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Geotechnical Investigation

Proposed Multi-Storey Building
319-327 Richmond Road
381 Churchill Aveue
380 Winona Avenue
Ottawa, Ontario

Prepared For

Richmond Churchill Limited Partnership

June 15, 2020

Report: PG5351-1 Revision 1

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Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Richmond Churchill Limited Partnership to conduct a geotechnical review of the available information for the proposed multi-storey building to be located at 319-327 Richmond Road, 381 Churchill Avenue, and 380 Winona Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

❏ determine the subsoil and groundwater conditions at this site by means of the available geotechnical information.

❏ provide geotechnical recommendations for the design of the proposed development based on the results of the test holes and other geotechnical information available.

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

2.0 Proposed Development

The proposed project will consist of a mix use multi-storey building with two levels of underground parking. It is expected that the development will include associated access roads and landscaped areas. It is also expected that the subject site will be municipally serviced.

The existing structure will be demolished to allow for the construction of the proposed complex.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Various field programs were conducted on the different lots composing the subject site for the aforementioned project. The boreholes were drilled with a truck-mounted drilling equipment.

Paterson carried a field investigation program of the 325 and 327 Richmond Road on May 9, 2012. At that time, 4 boreholes were advanced to a maximum depth of 6.83 m below the existing grade. Paterson also conducted the field program for the investigation of 381 Churchill Avenue North July 21, 2016. At that time, 4 boreholes were advanced to a maximum depth of 7.57 m below the existing grade. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG5351-1 - Test Hole Location Plan in Appendix 2.

A field program was conducted by others for the investigation of 319 Richmond Road on July 25 & 26, 2017. At that time, 5 boreholes were advanced to a maximum depth of 6.71 m below the existing grade. The field investigation program for the investigation of 380 Winona Avenue was carried out by others on January 16, 2020. At that time, 2 boreholes were advanced to a maximum depth of 7.62 m below the existing grade. The borehole locations are shown on Drawing PG5351-1 - Test Hole Location Plan in Appendix 2.

Sampling and In Situ Testing

Soil samples from the borehole were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. The depths at which the auger and split spoon samples were recovered from the test holes are shown on the borehole logs presented in Appendix 1.

Standard Penetration Testing (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the borehole logs. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

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

The subsurface conditions as recorded by others in the field are presented on the borehole logs in Appendix 1 of this report.

3.2 Field Survey

The locations of the test holes was selected to cover areas of potential environmental impact. Ground elevations and borehole locations were surveyed by others. The ground elevations is referenced to the geodetic datum. The location of the test holes and TBM are presented on Drawing PG5351-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined by others to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a residential dwelling to the north (380 Winona Avenue), an automotive service garage in the east (319 Richmond Road), two 2 storey mixed use buildings located in the south of the site (325 & 327 Richmond Road), and an apartment complex located to the west (381 Churchill Avenue North). The total site is surrounded by Churchill Avenue North to the west, Richmond Road to the south, and Winona Avenue to the East. The ground surface of the subject site is relatively flat and at-grade with the surrounding properties.

4.2 Subsurface Profile

Overburden

Based on the results of the geotechnical investigation conducted on the different lots, the subsurface profile encountered at the borehole locations generally consists of a fill layer or asphalt pavement structure, overlying a fill layer consisting predominantly of silty sand with cobble and gravel. Fair to excellent quality grey limestone bedrock was encountered under the fill layer. Practical refusal to augering was encountered at depths ranging from 1.2 - 1.8 m below ground surface.

Reference should be made to the borehole logs by others in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of interbedded limestone and dolomite of the Gull River formation. The overburden thickness is expected to range from 1 to 2 m.

4.3 Groundwater

Groundwater monitoring wells were installed as part of the investigations. Groundwater level measurements were recorded at the monitoring well locations and our findings are presented in Table 1. Long-term groundwater level can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected within the bedrock layer between 4 to 5 m depth. It should also be noted that the groundwater level is subject to seasonal fluctuations, therefore, groundwater could vary at the time of construction.

Table 1 - Groundwater Measurements at Monitoring Well Locations

Test Hole Location	Ground Surface Elevation (m)	GW Level Reading (m)	GW Elevation (m)	Date
Paterson Group PE3842, 2016				
BH1	67.45	5.02	62.43	July 21, 2016
BH2	67.31	5.53	61.78	July 21, 2016
Paterson Group PE2628, 2012				
BH1-12	65.85	2.3	63.55	May 9, 2012
BH2-12	68.04	4.1	63.94	May 9, 2012
Pinchin Report 266791.001, 2020				
MW-1	-	5.44	-	January 23, 2020
MW-2	-	6.67	-	January 23, 2020
Golder Report 1781559, 2017				
17-01	68.22	2.08	66.02	July 27, 2017
14-02	68.27	3.09	65.14	July 27, 2017
17-03	68.40	3.89	64.44	July 27, 2017
17-04	68.64	2.74	65.79	July 27, 2017
17-05	68.00	1.92	66.00	July 27, 2017
Note: Elevation referred to the geodetic datum				

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development from a geotechnical perspective. It is expected that the proposed structure will be founded over conventional shallow footings placed on good to excellent quality limestone bedrock.

Due to the bedrock elevation, it is expected that rock removal will be required for the construction of the two underground parking levels.

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

5.2 Site Grading and Preparation

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting may be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed. A pre-blast or construction survey 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 an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. A minimum of 1 m horizontal bench should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

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

Two parameters determine the recommended vibration limit, 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. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. A pre-construction survey is therefore recommended to minimize the risks of claims during or following the construction of the proposed building.

Stripping Depth

Topsoil and fill, containing deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Fill Placement

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

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

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed compact glacial till bearing surface can be designusing a bearing resistance of **200 kPa** at SLS and **375 kPa** at ULS.

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

Footings placed on a clean, surface sounded bedrock surface or lean concrete in-filled trench extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

Footing placed on a surface sounded bedrock bearing surface will be subjected negligible post-construction total and differential settlements.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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 silty sand or engineered fill above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the foundations considered at this site. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test shall be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

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

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the basement slab. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone for mechanical rooms and storage areas having a concrete slab. For parking areas of the underground parking garage, if an asphaltic concrete pavement structure is considered, refer to the pavement structures presented in Subsection 5.7.

All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compact to at least 98% of the material's SPMDD.

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 dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be incorporated to the 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 calculated with the seismic loading case.

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

Seismic Earth Pressures

The 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}) could 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. The vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

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

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

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

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

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is expected that the building footprint will occupy the entire boundaries of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a waterproofing and secondary drainage system placed over the grinded bedrock face.

A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at 1 m above the observed groundwater level down to the founding level. By waterproofing the vertical excavation side slopes and ensuring that the system continues vertically to the underside of the proposed foundation, it will be possible to lessen the groundwater volumes to discharge post development. This can be accomplished by placing a waterproofing membrane layer against the grinded bedrock face. A composite drainage system should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water breaching the waterproofing membrane system.

