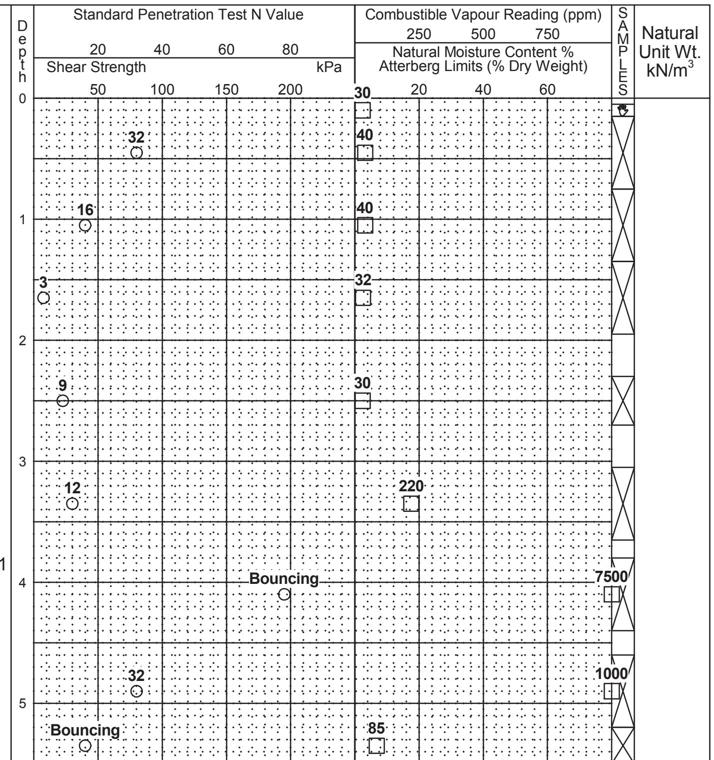
Datum: Geodetic

Logged by: ________ Checked by: ________

Split Spoon Sample	Combustible Vapour Reading
Auger Sample	Natural Moisture Content
SPT (N) Value	Atterberg Limits
Dynamic Cone Test	Undrained Triaxial at % Strain at Failure
Shelby Tube	
Shear Strength by Vane Test	Shear Strength by Penetrometer Test

SOIL DESCRIPTION	Geodetic m
	60
ASPHALTIC CONCRETE~50 mm	60.0
GRANULAR FILL Grey, moist, (loose)	59.5
SAND FILL Medium to coarse grained, some coarse gravel and silt, brown, moist, (compact)	56.6
SILTY CLAY Some sand, grey, moist, (firm)	56.1
	55.4
SILTY SAND TOP SAND TILL Fine to medium grained, some gravel, occasional cobbles and boulders, grey, very wet, (compact)	54.5

Refusal to Augering at 5.5 m depth



Additional Notes

1. A Hydrocarbon odour was detected in the soil samples retrieved from below 3.0 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 13 mm slotted standpipe was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.6	
4 days	3.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

LOG OF BOREHOLE BH LOGS 1999- 235328 - CUTTS MOTOR.GPJ TROW OTTAWA.GDT 7/13/17

Log of Borehole BH 05 (1999)

Project No: OTT-000235328-AO- (Prev MA13771)

Project: Phase II Environmental Site Assessment

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Figure No. 10

Page. 1 of 1

Date Drilled: 'July 12, 1999

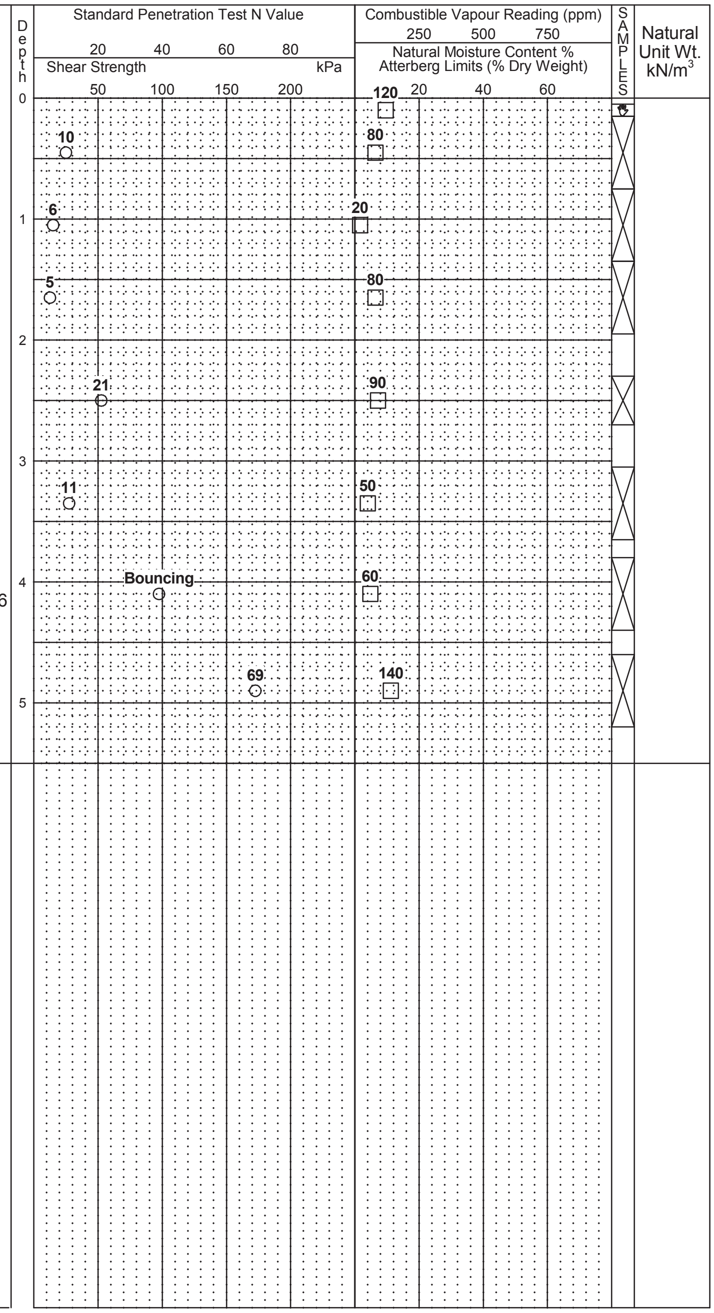
Drill Type: CME 55 Truck mount

Datum: Geodetic

Logged by: ________ Checked by: ________

Legend		Legend	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─⊖
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

SOIL DESCRIPTION	Geodetic m
	60.2
ASPHALTIC CONCRETE~50 mm	60.2
GRANULAR FILL Grey, moist, (loose)	
SAND FILL Medium to coarse grained, some coarse gravel and silt, brown, moist, (compact)	59.3
SILTY CLAY Some sand, grey, moist, (firm) -Becoming sandy silt with black staining below 3.8 m depth	57.9
SILTY SAND TOP SAND TILL Fine to medium grained, some gravel, occasional cobbles and boulders, grey, very wet, (compact)	56.0 56
	54.7
Refusal to Augering at 5.5 m depth	



Additional Notes
1. A Hydrocarbon odour was detected in the soil samples retrieved from below 3.0 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 50 mm monitoring well was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.3	
3 days	4.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29184

