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Geotechnical Investigation

Proposed Development
969-979 Wellington Street West &
26-40 Armstrong Street
Ottawa, Ontario

Prepared For

Magil Laurentian Reality Investments Inc.

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Report PG5494-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Magil-Laurention Realty Investments Inc. to conduct a geotechnical investigation for the proposed development to be located at 969-979 Wellington Street West and 26-40 Armstrong 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.

❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

It is our understanding that the proposed development will consist of a multi-storey building with two levels of underground parking. It is further understood that the building footprint will occupy the majority of the subject site. Access lanes and landscaped areas are anticipated.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Previous environmental field programs were carried out at the subject site by Paterson in March, 2013 and August 2016. During the 2013 field program, a total of two boreholes were drilled to a maximum depth of 5.7 m below existing ground surface. During the 2016 field program, a total of four boreholes were drilled to a maximum depth of 7.5 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. Additionally, a previous environmental investigation was carried out by others in May, 2012. At that time, two boreholes were advanced to a maximum depth of 5.6 m below existing ground surface. The locations of the boreholes are shown on Drawing PG5494-1 - Test Hole Location Plan included in Appendix 2.

The most recent field program was carried out March 9, 2020 which consisted of extending a total of two boreholes to a maximum depth of 5.1 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the site and taking into consideration underground utilities and site features. The locations of the boreholes are shown on Drawing PG5494-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon or the auger flights. The split-spoon, auger and rock core samples were classified on site and placed in sealed plastic bags. 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 presented as SS, AU and RC, respectively, on the Soil Profile and Test Data sheets.

Standard Penetration Tests (SPT) were conducted and 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 sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at three locations during the previous 2016 investigation, BH 1, BH 2 and BH 3, and at two locations during the 2020 investigation to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

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

Groundwater monitoring wells were installed in boreholes BHMW1, BHMW2, BH 1-13, BH 1, BH 2, and BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located at the northeast corner of the intersection of Wellington Street and Garland Street. An assumed elevation of 100.00 m was provided for the TBM. The borehole locations and the ground surface elevation of the borehole locations are presented on Drawing PG5494-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples recovered from the subject site were visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is located at the southeast corner of the intersection of Wellington Street West and Somerset Street West. The site consists of the entire block, surrounded by Armstrong Street to the west, Hilda Street to the north, Wellington Street West to the east, and Garland Street to the south. The site is currently occupied by an at grade asphalt parking lot located at the western portion of the site. There is an existing multi storey office building Located at the southern portion of the site. The eastern portion of the site is currently occupied by an automotive service garage and a commercial building. The northern portion of the site is occupied by four residential dwellings. The ground surface across the subject site is at grade with the bordering streets.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of a pavement structure overlying a silty sand fill with crushed stone. Grey limestone bedrock was encountered underlying the above noted layers at depths ranging from 0.4 to 1.4 m depth at the borehole locations. Bedrock samples recovered during the current field investigation indicate a fair to excellent quality grey limestone bedrock. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone with shale of the Bobcaygeon Formation with an overburden drift thickness of 0 to 5 m depth.

Specific details of the subsoil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

Based on field observations, observations at adjacent sites and the recovered soil samples' moisture levels, consistency and colouring, the long-term groundwater table is not present within the overburden soils.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BHMW1	99.57	2.43	97.14	October 28, 2019
BHMW2	99.46	2.52	96.94	October 28, 2019
BH 1-13	99.26	3.33	95.93	April 3, 2016
BH 1	98.91	3.12	95.79	October 28, 2019
BH 2	99.43	1.87	97.56	October 28, 2019
BH 3	99.72	2.30	97.42	October 28, 2019

Note: The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located in front of 987 Wellington Street West. An assumed elevation of 100.00 m was provided for the TBM.

Groundwater levels are subject to seasonal fluctuations and therefore levels could differ at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed multi-storey building is expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the two levels of underground parking. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal should be required for the construction of the parking garage levels.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will 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. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. 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 recommended to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

Horizontal rock anchors may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

Fill placed 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. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the 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 lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface could be designed for 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 loose materials, and should not contain surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5, can be provided if founded on limestone bedrock which is free of seams, fractures and voids within 1.5 m below the founding level. This should be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within all the footing footprints. A minimum of one probe hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response is **Class C** for the shallow foundations at the subject site. A higher seismic site class, such as Class A or B is available for the subject site. However, a site specific seismic shear wave test is required to provide the higher site classes according to the 2012 Ontario Building Code. Soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone. Alternatively, excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