It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower level area.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greaterpart of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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 along the interior perimeter of the foundation wall and one drainage line within each bay. The spacing of the underfloor drainage system should be confirmed at the time of backfilling the floor completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

It is understood that 2 underground parking levels are planned for the proposed development, with the lower portion of the foundation having a groundwater infiltration control system in place. The existing buildings along the north portion are expected to be founded over bedrock or within the glacial till above the bedrock surface. Based on field observations and assessment, the groundwater level is anticipated at a 4 to 5 m depth below existing grade. A local groundwater lowering is expected under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal groundwater lowering. It should also be noted that the lower portion of the foundation walls will be waterproofed which will limit groundwater lowering within the subject site and surroundings. Since the neighbouring structures are founded within native glacial till or directly over a bedrock bearing surface based on available soils information. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

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 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance 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 may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. 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 design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

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

Generally, the shoring systems should be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 4 - 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 calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, a minimum factor of safety of 1.5 should be used.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 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 and compacted to 95% of the material's SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the shallow excavation. 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.

For typical ground or surface water volumes being pumped during the construction phase, ranging between 50,000 to 400,000 L/day, there is a requirement to register on the Environmental Activity and Sector Registry (EASR). A minimum of 2 to 4 weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application. Based on the current design details, it is expected that an EASR permit would be sufficient for the anticipated water pumping requirements during construction.

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 5.1. During steady state conditions, it's expected that groundwater flow will be low to moderate (less than 10,000 L/day). A more accurate estimate can be provided at the time of construction.

6.6 Winter Construction

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

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

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

7.0 Recommendations

For the foundation design data provided herein to be applicable, a materials testing and observation services program is required to be completed. The following aspects should be performed by the geotechnical consultant:

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

❏ Sampling and testing of the concrete and fill materials.

❏ Review of the vertical rock face during excavation.

❏ Observation of the placement of the foundation insulation, if applicable.

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

❏ Periodic inspection of the drainage system and waterproofing installation.

❏ Observation of all subgrades prior to backfilling.

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

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

A report confirming the construction has been conducted in general accordance with the 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 made in this report are in accordance with our present understanding of the project. A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Richmond Churchill Limited Partnership Group or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 15, 2020
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Joey R. Villeneuve, M.A.Sc., P.Eng.

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Report Distribution

☐ Richmond Churchill Limited Partnership Group
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

Borehole Logs by Others

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
325-331 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

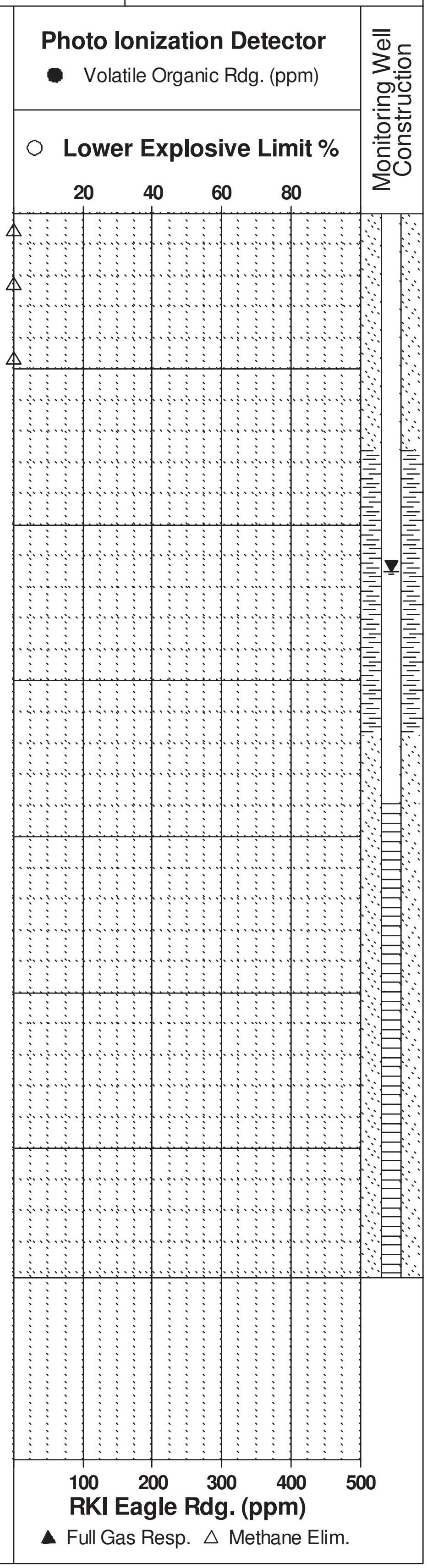
BORINGS BY CME 55 Power Auger

DATE May 9, 2012

FILE NO. PE2628

HOLE NO. BH 1

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Sand and gravel	0.20	AU	1				
FILL: Brown silty sand with clay, gravel, brick, cobbles		AU	2				
		SS	3	92		1	
	1.80	SS	4	0	50+		
BEDROCK: Grey limestone		RC	1	90	31	2	
		RC	2	100	100	3	
		RC	3	100	100	4	
						5	
	6.83	RC	4	76	73	6	
End of Borehole							
(GWL @ 2.3m-May 17, 2012)							



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
325-331 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

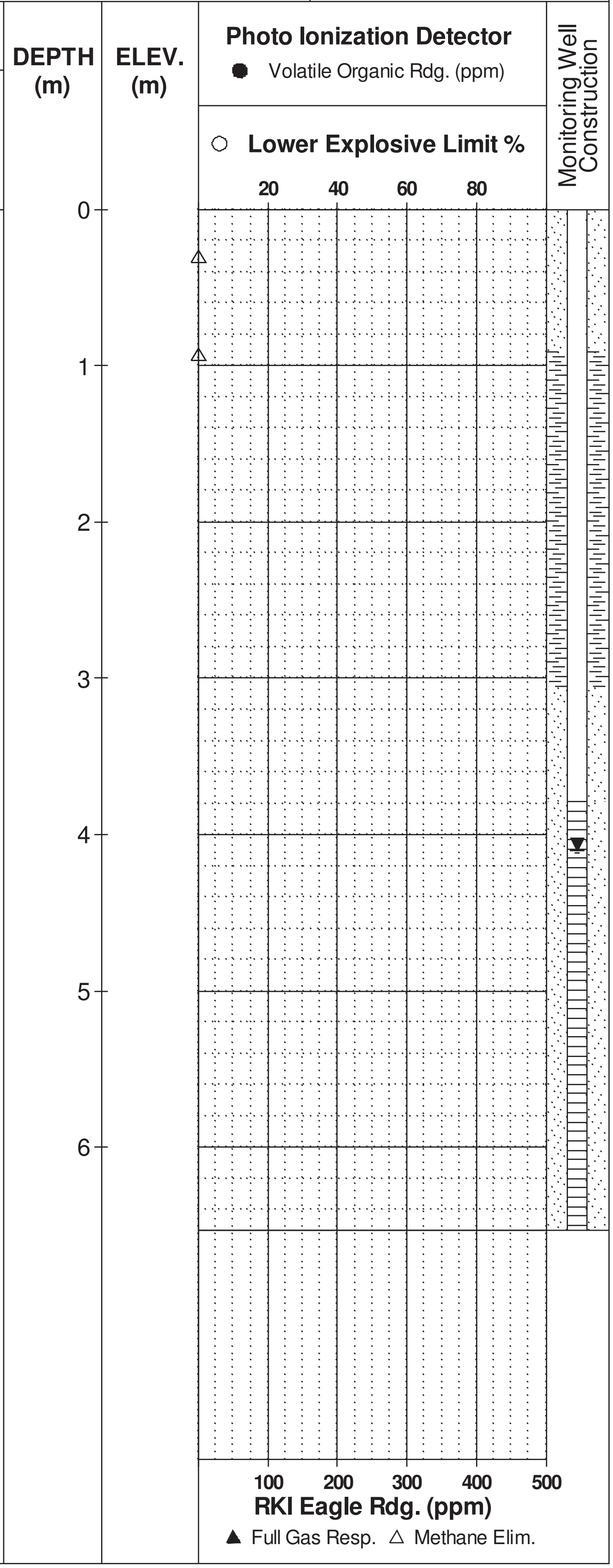
BORINGS BY CME 55 Power Auger

DATE May 9, 2012

FILE NO. PE2628

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Crushed stone with sand	AU	1		
0.60				
FILL: Brown silty sand with gravel, trace clay	SS	2		
1.24				
BEDROCK: Grey limestone	RC	1		0
	RC	2	100	0
	RC	3	100	83
	RC	4	100	100
	RC	5	100	100
6.53				
End of Borehole				
(GWL @ 4.1m-May 17, 2012)				