Report Date: 20-Nov-2019

Order Date: 18-Nov-2019

Project Description: PG5044

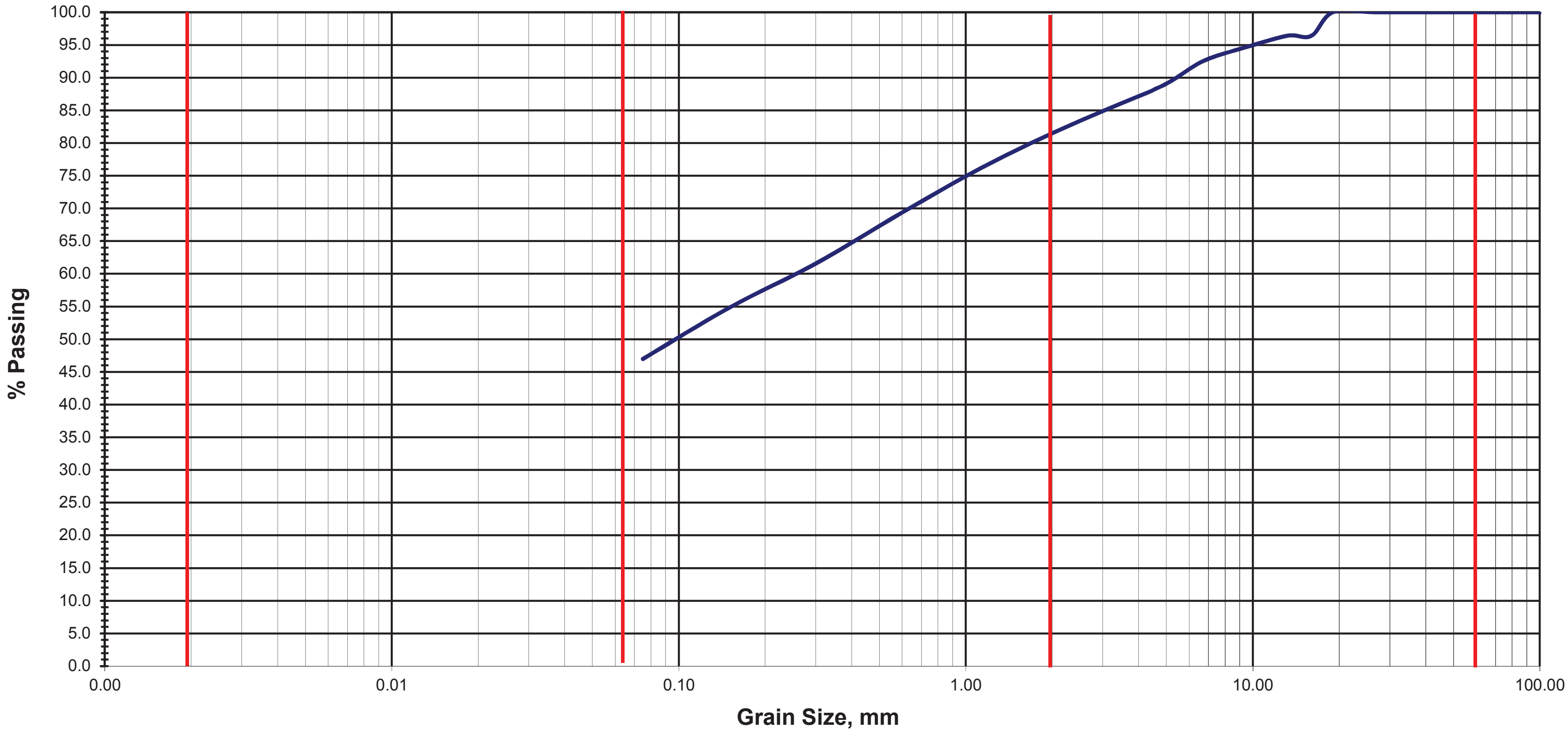
	Client ID:	BH3-SS5 10'-12'	-	-	-
	Sample Date:	01-Nov-19 11:00	-	-	-
	Sample ID:	1947113-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.39	-	-	-
Resistivity	0.10 Ohm.m	49.6	-	-	-
Anions					
Chloride	5 ug/g dry	32	-	-	-
Sulphate	5 ug/g dry	28	-	-	-

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Method of Test for Sieve Analysis of Aggregate

ASTM C-136 (LS-602)

Grain Size Distribution Curve



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

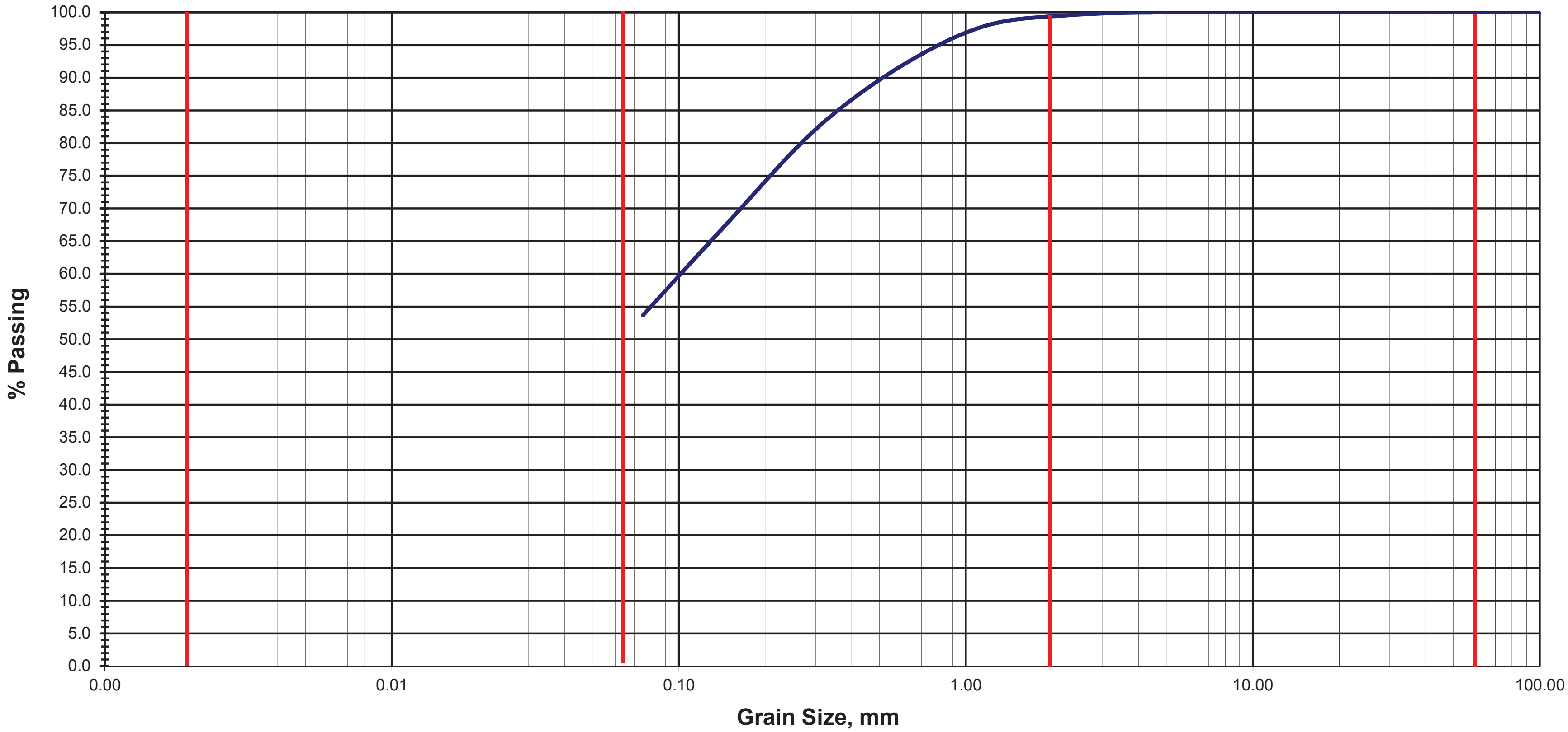
Exp Project No.: OTT-00235328-A0	Project Name : Preliminay Geotechnical Investigation - Proposed 14 Storey Building				
Client : 1924324 Ontario Inc.	Project Location : 1335 Bank Street, Ottawa, Ontario				
Date Sampled : December 24, 2013	BOREHOLE	MW1A	SAMPLE	SS5	Depth (m) : 3.8 - 4.4
Sample Description :	Silty Clayey Sand, Some Gravel				Figure : 11

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Method of Test for Sieve Analysis of Aggregate

ASTM C-136 (LS-602)

Grain Size Distribution Curve



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

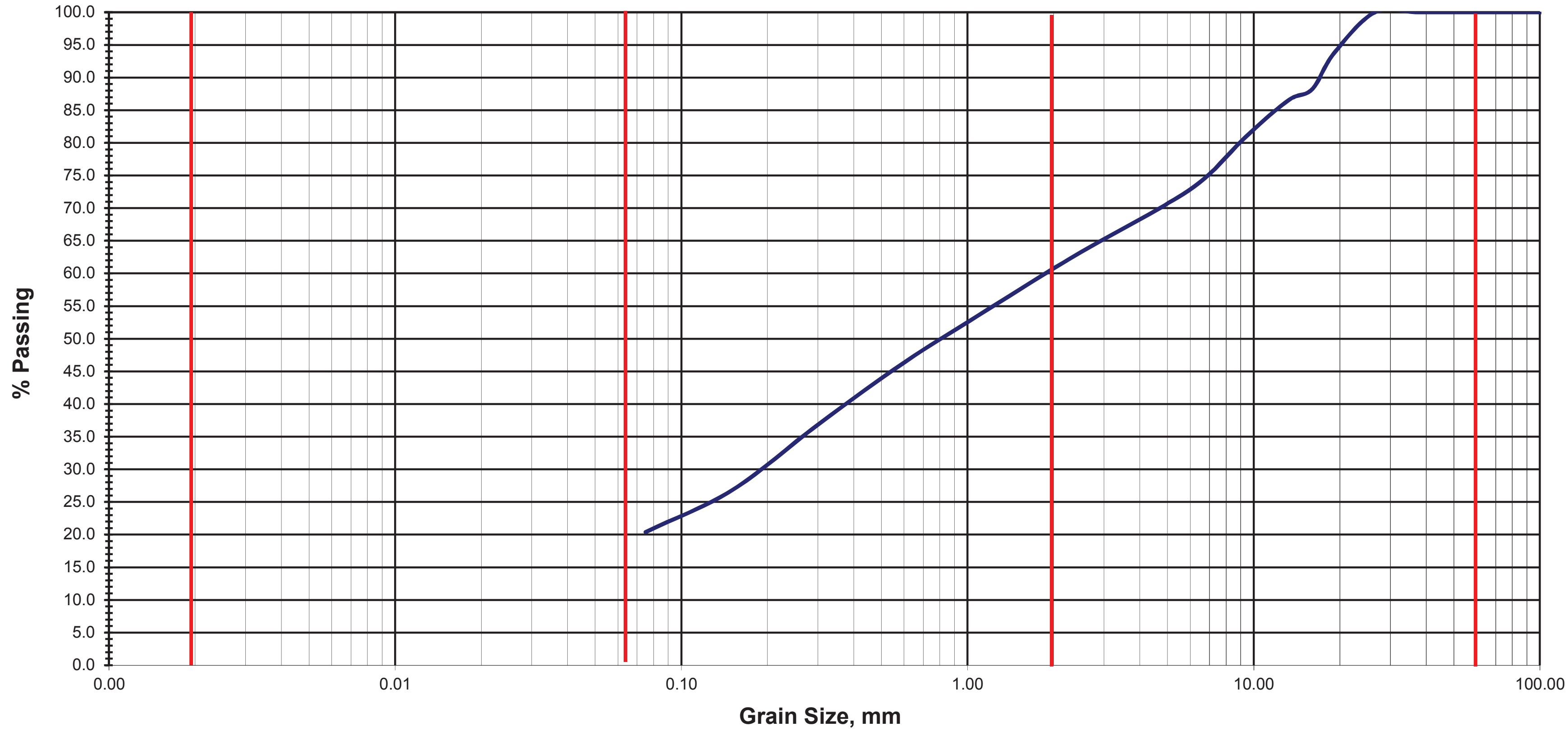
Exp Project No.:	**OTT-00235328-A0**	**Project Name :**	**Preliminay Geotechnical Investigation - Proposed 14 Storey Building**				
Client :	**1924324 Ontario Inc.**	**Project Location :**	**1335 Bank Street, Ottawa, Ontario**				
Date Sampled :	**December 23, 2013**	**BOREHOLE**	**MW3**	**SAMPLE**	**SS3**	**Depth (m) :**	**2.3 - 2.9**
Sample Description :	**Silty Sand, Some Clay**					**Figure :**	**12**