It is expected that the basement walls are to be poured against a drainage system, which will be placed against the exposed bedrock face. A nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against a drainage membrane placed directly over the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, 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. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

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

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

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

5.7 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 by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that 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 anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on the testing results, the unconfined compressive strength of the limestone bedrock below the subject site ranges between 80 and 100 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 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock. Therefore, Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

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 Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

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	1.2	0.55	1.75	250
	2	0.8	2.8	500
	3.2	1.4	4.6	1000
	5.3	2.2	7.5	2000
125	1	0.5	1.5	250
	1.7	0.7	2.4	500
	2.6	1.1	3.7	1000
	4.1	1.8	5.9	2000

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

Table 4 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 5 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

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 sub-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 SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room is available for exterior backfill. It is suggested that this system could be as follows:

❏ Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface);
❏ composite foundation drainage layer

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 footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, 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.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and 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.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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.

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 designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

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, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 6 - 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, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock 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 pipe obvert 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 SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to existing groundwater level and inferred depths of the proposed footings, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 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, typically 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, and EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage 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 (i.e.- less than 50,000 L/day) with 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.

Impacts on Neighbouring Structures

Based on our observations, the groundwater level is anticipated at a 2 to 3 m depth and within the bedrock. Therefore, a local groundwater lowering is anticipated 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 temporary groundwater lowering.

The neighbouring structures are expected to be founded within native glacial till and/or directly over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means. 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 difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed 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 pH of the sample indicate that they are not significant factors is creating a corrosive environment for exposed ferrous metal at this site, whereas the resistivity is indicative of a moderate to very aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation 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.

☐ 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 grading plan, drawings and specifications are completed.

A geotechnical investigation 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. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Magil Laurentian Realty Investments Inc., 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
August 28, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc., P.Eng., ing.

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Magil Laurentian Realty Investments Inc.
- ☐ Paterson Group

APPENDIX 1

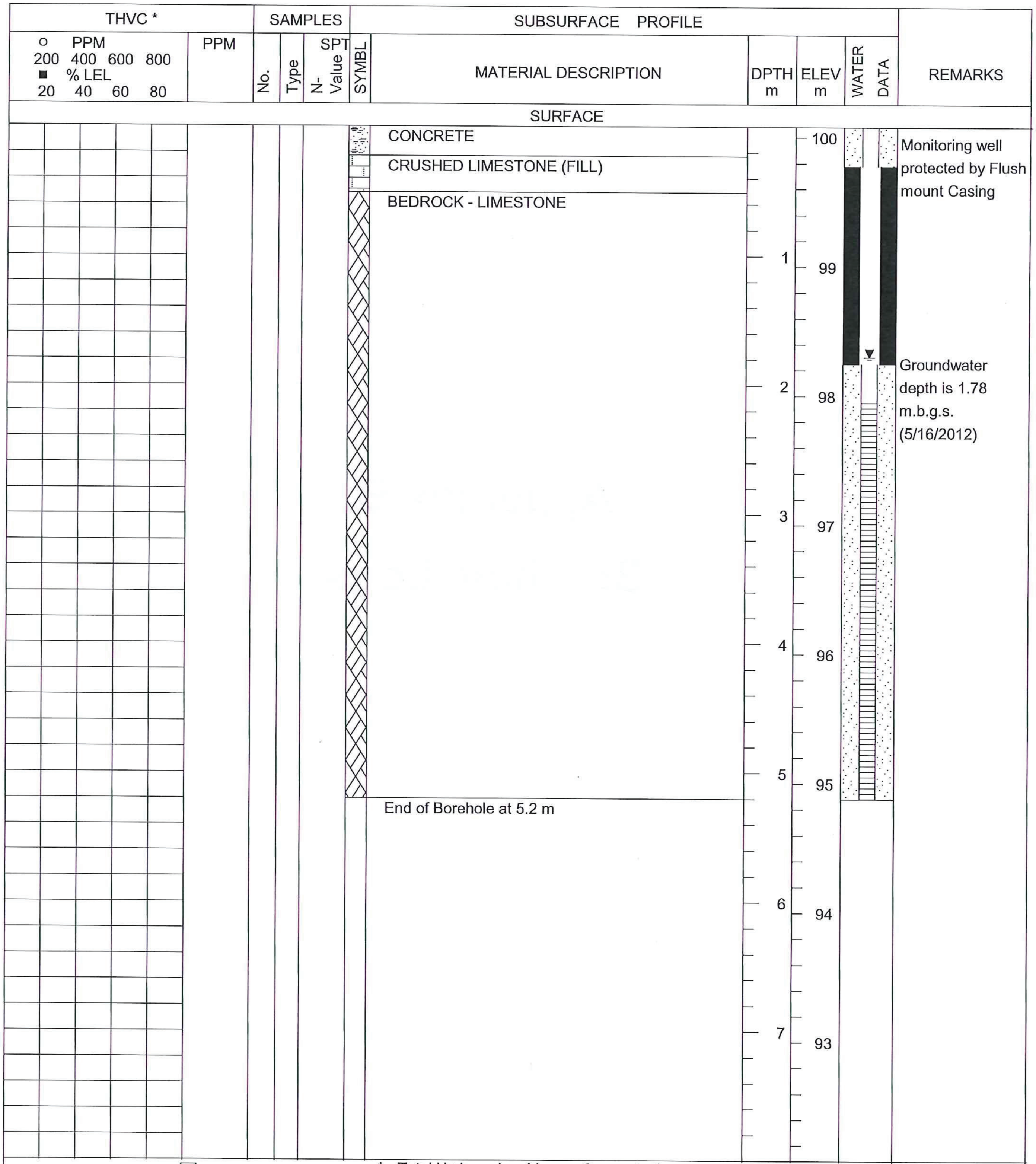
SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