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
325-331 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

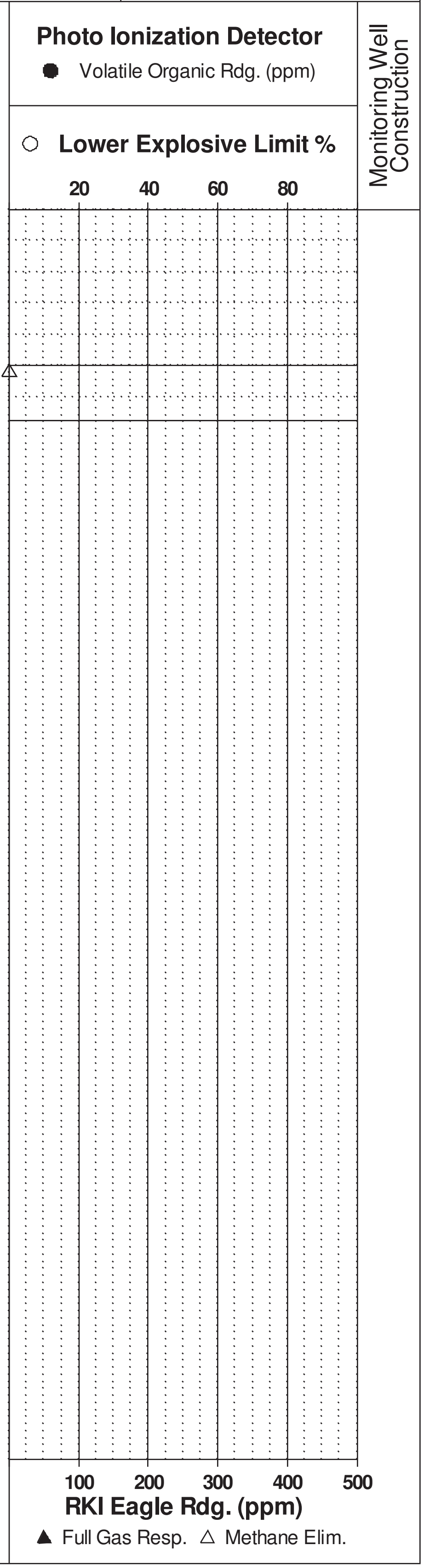
BORINGS BY CME 55 Power Auger

DATE May 9, 2012

FILE NO. PE2628

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Crushed stone	▨						
0.69							
FILL: Brown silty sand with gravel, cobbles	▨	RC	1	67	19	1	
1.35							
End of Borehole Practical refusal to augering @ 1.35m depth							



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
325-331 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

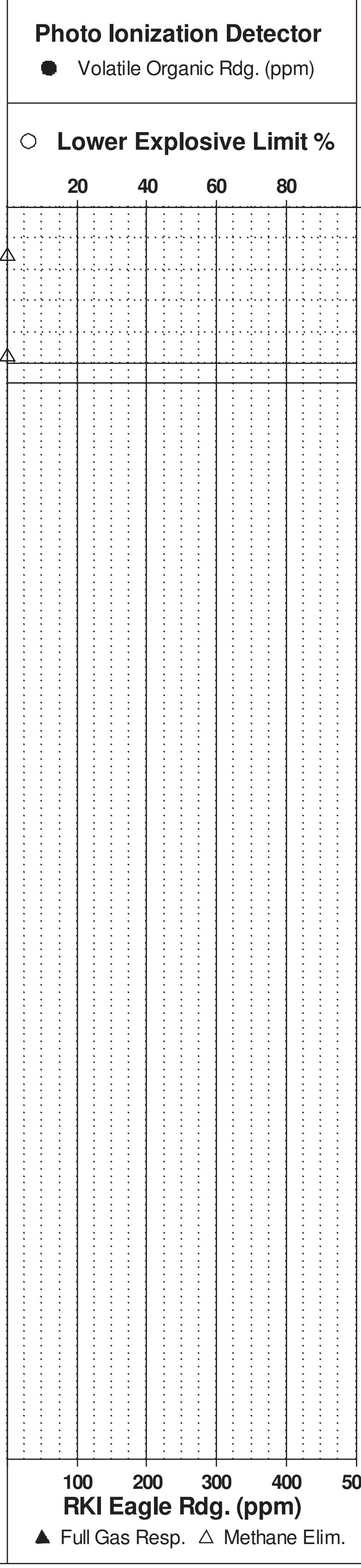
BORINGS BY CME 55 Power Auger

DATE May 9, 2012

FILE NO. PE2628

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0		
FILL: Crushed stone 0.60	▨	AU	1					
FILL: Brown silty sand with gravel, cobbles 1.12	▨	SS	2		50+	1		
End of Borehole Practical refusal to augering @ 1.12m depth								



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
381 Churchill Avenue North
Ottawa, Ontario

DATUM Geodetic

FILE NO. PE3842

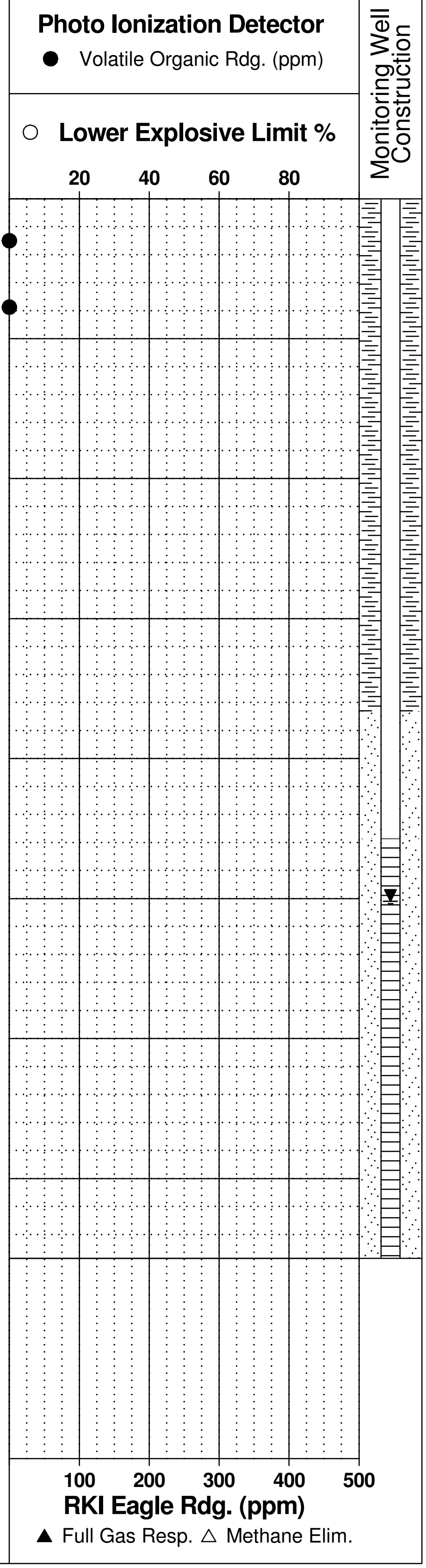
REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger

DATE July 21, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.45
Asphaltic concrete 0.05							
FILL: Crushed stone 0.79		AU	1				
		SS	2	25	50+		
		RC	1	77	18	1	66.45
						2	65.45
		RC	2	100	51		
						3	64.45
		RC	3	99	93		
BEDROCK: Grey limestone, some mud seams						4	63.45
						5	62.45
		RC	4	93	64		
						6	61.45
		RC	5	100	100		
						7	60.45
7.57							
End of Borehole							
(GWL @ 5.02m-July 25, 2016)							



SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
381 Churchill Avenue North
Ottawa, Ontario

DATUM Geodetic

FILE NO. PE3842

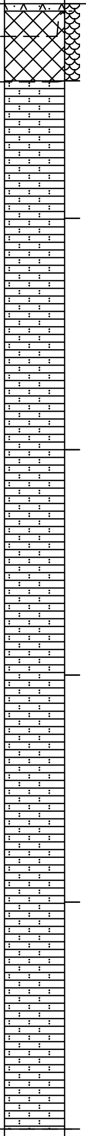
REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE July 21, 2016

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone	0.51
BEDROCK: Grey limestone	7.44
End of Borehole	
(GWL @ 5.53m-July 25, 2016)	

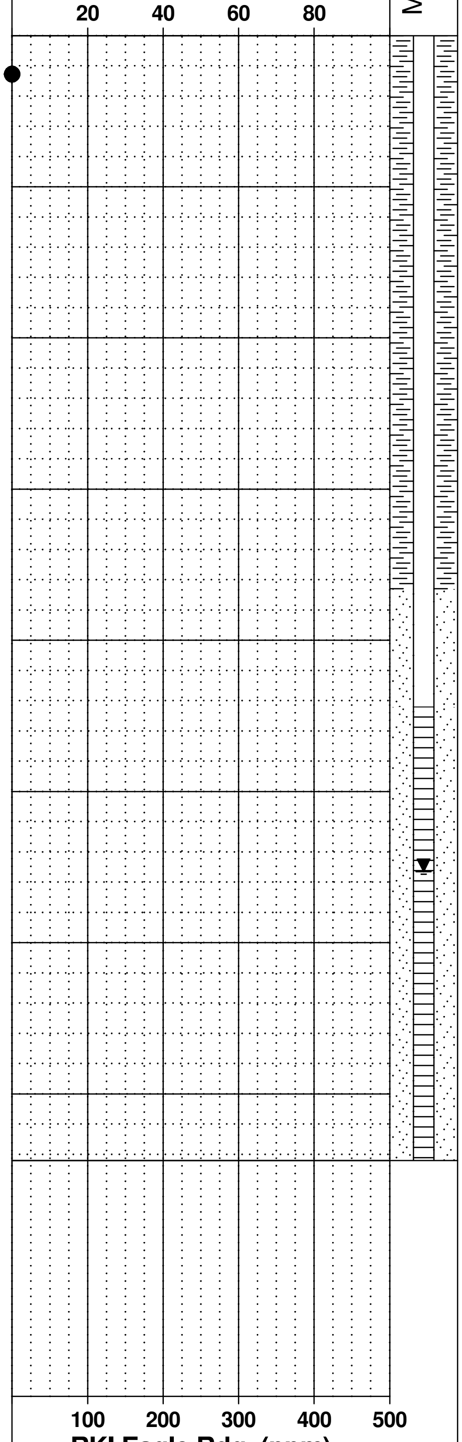


SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
RC	1	84	0
RC	2	95	58
RC	3	97	86
RC	4	100	86
RC	5	98	97

DEPTH (m)	ELEV. (m)
0	67.31
1	66.31
2	65.31
3	64.31
4	63.31
5	62.31
6	61.31
7	60.31

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**



RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
381 Churchill Avenue North
Ottawa, Ontario

DATUM Geodetic

REMARKS

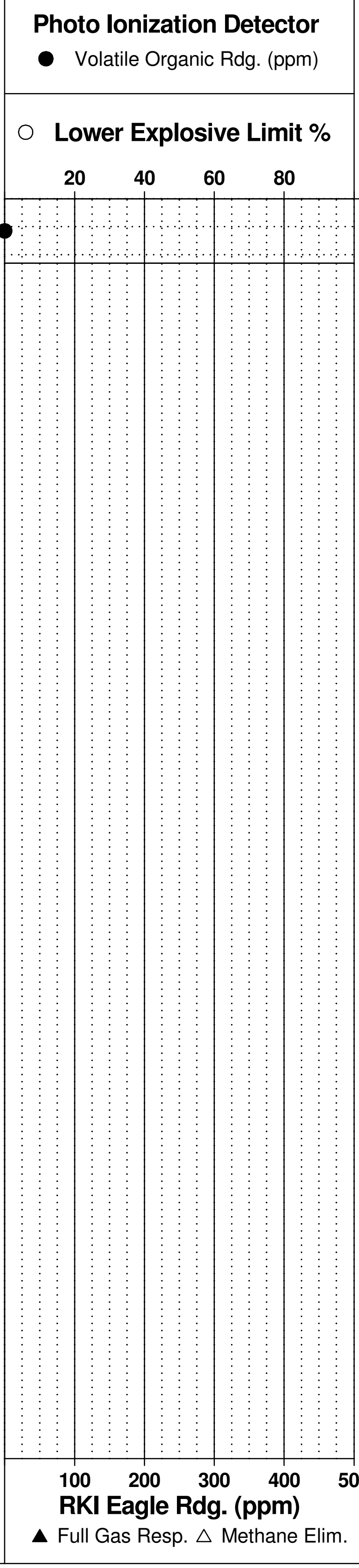
BORINGS BY CME 55 Power Auger

DATE July 21, 2016

FILE NO. PE3842

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0	67.25	
Asphaltic concrete 0.05								
FILL: Silt, some clay, sand and gravel 0.46		AU	1					
End of Borehole Practical refusal to augering at 0.46m depth								



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
381 Churchill Avenue North
Ottawa, Ontario