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Method of Test for Sieve Analysis of Aggregate

ASTM C-136 (LS-602)

Grain Size Distribution Curve



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			
Modified M.I.T. Classification										

Exp Project No.:	OTT-00235328-A0	Project Name :	Preliminay Geotechnical Investigation - Proposed 14 Storey Building		
Client :	1924324 Ontario Inc.	Project Location :	1335 Bank Street, Ottawa, Ontario		
Date Sampled :	December 24, 2013	BOREHOLE MW1	SAMPLE SS8	Depth (m) :	6.1 - 6.8
Sample Description :	Sand and Gravel, Some Clay			Figure :	13

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@gprmtl.com
www.geophysicsgpr.com

December 16th, 2016

Transmitted by email: ismail.taki@exp.com
Our Ref.: M-16378

Mr. Ismail M. Taki, M.Eng., P.Eng.
Manager, Geotechnical Services
exp Services Inc.
100 – 2650 Queensview Drive
Ottawa (ON) K2B 8H6

Subject: **Shear-Wave Velocity Soundings, 1335 Bank Street, Ottawa**

[Project: OTT-00235328-AO]

Dear Sir,

Geophysics GPR International Inc. has been requested by **exp** Services Inc. to carry out seismic shear wave surveys at 1335, Bank Street, in Ottawa. The geophysical investigations used the Multi-channel Analysis of Surface Waves (MASW), the Extended SPatial AutoCorrelation (ESPAC), and the seismic refraction methods. From the subsequent results, the $\overline{V}_{S30}$ value was calculated to identify the Site Class.

The surveys were carried out on the evening of December 5th, by Mr. Charles Trottier, M.Sc., Phys. and Mr. Alexis Marchand, intern. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendices.

The following paragraphs briefly describe the survey design, the principles of the test methods, and the results in graphic and table format.

ISO 9001

METHODS PRINCIPLES

MASW Survey

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves ("ground roll"). The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones spread axis. Conversely, the ESPAC is considered a "passive" method, using the low frequency "noises" produced far away. The method can also be used with "active" seismic source records. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion. The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model.

Seismic Refraction Survey

The method consists in measuring the propagation delays of the direct and refracted seismic waves (P and/or S) produced by an artificial source in the axis of a seismic spread. The seismic velocities of the materials can be directly calculated, then the refractors depths.

More detailed descriptions of the methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

INTERPRETATION METHODS

MASW surveys

The main processing sequence involved data inspection; spectral analysis ("phase shift" for MASW, and cross-correlation for ESPAC); picking the fundamental mode; and 1D inversion of the MASW and ESPAC shot records using the SeisImagerSW™ software. The data inversions used a non-linear least square method.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, localized surface seismic velocities variations, and/or dipping of overburden layers or rock. In general the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

Seismic Refraction surveys

The considered seismic wave's arrival times were identified for each geophone. The General Reciprocal Method was used, with signal sources at both ends of the seismic spread, in order to consider seismic wave propagation for two opposite directions. The measurements were realised to calculate the rock depth, and its seismic velocity. Conversely to the MASW method, the seismic rock velocity measured by seismic refraction is only representative of its superior part, due to the evanescent nature of the refracted wave.

Survey Design

Seismic soundings were realised on the south-west parking of the Pebb Building, located approximately 30 metres east of the 1335 Bank Street site, due to the available free space necessary for the surveys. The geophone spacing for the main spread was of 3 metres, which means that the total length of a 24 geophones spread was of 69 metres. A second shorter seismic spread, with geophone spacing of 1 metre, was dedicated to the near surface materials.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW, and 50 µs for the seismic refraction method. The records included a pre-trig 10 ms portion. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

Unlike the refraction method, which allows producing a result point beneath each geophone, the shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length. The seismic records were realized with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. An 18 pounds sledgehammer was used as the primary energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

From the GRM seismic refraction, a dense material (V_P = 2390 m/s) was calculated approximately 3.2 meters deep, and the rock (V_P = 3315 m/s) average depth was calculated approximately 6 metres deep (4.8 to 6.9 metres), ±1 metre. Seismic Refraction Tomography presented a possible dense material near to 2.7 metres deep, and the average rock depth could be approximately to 5.8 metres deep (4.8 to 6.5 metres), ±1 metre. It should be noted that the investigated topographic surface was close to 1 meter below the Site topographic surface. Considering the calculated rock V_P value, and a reasonable Poisson ratio between 0.30 and 0.25, the corresponding expected V_S value could be from 1775 and 1915 m/s.

The rock depth was reported by the boreholes reports for the site of interest (**exp**, December 2013 and September 2016), between 6.6 to 7.5 metres deep.

The MASW V_S calculated results are illustrated at Figure 5, and they are also presented at Table 1.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface up to 30 metres. This value reflects an equivalent homogeneous single layer response.

The calculated $\overline{V}_{S30}$ value is 874.5 m/s (cf. Table 1), corresponding to the Site Class “B”. Considering general empirical relations involving V_S and N, an empirical value of 159 m/s could be assigned to the extra 1 metre of soil (most probably fill materials) for the site of interest. In such case the $\overline{V}_{S30}$* value would be around 845 m/s, leading to the same Site Class, even if more conservative. One should therefore note that Site Classes A and B cannot be used if there more than 3 metres of unconsolidated material between the rock and the lower portion of the mat foundations.

Considering the lower portion of the mat foundation being 3 metres above the rock, the $\overline{V}_{S30}$* value would be 1679.7 m/s, corresponding to the Site Class “A”.

CONCLUSION

Seismic surveys were carried out with the MASW, ESPAC analysis methods, and the seismic refraction method, to calculate the $\overline{V}_{S30}$ value for the Site Class determination. The seismic spreads were laid on the south-west parking of the Pebb Building, located approximately 30 metres adjacent east of the 1335 Bank Street site. The $\overline{V}_{S30}$ calculation is presented in Table 1.

The calculated $\overline{V}_{S30}$ value of the actual adjoined site is 875 m/s, while it's $\overline{V}_{S30}{}^{*}$ value (considering a 1 metre of extra fill materials) could be 845 m/s, both corresponding to a Site Class "B" (cf. Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12). As observed from the boreholes reports and as calculated by seismic refraction, there should be approximately 7 metres of unconsolidated material above the rock. This condition does not allow using the Site Class "B", exceeding the 3 metres limit. The Site Class "C" should be used in such case.

Considering the case the lower portion of the mat foundation could be 3 metres above the rock, the $\overline{V}_{S30}{}^{*}$ value would be 1680 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30}{}^{*}$ > 1500 m/s).

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, soft clays, high moisture content etc. can supersede the Site Classification provided in this report based on the $\overline{V}_{S30}$ values.