DATUM Geodetic

FILE NO. PE3842

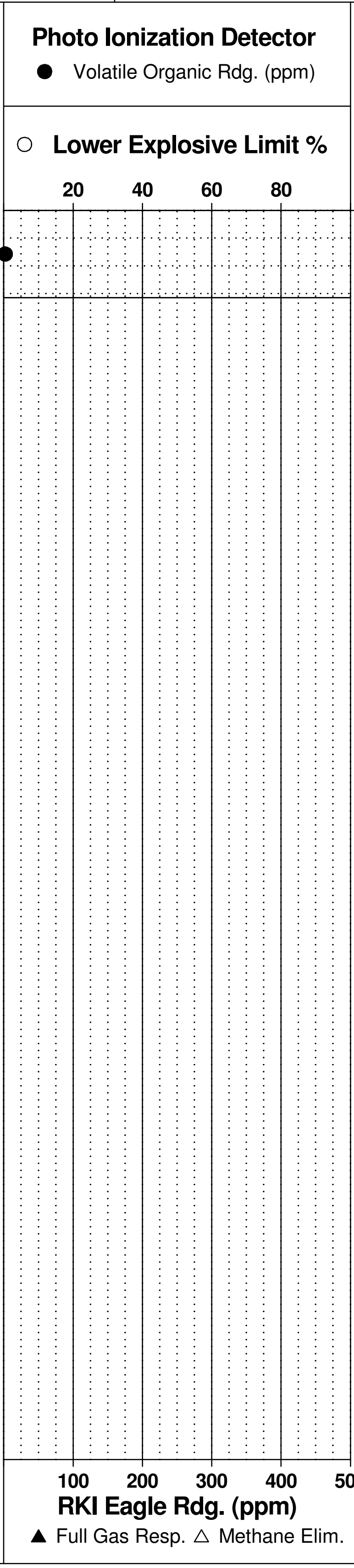
REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger

DATE July 21, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	67.38
Asphaltic concrete 0.05							
FILL: Silt, some clay, sand and gravel 0.63		AU	1				
End of Borehole Practical refusal to augering at 0.63m depth							



Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

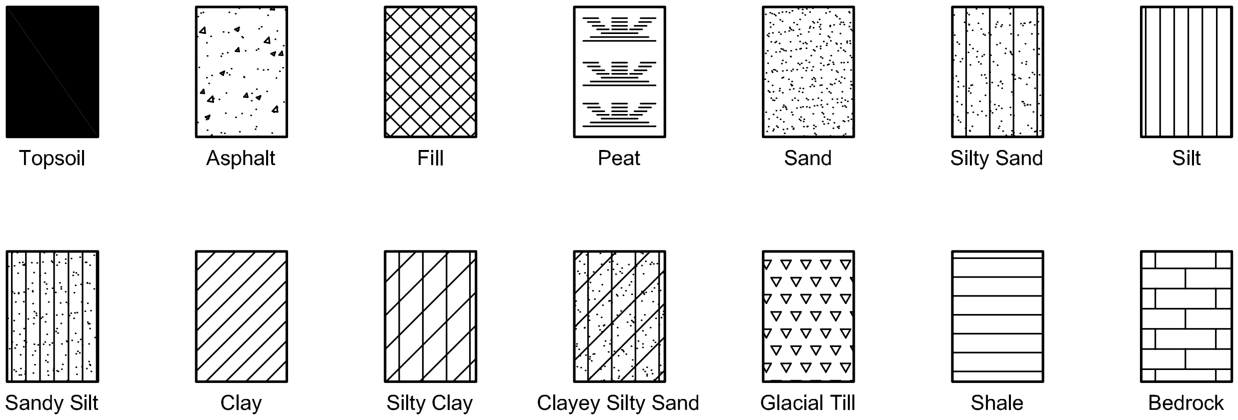
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

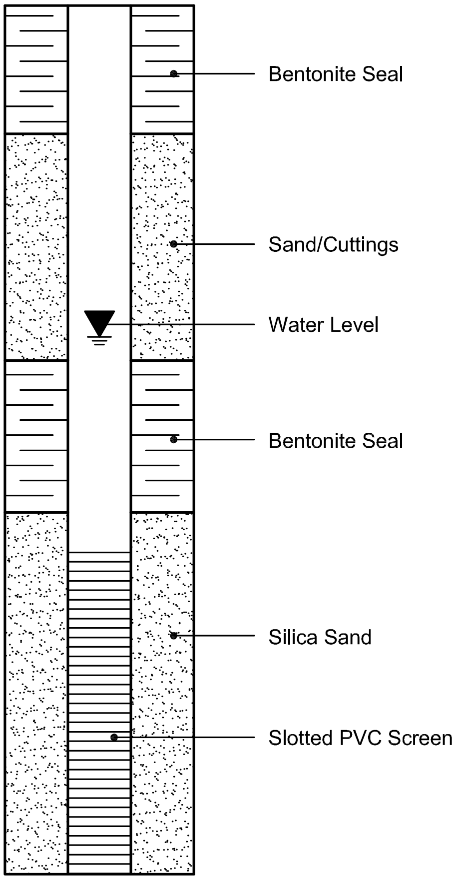
SYMBOLS AND TERMS (continued)