The V_S values calculated are representative of the in situ materials, and are not corrected for the total and effective stresses.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Project Manager

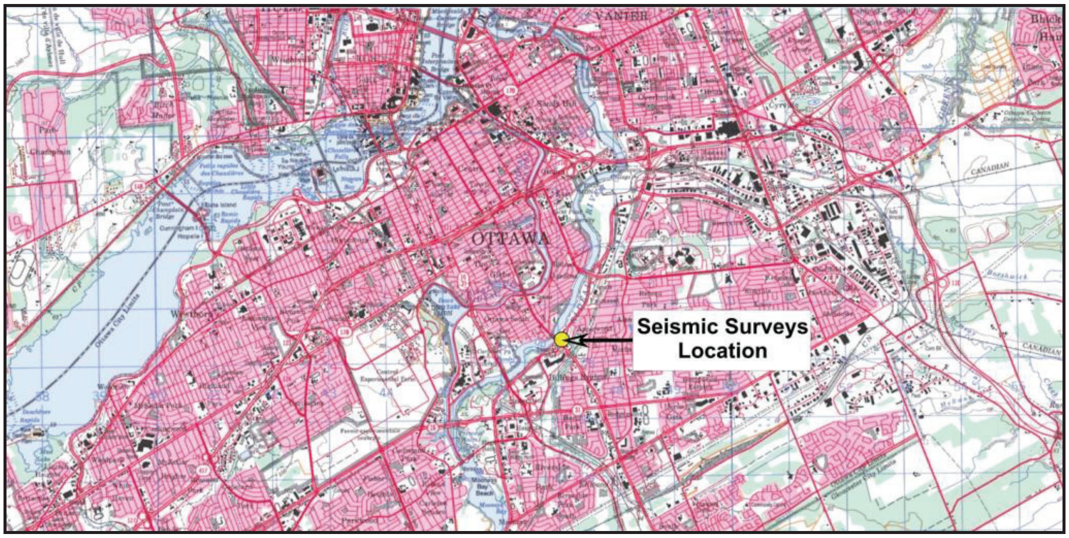


Figure 1: Regional location of the Site
(source: topographic sheet 31 G/05)

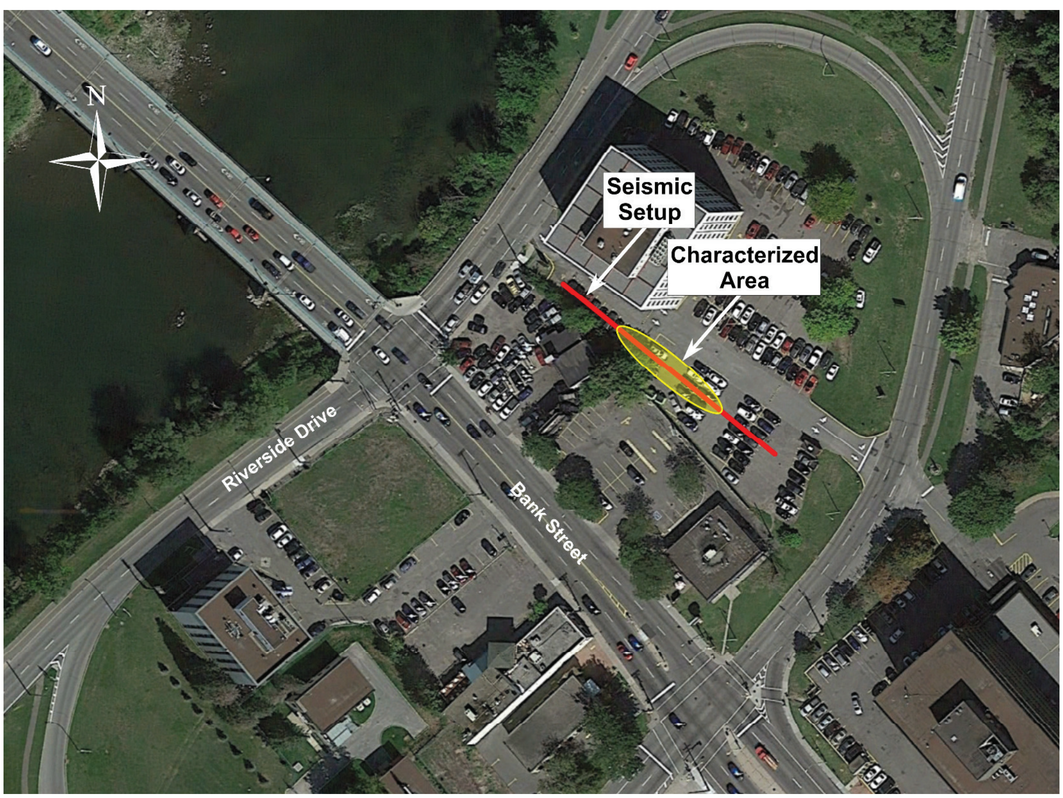


Figure 2: Location of the seismic spreads
(source : *Google Earth*™)

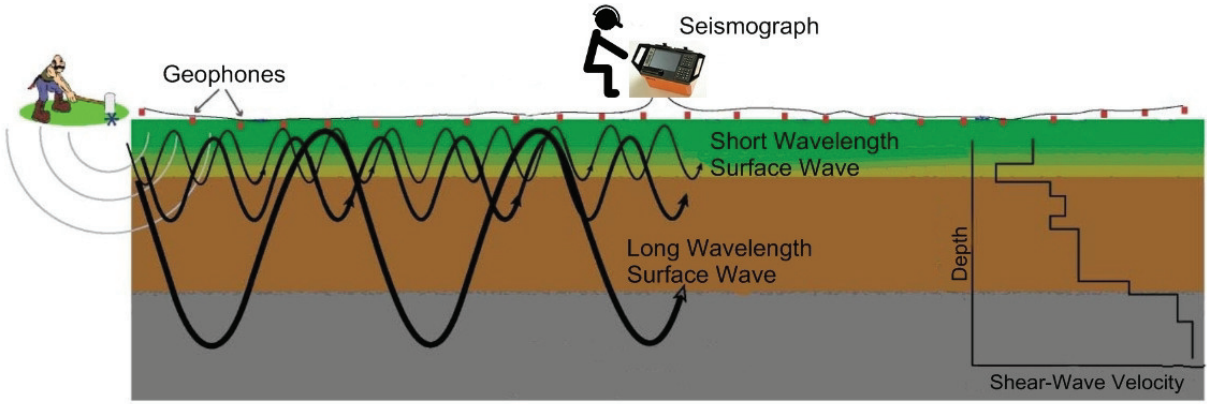


Figure 3: MASW Operating Principle

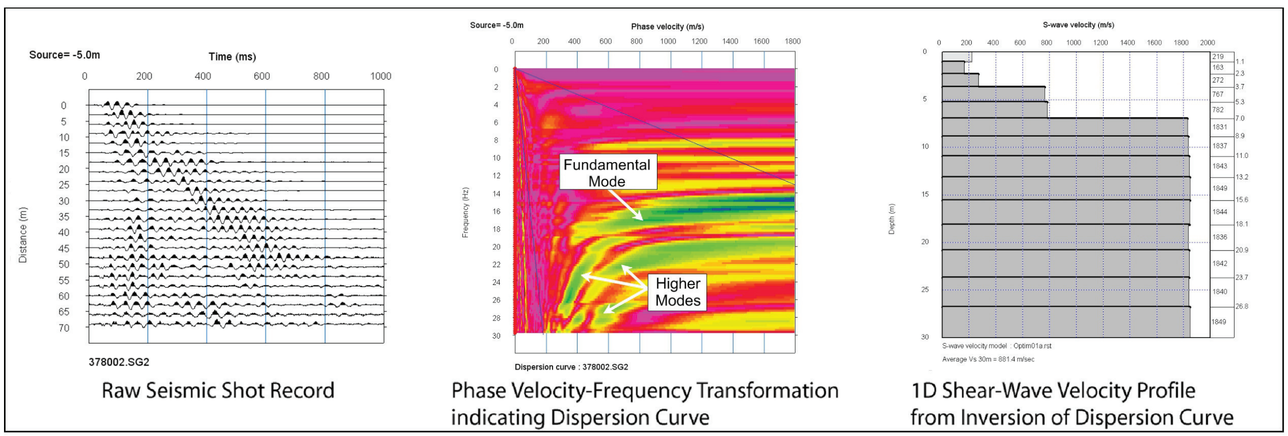


Figure 4: Example of a MASW/ESPAC record, Phase Velocity - Frequency curve and resulting 1D Shear Wave Velocity Model

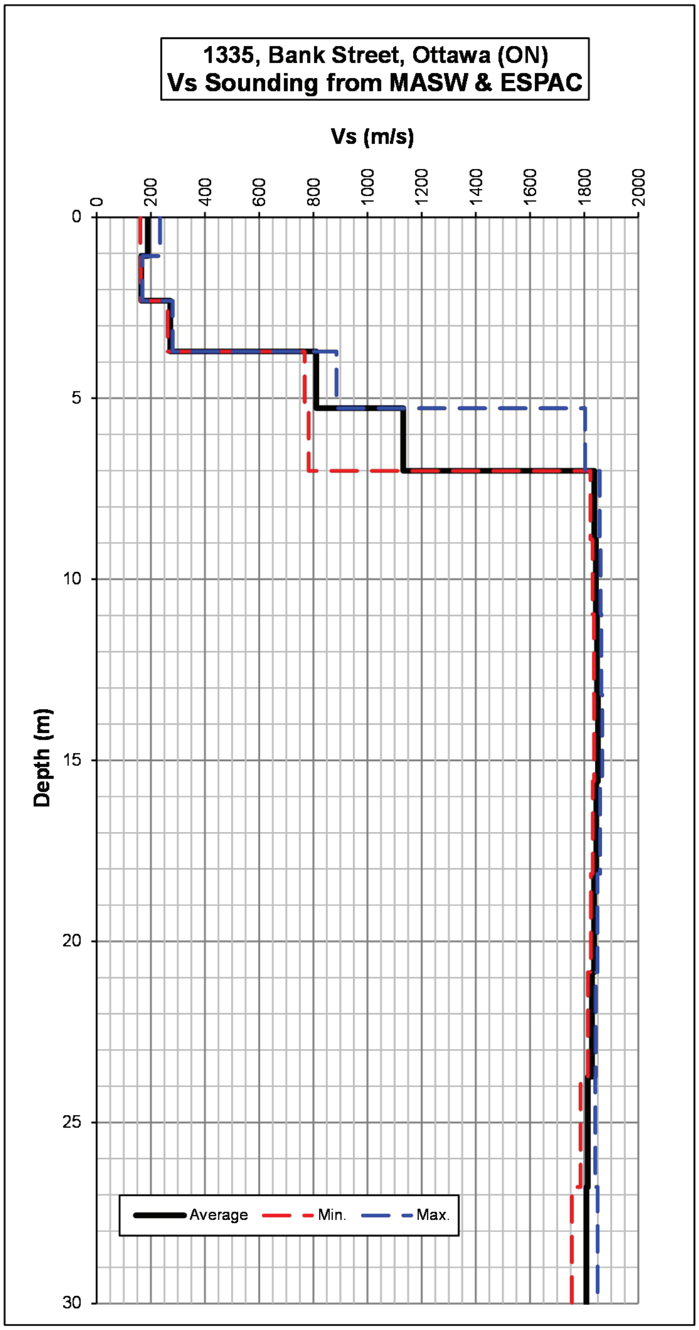


Figure 5: MASW Shear-Wave Velocity Soundings

TABLE 1

V_{S30} Calculation for the Site Class

Depth	Vs			Thickness	Cumulated Thickness	Delay for Avg. Vs	Cumulated Delay	Average Vs for Given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	***161.6***	188.8	234.3					
1.07	***163.0***	***165.4***	168.8	1.07	1.07	0.005674	0.005674	188.8
2.31	262.1	272.2	280.8	1.24	2.31	0.007475	0.013149	175.5
3.71	767.7	811.3	885.4	1.40	3.71	0.005148	0.018297	202.7
5.27	782.9	1132.1	1804.0	1.57	5.27	0.001930	0.020227	260.8
7.01	1823.4	1837.4	1857.6	1.73	7.01	0.001529	0.021756	322.0
8.90	1830.7	1842.9	1861.0	1.90	8.90	0.001032	0.022788	390.6
10.96	1836.3	1847.6	1863.5	2.06	10.96	0.001118	0.023906	458.5
13.19	1836.9	1851.2	1866.8	2.23	13.19	0.001204	0.025110	525.2
15.58	1831.6	1845.1	1859.0	2.39	15.58	0.001291	0.026401	590.0
18.13	1824.5	1836.4	1848.8	2.55	18.13	0.001385	0.027786	652.6
20.85	1813.7	1828.7	1842.7	2.72	20.85	0.001481	0.029267	712.5
23.74	1786.6	1813.8	1841.0	2.88	23.74	0.001577	0.030845	769.5
26.79	1754.3	1808.0	1849.4	3.05	26.79	0.001681	0.032526	823.5
30	1754.3	1808.0	1849.4	3.21	30.00	0.001778	0.034304	874.5

V_{S30} (m/s) =	**874.5**
Site Class :	**B ***

TABLE 2

V_{S30}* Calculation for the Site Class (mat foundation 3 m above the rock)

Depth	Vs			Thickness	Cumulated Thickness	Delay for Avg. Vs	Cumulated Delay	Avg. Vs at given depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	161.6	188.8	234.3	**Considering the lower portion of the mat foundation 3 metres above the rock**				
1.07	163.0	165.4	168.8					
2.31	262.1	272.2	280.8					
3.71	767.7	811.3	885.4					
4.0	767.7	811.3	885.4	0.00				
5.27	782.9	1132.1	1804.0	1.27	1.27	0.001571	0.001571	811.3
7.01	1823.4	1837.4	1857.6	1.73	3.01	0.001529	0.003100	969.5
8.90	1830.7	1842.9	1861.0	1.90	4.90	0.001032	0.004132	1186.2
10.96	1836.3	1847.6	1863.5	2.06	6.96	0.001118	0.005250	1326.1
13.19	1836.9	1851.2	1866.8	2.23	9.19	0.001204	0.006454	1423.4
15.58	1831.6	1845.1	1859.0	2.39	11.58	0.001291	0.007745	1494.7
18.13	1824.5	1836.4	1848.8	2.55	14.13	0.001385	0.009130	1547.9
20.85	1813.7	1828.7	1842.7	2.72	16.85	0.001481	0.010611	1588.1
23.74	1786.6	1813.8	1841.0	2.88	19.74	0.001577	0.012188	1619.3
26.79	1754.3	1808.0	1849.4	3.05	22.79	0.001681	0.013870	1642.9
34.0	1754.3	1808.0	1849.4	7.21	30.00	0.003990	0.017860	1679.7

V_{S30}*(m/s) =	**1679.7**
Site Class :	**A**

*: Site Classes A and B are not to be used if there is more than 3 m of unconsolidated material between the rock and the bottom of the spread footing or mat foundation.

2650 Queensview Drive
Ottawa, Ontario, K2B 8H6, CANADA
T: 1.613.688.1899 • www.EXP.com

March 6, 2018

1924324 Ontario Inc.
c/o Mr. Robert Haslett
Haslett Construction
414 Churchill Avenue North
Ottawa, Ontario K2A 1Z9

Via e-mail: rob@haslettconstruction.com

Project Name: 1335 Bank Street, Ottawa, Ontario

Project Number: OTT-00235328-A2

Subject: Site Plan Control Approval Application, 1335 Bank Street, Ottawa, Ontario

Dear Robert:

The purpose of this letter is to address item G(a) of the City of Ottawa letter D07-12-17-0101 dated December 13, 2017 regarding Site Plan Control Approval Application, 1335 Bank Street (2nd Review). Item G(a) relates to the stability of the Rideau River bank and geotechnical considerations related to unstable slopes, as addressed in Council's Slope Stability Guidelines for Development Applications in the City of Ottawa, 2004. The proposed development is located ta the intersection of Bank Street and Riverside Drive South (Figure 1).

To expedite the above requirement, the following work was undertaken:

1.) A cross-section of the river bank and Riverside Drive South located adjacent to and west of the proposed building was surveyed.

2.) The geotechnical conditions in the area were reviewed.

3.) Comments on the stability of the river bank are provided.

Cross-Section Profile

A cross-section of the river bank and Riverside Drive South located adjacent to and west of the proposed building location was undertaken (Figure 2). It revealed that the invert of the Rideau River in the vicinity of the proposed building is at Elev. 55 m approximately. The crest of the slope is at Elev. 60 m with an overall slope inclination of 2.9H:1V approximately. The design high water level in the Rideau River was taken as Elev. 57.75 m as suggested by Rideau Valley Conservation Authority. The proposed building will be located approximately 15 m east of the crest of the slope resulting in an overall slope from the river bed to the building of approximately 6H:1V. Riverside Drive South is located between the crest of the slope and the proposed building.

Geotechnical Conditions

Based on the geotechnical investigations undertaken at the site of the proposed building, the closest borehole drilled to the river bank was Borehole No. 1 of the 1990 geotechnical investigation (Figure 3). A review of the borehole log (Figure 4) indicates that the river bank is expected to comprise of some surficial sand fill, underlain by a thin layer of silty clay beneath which silty sand is expected to extend to the weathered shale bedrock at Elev. 54.6 m approximately.

Slope Stability Conditions

The slope, which is essentially comprised of granular materials, is expected to be stable at an inclination of 2.5H:1V to 3H:1V. As indicated previously, Riverside Drive South is located between the river bank and the proposed building. This roadway has been in place for a long time (more than 30 years) and to the best of our knowledge, there has not been any slope stability related problems with the roadway. Therefore, in our opinion, the river bank is currently stable. The proposed building would be founded below the river bed and as such will not exert any increased stress on the river bank. Therefore, construction of the proposed building will not adversely impact the stability of the river bank.

We trust that the information contained in this letter will be satisfactory for your purposes. Should you have any questions, please contact this office.

Sincerely,

EXP Services Inc.