STRATA PLOT

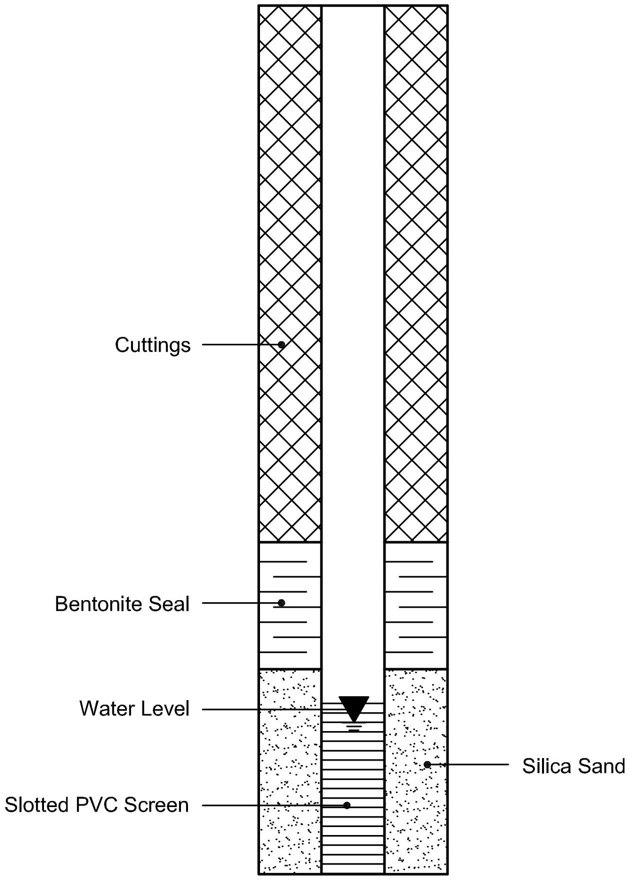


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 1781559

RECORD OF BOREHOLE: 17-01

SHEET 1 OF 1

LOCATION: N 5026844.4 ;E 441042.3

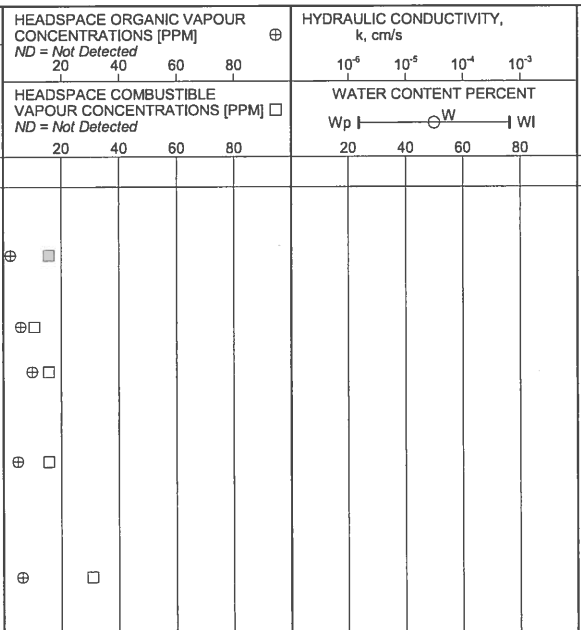
BORING DATE: July 25 & 26, 2017

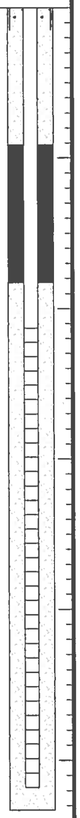
DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	68.22				
0	Power Auger HW Casing	FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)	0.00				Flush Mount Casing
		FILL - (SW) SAND, some silt, angular; grey	67.99 0.23	1	GRAB	-	Silica Sand
1		FILL - (SP) SAND, some gravel, trace silt; grey; non-cohesive, moist to wet, loose	67.12 1.10	2	SS	8	Bentonite Seal
2				3	SS	3	Silica Sand
		- slight hydrocarbon-like odour and dark grey staining below 2.59 m depth		4	SS	4	
3	"H" Tri-cone Open Hole	BEDROCK (Not Sampled)	65.32 2.90				38 mm Diam. PVC #10 Slot Screen
4							
5		End of Borehole	62.89 5.33				
6							
7							
8							
9							
10							





DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED: EDW

PROJECT: 1781559

RECORD OF BOREHOLE: 17-02

SHEET 1 OF 1

LOCATION: N 5026831.5 ;E 441033.5

BORING DATE: July 25, 2017

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20, 40, 60, 80)

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3})

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	HEADSPACE VAPOUR CONCENTRATIONS	ADDITIONAL LAB. TESTING
0		GROUND SURFACE	68.27					
	Split Spoons HW Casing	FILL - (SW) gravelly SAND, angular; grey	0.00	1	GRAB	-	☐⊕ ND	
		FILL - (SP) SAND, fine to medium, some gravel; brown; non-cohesive, moist, compact to loose	68.02 0.25	2	SS	12	☐⊕ ND	
1				3	SS	5	☐⊕ ND	
2	"H" Tri-cone Open Hole	BEDROCK (Not Sampled)	66.52 1.75					
6		End of Borehole	62.02 6.25					

PIEZOMETER OR STANDPIPE INSTALLATION

Flush Mount Casing

Sand and Gravel

Native Cuttings and Bentonite Mix

Bentonite Seal

Silica Sand

38 mm Diam. PVC #10 Slot Screen

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED: EDW

PROJECT: 1781559

RECORD OF BOREHOLE: 17-03

SHEET 1 OF 1

LOCATION: N 5026856.2 ;E 441021.5

BORING DATE: July 26, 2017

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected 20 40 60 80 / HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		68.40							
0	Power Auger HW Casing	ASPHALTIC CONCRETE		0.05							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)		68.17 0.23							
1		FILL - (SM/GM) SILTY SAND, some gravel, trace brick, slag, organics; dark brown to black; non-cohesive, moist, loose			1 2 3	SS SS SS	13 5 5	☐⊕ ND ☐⊕ ND ☐⊕ ND			Silica Sand
2	"H" Tri-cone Open Hole	BEDROCK (Not Sampled)		66.73 1.67							
3											Bentonite Seal Silica Sand
4											
5											38 mm Diam. PVC #10 Slot Screen
6											
		End of Borehole		61.69 6.71							
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED: EDW

RECORD OF BOREHOLE: 17-04

PROJECT: 1781559

SHEET 1 OF 1

LOCATION: N 5026839.4 ;E 441025.1

BORING DATE: July 25, 2017

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ☐ (ND = Not Detected)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	68.64							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	0.00							Flush Mount Casing
		FILL - (SW) gravelly SAND; brown (PAVEMENT STRUCTURE)	0.09	1	GRAB	-	⊕☐			Silica Sand
		FILL - (SM/GM) SILTY SAND and GRAVEL; grey, contains rubble fill mix (cinders, ash, brick, ceramics, organics)	68.19 0.45	2	SS	16	⊕ ☐			
1										Native Cuttings and Bentonite Mix
		(SP) gravelly SAND, fine to medium, some fines; red brown; non-cohesive, moist, compact	67.02 1.62	3	SS	25	⊕☐ ND			
				4	SS	5	⊕☐ ND			
2	"H" Tri-cone Open Hole	BEDROCK (Not Sampled)	66.61 2.03							Bentonite Seal
3										Silica Sand
4										
5										38 mm Diam. PVC #10 Slot Screen
6										
		End of Borehole	62.39 6.25							
7										
8										
9										
10										

Headspace scale: 20 40 60 80

Hydraulic conductivity scale: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

Water content: Wp ⊢—○W—⊣ Wl; 20 40 60 80

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED: EDW

PROJECT: 1781559

RECORD OF BOREHOLE: 17-05

SHEET 1 OF 1

LOCATION: N 5026847.9 ;E 441045.6

BORING DATE: July 26, 2017

DATUM: Geodetic

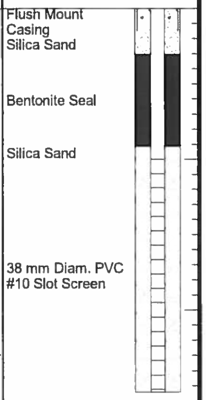
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	ELEV. DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl	ADDITIONAL LAB. TESTING
	68.00	GROUND SURFACE						
0	0.00	ASPHALTIC CONCRETE						
	0.09	FILL - (SW) gravelly SAND, angular; brown (PAVEMENT STRUCTURE)						
	0.25	FILL - (SP) SAND, some gravel, trace fines; brown	1	GRAB		⊕□		
1			2	SS		⊕□		
						⊞		
	66.40 1.60	FILL - (SP) SAND, some gravel; brown to black (stained); non-cohesive, very moist to wet, very loose - strong hydrocarbon-like odour from 1.98 to 2.54 m depth	3	SS		⊕□		
2			4	SS		505⊕		5%LEL
	65.46 2.54	End of Borehole Split Spoon Refusal on Probable Bedrock						
3								
4								
5								
6								
7								
8								
9								
10								

PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PAH
CHECKED: EDW

Log of Borehole: MW-1

Project #: 266791.001 **Logged By:** MK

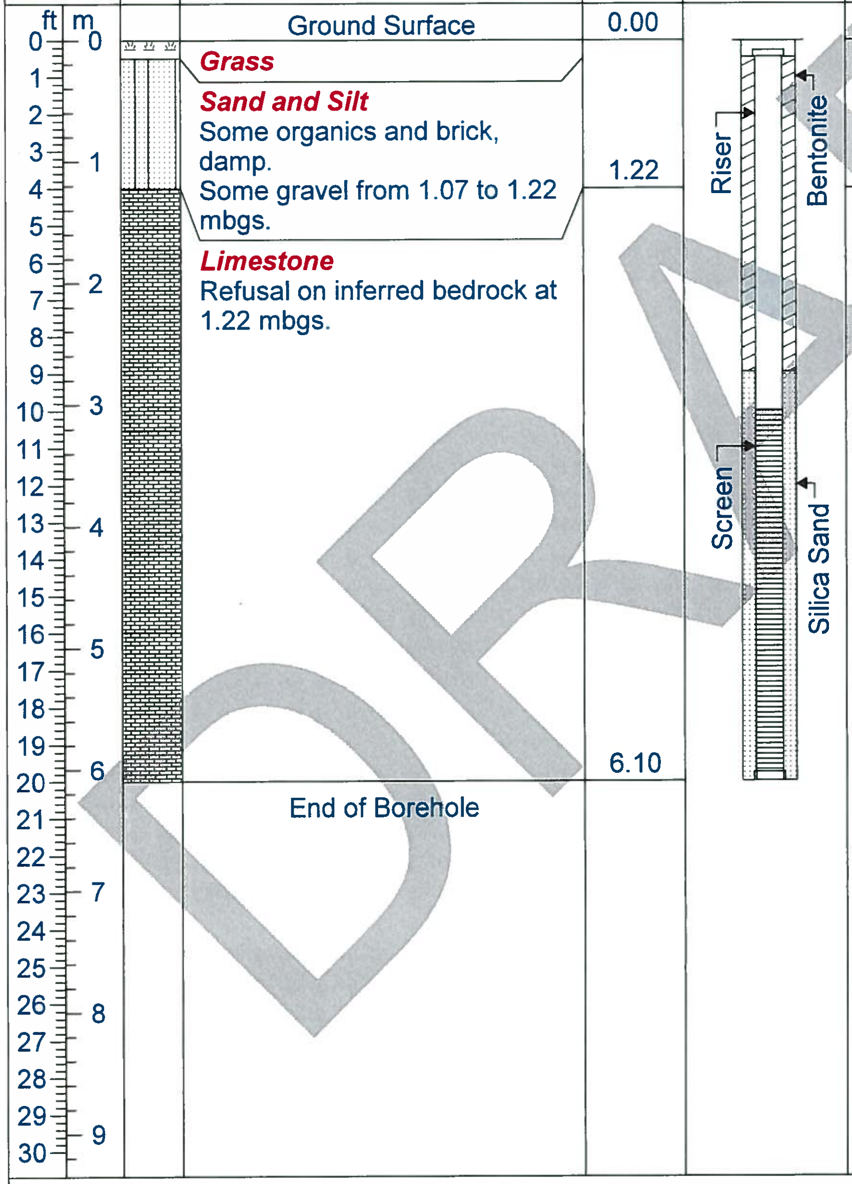
Project: Phase II Environmental Site Assessment

Client: Richmond Churchill Limited Partnership

Location: 380 Winona Avenue, Ottawa, Ontario

Drill Date: January 16, 2020

SUBSURFACE PROFILE				SAMPLE			
Depth (ft / m)	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration* (ppm) CGI/PID	Laboratory Analysis
0 ft / 0 m	Ground Surface	0.00					
	Grass			40	SS-1	0/0	
	Sand and Silt Some organics and brick, damp. Some gravel from 1.07 to 1.22 mbgs.	1.22			SS-2	0/0	PHCs, PAHs, VOCs, pH, Grain Size
	Limestone Refusal on inferred bedrock at 1.22 mbgs.	6.10					
	End of Borehole						



Contractor: Strata Drilling Group Inc.

Drilling Method: Direct Push

Well Casing Size: 5.1 cm

Note:
* Soil vapour concentrations measured using a RKI Eagle 2 equipped with a combustible gas indicator (CGI) and a photoionization detector (PID).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: MW-2

Project #: 266791.001 **Logged By:** MK

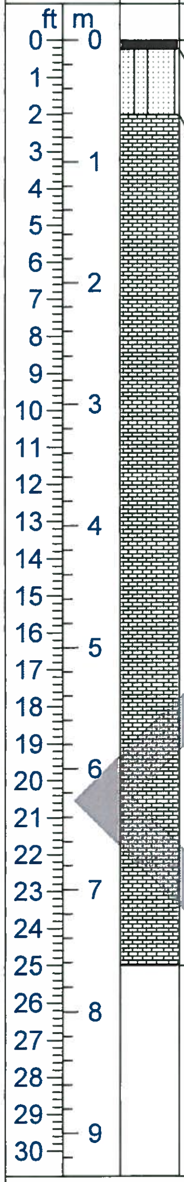
Project: Phase II Environmental Site Assessment

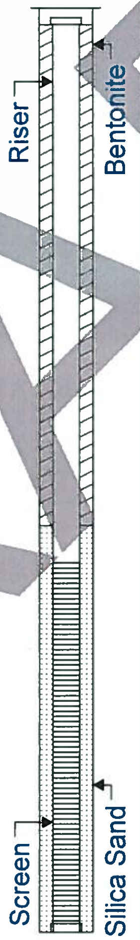
Client: Richmond Churchill Limited Partnership

Location: 380 Winona Avenue, Ottawa, Ontario

Drill Date: January 16, 2020

SUBSURFACE PROFILE					SAMPLE			
Depth	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration* (ppm) CGI/PID	Laboratory Analysis
		Ground Surface	0.00					
		Asphalt			40	SS-1	0/0	PHCs, PAHs, VOCs, pH
		Sand and Silt Some gravel, damp.	0.61					
		Limestone Refusal on inferred bedrock at 0.61 mbgs.	7.62					
		End of Borehole						





Contractor: Strata Drilling Group Inc.

Drilling Method: Direct Push

Well Casing Size: 5.1 cm

Note:
* Soil vapour concentrations measured using a RKI Eagle 2 equipped with a combustible gas indicator (CGI) and a photoionization detector (PID).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

APPENDIX 2

Figure 1 - Key Plan

Drawing PG5351-1 - Test Hole Location Plan

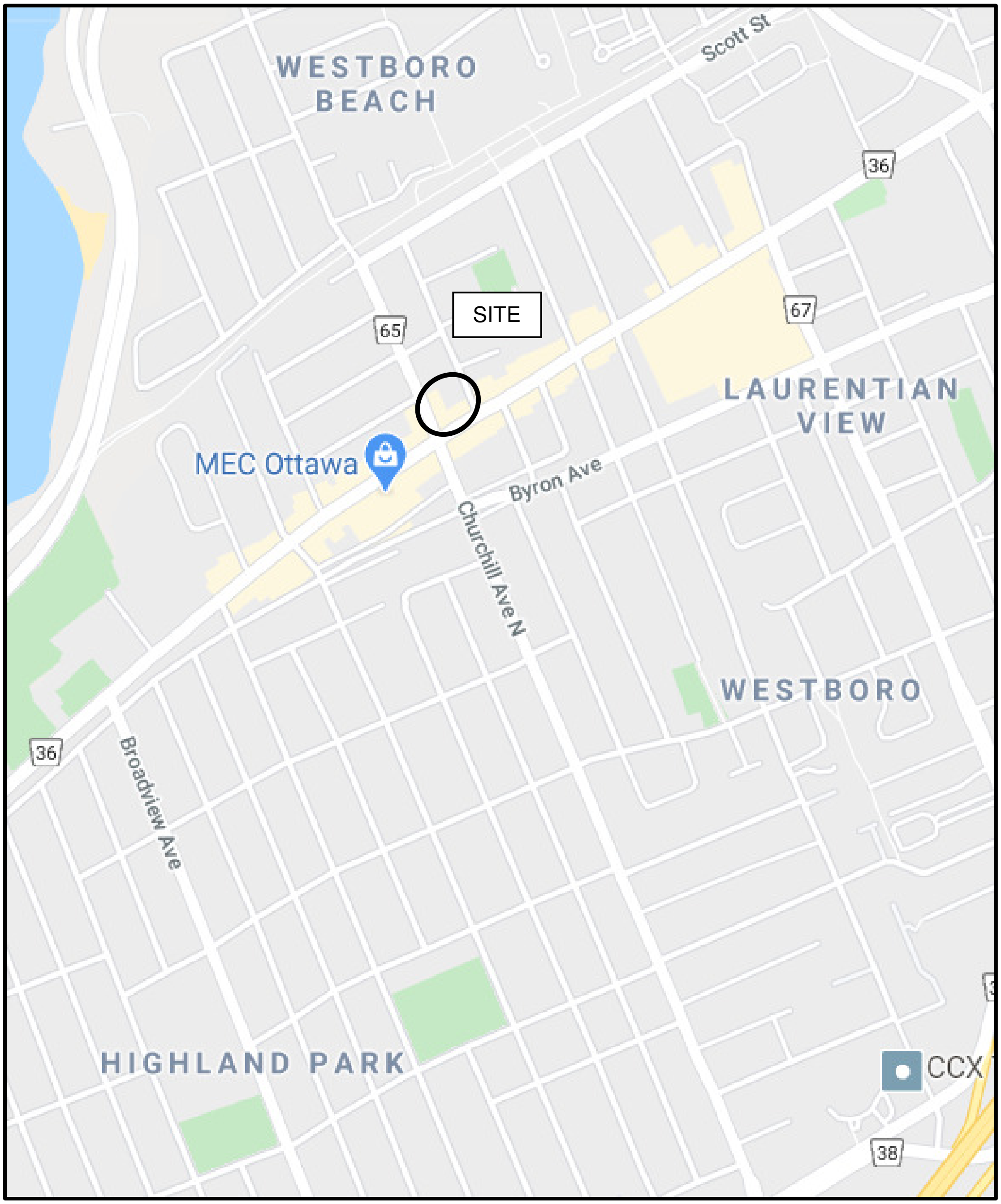
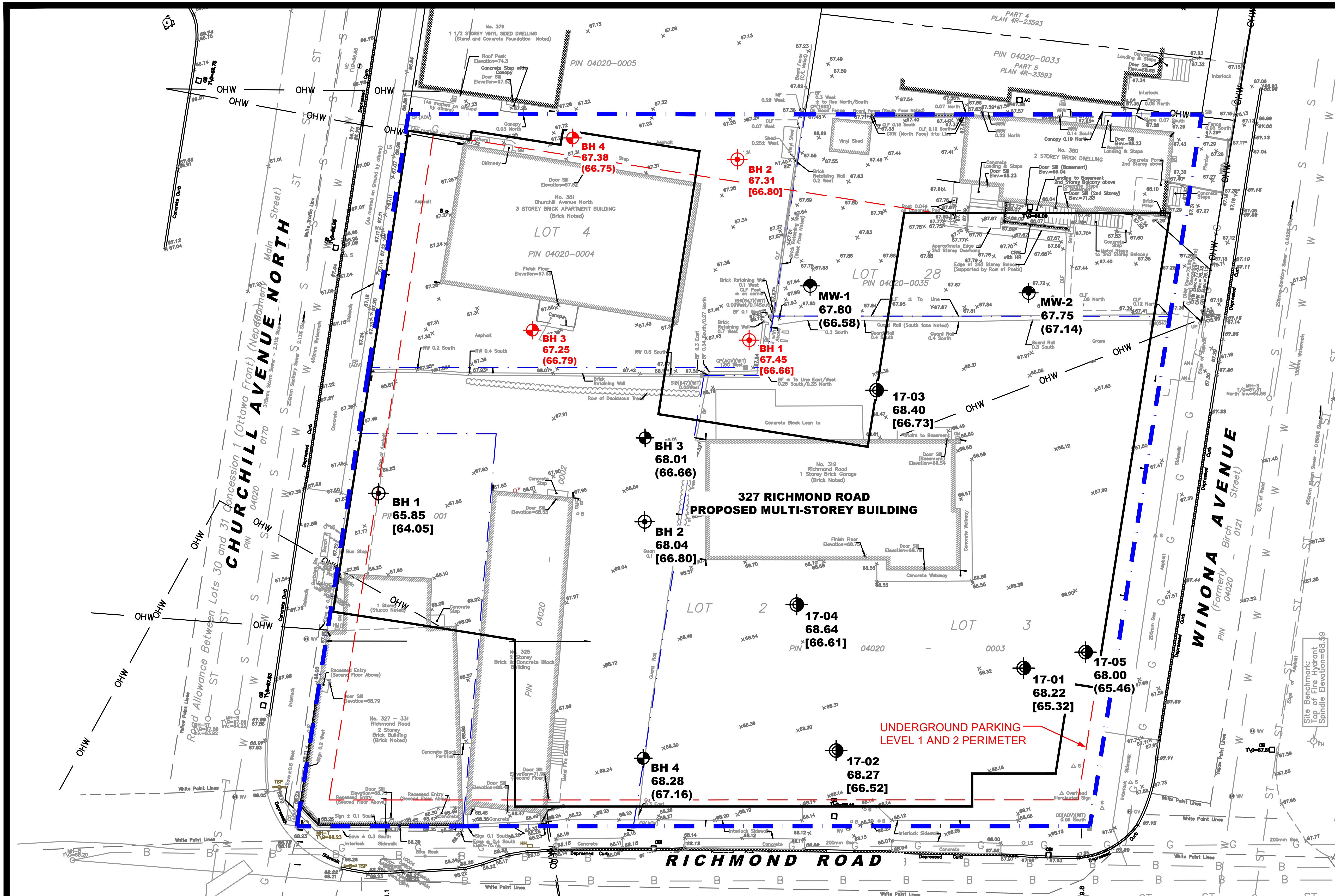


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT, PE3842, 2016)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT, PE3842, 2016)
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT, PE2628, 2012)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT, PE2628, 2012)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PINCHIN REPORT, 266791.001, 2020)
⊕	BOREHOLE LOCATION (GOLDER REPORT,1781559, 2017)
68.40	GROUND SURFACE ELEVATION (m)
[66.73]	BEDROCK SURFACE ELEVATION (m)
(65.46)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

SURVEY PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS FROM PATERSON GROUP REPORT PE2842 AND PE2628 WERE INFERRED FROM TOPOGRAPHIC SURVEY PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

p:\autocad drawings\geotechnical\pg53xx\pg5351\pg5351-1-test hole location plan (rev.01).dwg

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consulting engineers

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NO.	REVISIONS	DATE	INITIAL
1	REVISED AS PER CLIENT COMMENTS	15/06/2020	JV

RICHMOND CHURCHILL LIMITED PARTNERSHIP
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
327 RICHMOND ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 05/2020
Drawn by: YA
Report No.: PG5351-1
Checked by: JV
Dwg. No.: PG5351-1
Approved by: DJG
Revision No.: 1