LICENSED PROFESSIONAL ENGINEER
Mar 6/18
S. K. AGGARWAL
PROVINCE OF ONTARIO

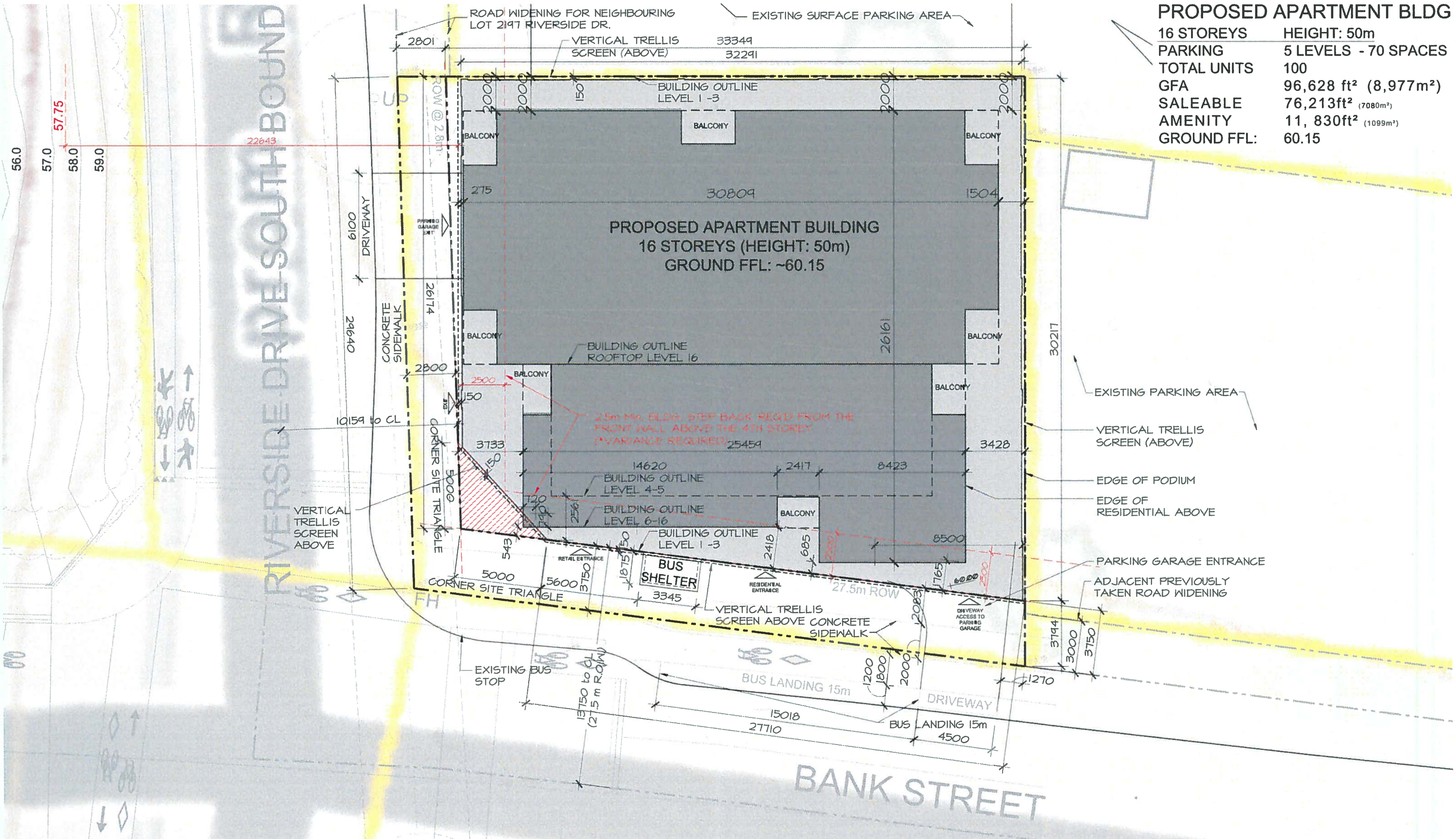
Surinder Aggarwal, M.Sc., P.Eng.
Senior Project Manager, Geotechnical Services
Earth and Environmental

Ismail Taki, M.Eng., P.Eng.
Manager, Geotechnical Services
Earth and Environmental

Attachments:

Figure 1: Site Plan
Figure 2: Rideau River Cross-Section Adjacent to 1335 Bank Street
Figure 3: Borehole Location Plan
Figure 4: Log of Borehole BH 01 (1999)

cc: Christine McCuaig - christine@lloydphillips.com



PROPOSED APARTMENT BLDG	
16 STOREYS	HEIGHT: 50m
PARKING	5 LEVELS - 70 SPACES
TOTAL UNITS	100
GFA	96,628 ft² (8,977m²)
SALEABLE	76,213ft² (7080m²)
AMENITY	11, 830ft² (1099m²)
GROUND FFL:	60.15

1335 BANK STREET

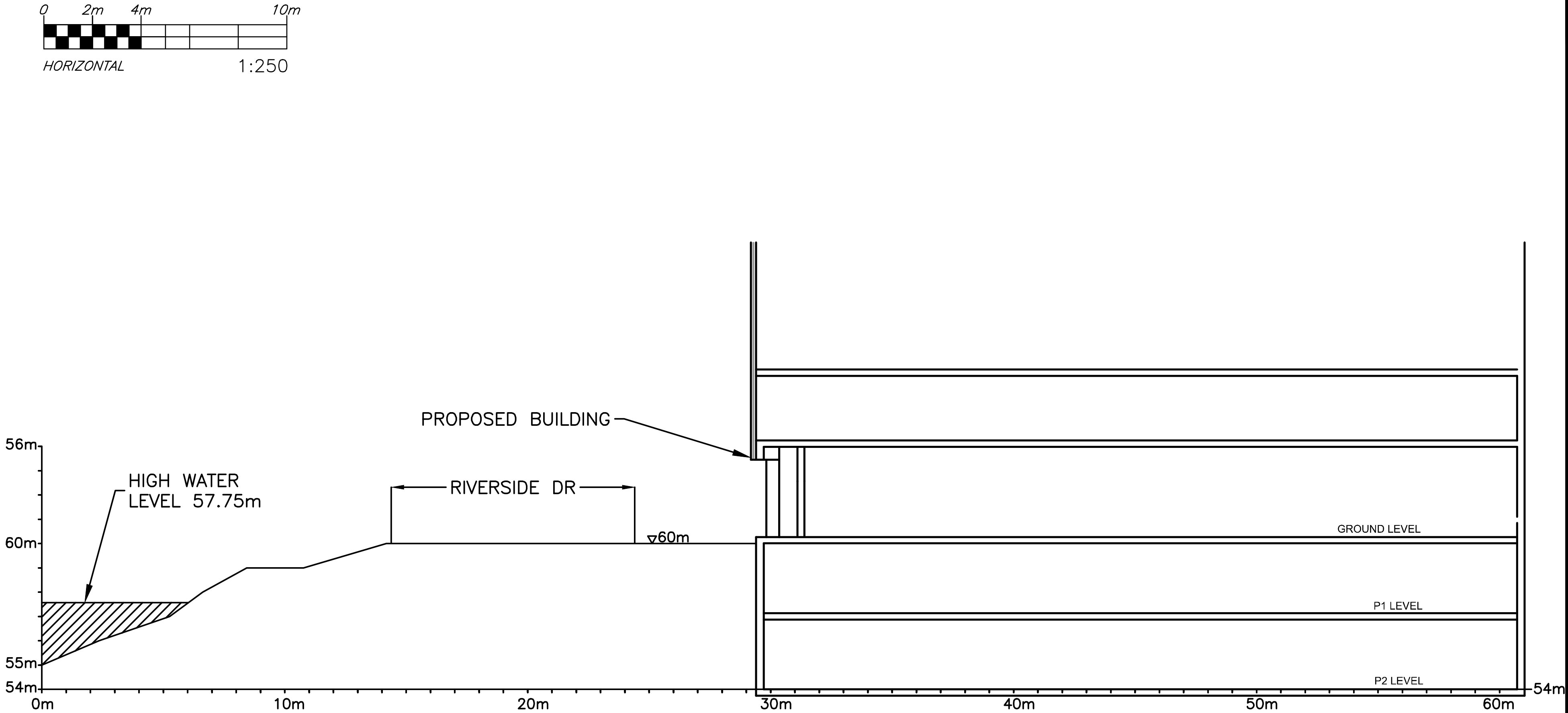
1 9 2 4 3 2 4 ontario inc

SITE PLAN

Feb 2, 2018
1:200

HOBIN

Figure 1



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exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6
www.exp.com

DESIGN	IT
DRAWN	MZG
DATE	2018-02-22
FILE NO	235328

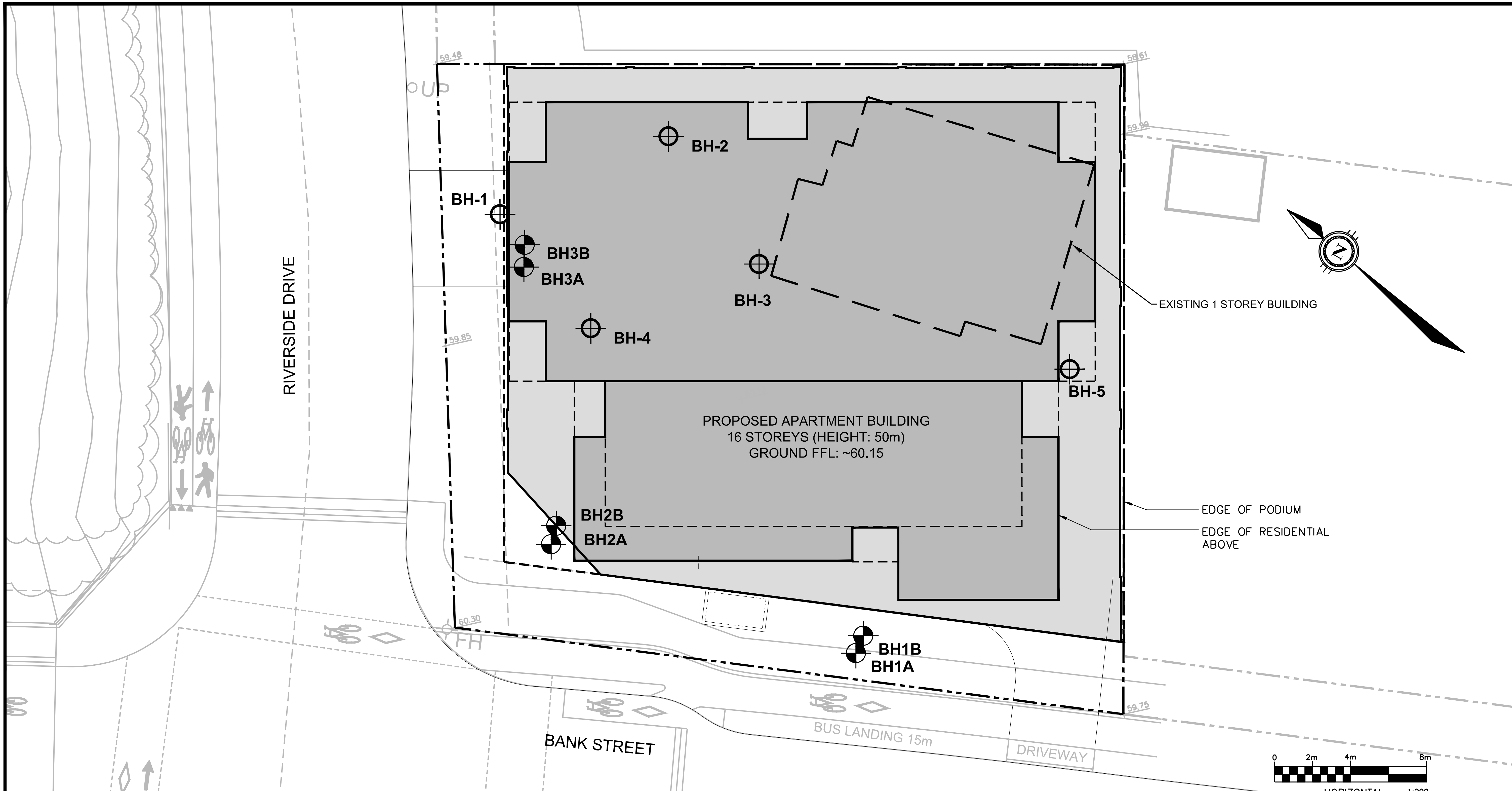
PROPOSED HIGH RISE DELUXE
1335 BANK STREET
OTTAWA, ON

RIDEAU RIVER CROSS SECTION ADJACENT TO PROPOSED 1335 BANK ST

SCALE 1:250

SKETCH NO

FIG 2



NOTES :

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE DRAWING OBTAINED FROM HOBIN ARCHITECTURE INCORPORATED, SITE PLAN DATED MAY 25, 2017

LEGEND

BH1	BOREHOLE NUMBER AND LOCATION
BH-1	BOREHOLE NUMBER AND LOCATION BY OMM TROW (**exp** SERVICES INC.), PROJECT NO. MA13371A IN 1999

exp Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

scale: 1:200	CLIENT: 1924324 ONTARIO INC.	project no.: OTT-00235328-A2
design: S.A. checked: I.T.		date: JULY 2017
drawn by: M.N.	TITLE: BOREHOLE LOCATION PLAN 1335 BANK STREET, OTTAWA, ONTARIO	drawing no.: FIG 3

Filename: r:\230000\235000\235328 1335 bank st\geotech\235328- fig 1 -2 bh plan new base .dwg
Last Saved: 7/7/2017 9:59:17 AM Plotted by: NugentM
Last Plotted: 7/7/2017 10:20:15 AM Pen Table:: trow standard, july 01, 2004.ctb

Log of Borehole BH 01 (1999)

Project No: OTT-000235328-A2 - (Prev MA13771)

Figure No. 4

Project: Phase II Environmental Site Assessment

Page. 1 of 1

Location: Proposed 16 storey Mixed Use Building, 1335 Bank Street, Ottawa, ON

Date Drilled: 'July 12, 1999

Drill Type: CME 55 Truck mount

Datum: Geodetic

Logged by: ______ Checked by: ______

Legend		Legend	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Geodetic (m)	Depth (m)	Soil Description	SPT N Value	Combustible Vapour Reading (ppm)
59.9 / 59.9	0	**ASPHALTIC CONCRETE** ~50 mm		10
59.6		**GRANULAR FILL** Grey, moist, (loose)	50	12
		SAND FILL Medium to coarse grained, some gravel, occasional asphalt, wood and brick pieces in the upper levels, brown, moist, (compact to loose)	42	12
			5	32
			15	40
			7	30
56.1 / 55.9		**SILTY CLAY TO CLAY AND SILT** Black staining, grey, wet, (very stiff)	18	8000
55.7 (water level)		**SILTY SAND** Fine to medium grained, some gravel, black staining in upper levels, occasional cobbles and boulders, grey, wet, (dense)	46	200
54.6		**SHALE TILL TO WEATHERED SHALE BEDROCK** Occasional sandy seems, grey, wet, (dense)	40	98
			10	50
53.0				

Column headings: GWL; SYMBOL; SOIL DESCRIPTION; Geodetic m; Depth; Standard Penetration Test N Value (20, 40, 60, 80); Shear Strength kPa (50, 100, 150, 200); Combustible Vapour Reading (ppm) (250, 500, 750); Natural Moisture Content %, Atterberg Limits (% Dry Weight) (20, 40, 60); SAMPLES; Natural Unit Wt. kN/m³

Refusal to Augering at 6.9 m depth

Additional Notes

1. A strong Hydrocarbon odour was detected in the soil samples retrieved from below 3.8 m depth of this borehole.
2. Vapour reading conducted on the soil samples using a Trace-Techtor Vapour Analyser calibrated for Hexane. All readings are in parts per million

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A 50 mm monitoring well was installed in the borehole
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-000235328-AO- (Prev MA13771)

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.6	
3 days	4.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5044-1 - TEST HOLE LOCATION PLAN

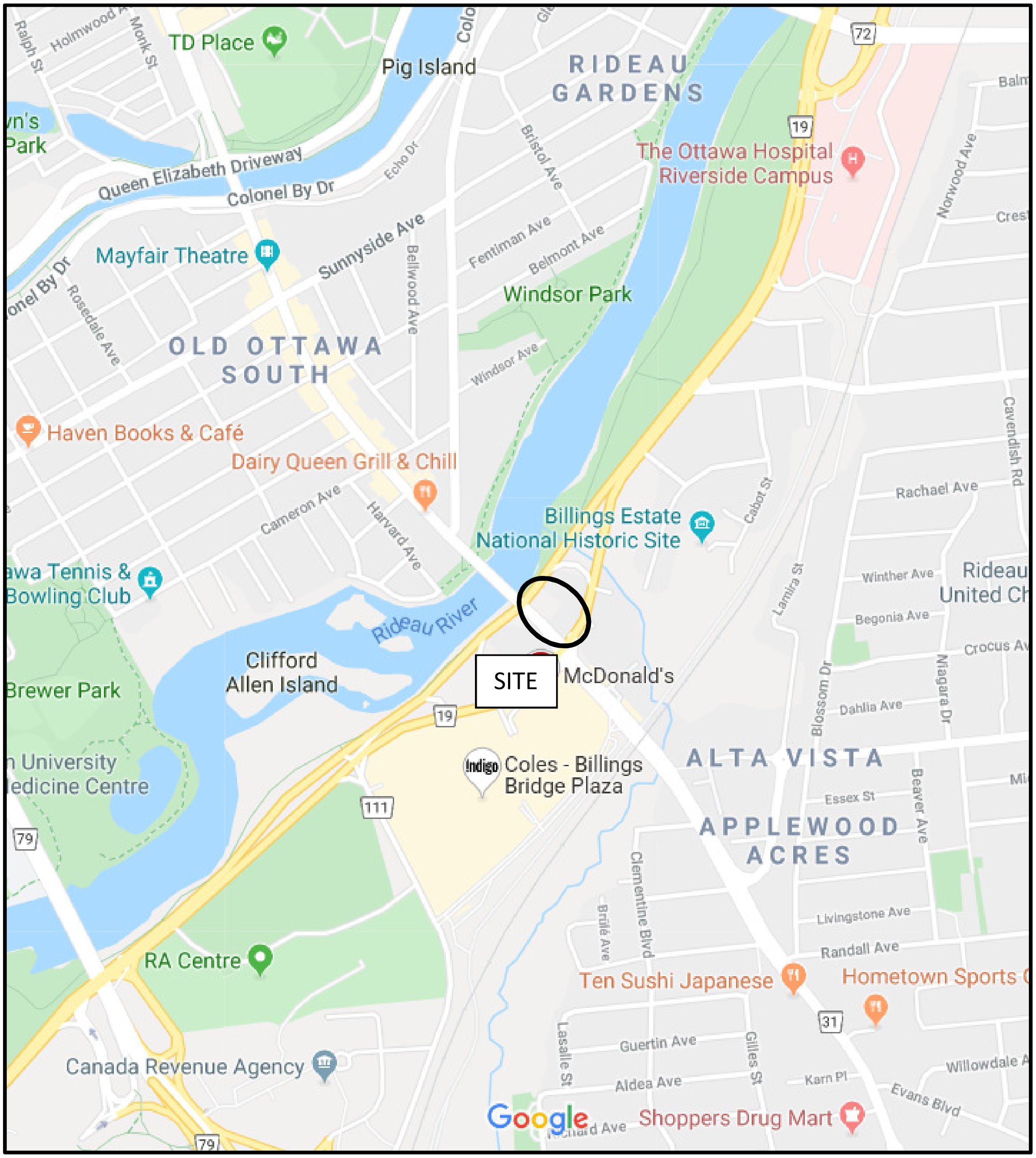


FIGURE 1

KEY PLAN

patersongroup

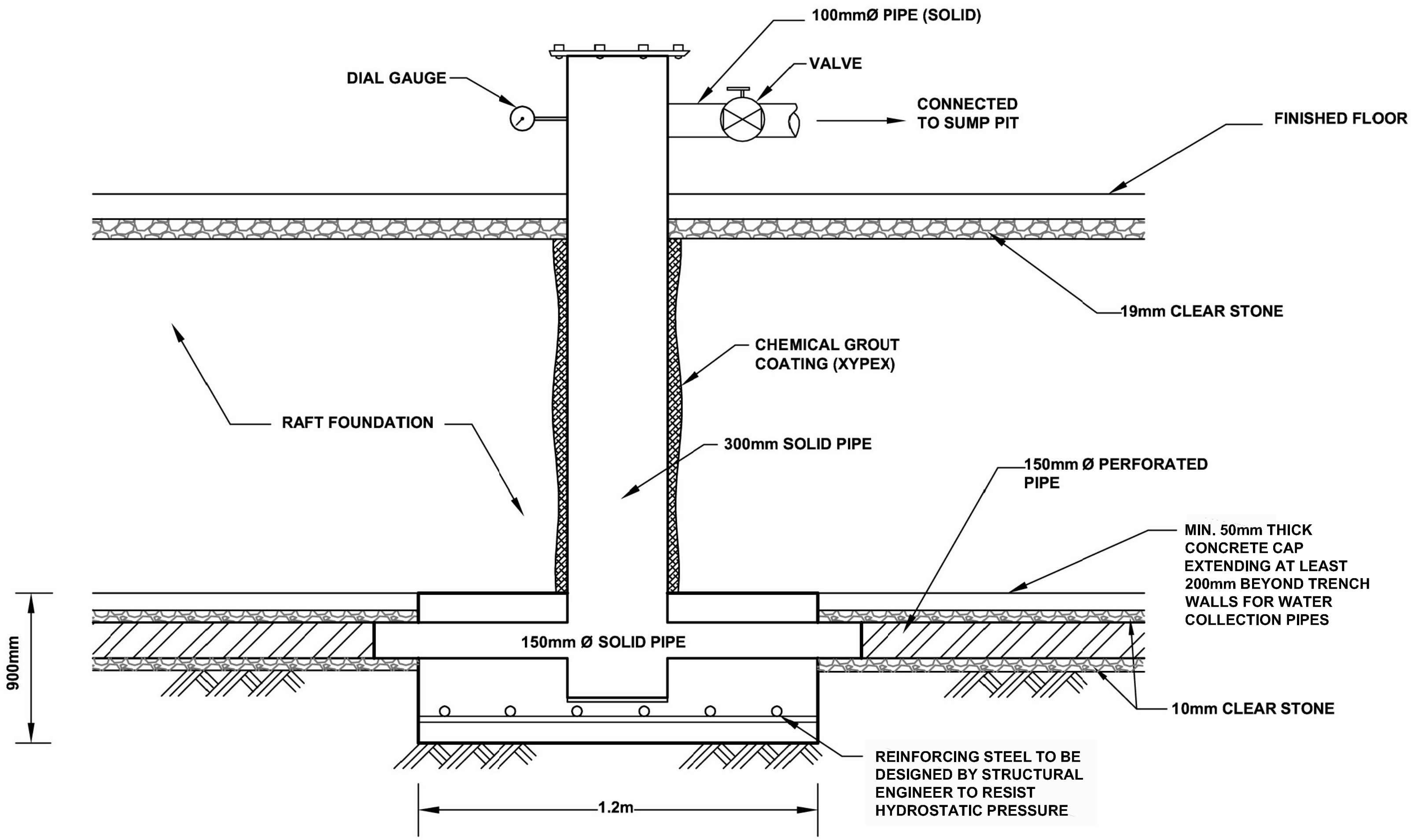


FIGURE 2 - PRESSURE RELIEF CHAMBER

patersongroup
consulting engineers

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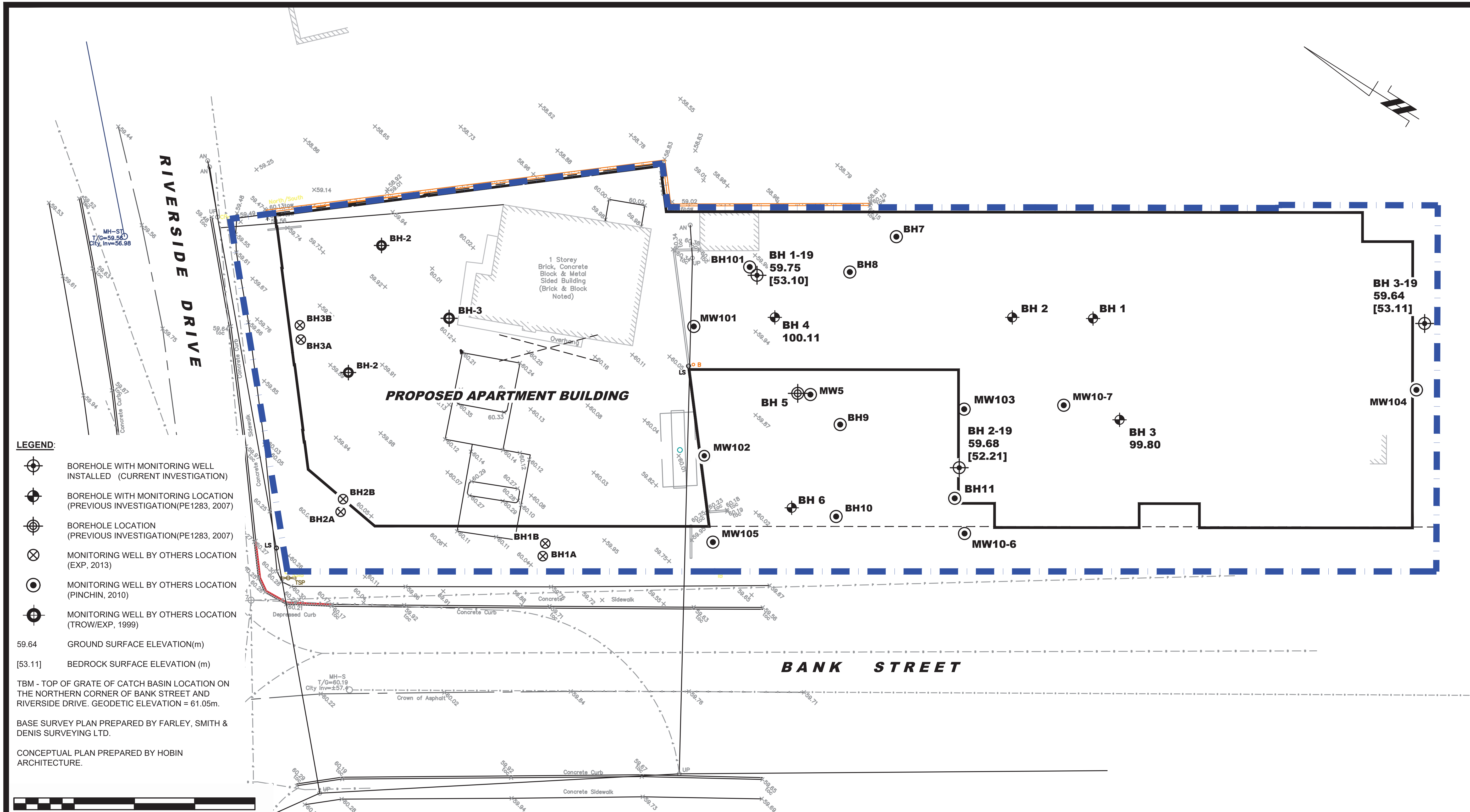

LEGEND:

(symbol)	BOREHOLE WITH MONITORING WELL INSTALLED (CURRENT INVESTIGATION)
(symbol)	BOREHOLE WITH MONITORING LOCATION (PREVIOUS INVESTIGATION(PE1283, 2007)
(symbol)	BOREHOLE LOCATION (PREVIOUS INVESTIGATION(PE1283, 2007)
(symbol)	MONITORING WELL BY OTHERS LOCATION (EXP, 2013)
(symbol)	MONITORING WELL BY OTHERS LOCATION (PINCHIN, 2010)
(symbol)	MONITORING WELL BY OTHERS LOCATION (TROW/EXP, 1999)
59.64	GROUND SURFACE ELEVATION(m)
[53.11]	BEDROCK SURFACE ELEVATION (m)

TBM - TOP OF GRATE OF CATCH BASIN LOCATION ON THE NORTHERN CORNER OF BANK STREET AND RIVERSIDE DRIVE. GEODETIC ELEVATION = 61.05m.

BASE SURVEY PLAN PREPARED BY FARLEY, SMITH & DENIS SURVEYING LTD.

CONCEPTUAL PLAN PREPARED BY HOBIN ARCHITECTURE.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

BOULET CONSTRUCTION

GEOTECHNICAL INVESTIGATION - PROPOSED MULTI-STOREY BUILDING
1335 & 1339 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 11/2019
Drawn by: RCG	Report No.: PG5044
Checked by: SD	PG5044-1
Approved by: DJG	Revision No.:

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