LOG OF BOREHOLE/MONITORING WELL BHMW1

REF. No.: OE-OT-014804	**DST CONSULTING ENGINEERS INC.**
CLIENT: Carling Otto	
PROJECT: Phase II Environmental Site Assessment	METHOD: Portable Drill
LOCATION: 973 Wellington Street West, Ottawa, Ontario	DIAMETER: 31.75 mm ID
SURFACE ELEVATION: 100.08 meters	DATE: May 14, 2012



CONCRETE

CRUSHED LIMESTONE (FILL)

BEDROCK - LIMESTONE

End of Borehole at 5.2 m

Monitoring well protected by Flush mount Casing

Groundwater depth is 1.78 m.b.g.s. (5/16/2012)

⊟ Coring

m.b.t.c. Meter Below Top of Casing

m.b.g.s. Meter Below Grade Surface

* - Total Hydrocarbon Vapour Concentration

NR - No Recovery

ND - Not Detectable

RKI Eagle

Sheet 1 of 1

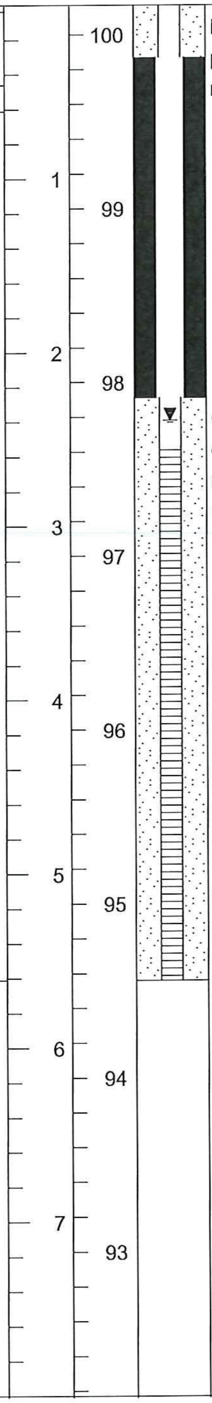
APPENDIX B

LOG OF BOREHOLE/MONITORING WELL BHMW2

DST CONSULTING ENGINEERS INC.

REF. No.: OE-OT-014804
CLIENT: Carling Otto
PROJECT: Phase II Environmental Site Assessment
LOCATION: 973 Wellington Street West, Ottawa, Ontario
SURFACE ELEVATION: 100.17 meters
METHOD: Portable Drill
DIAMETER: 31.75 mm ID
DATE: May 15, 2012

THVC * ○ PPM 200 400 600 800 ■ % LEL 20 40 60 80	PPM	SAMPLES No.	SAMPLES Type	SAMPLES SPT N-Value	SUBSURFACE PROFILE SYMBL	SUBSURFACE PROFILE MATERIAL DESCRIPTION	REMARKS
						SURFACE	
			⊟			ASPHALT- 0.038m	Monitoring well protected by Flush mount Casing
○	25		⊟			GRAVEL & SAND (FILL) - Brown	
			⊟			SAND (FILL) - Grey	
						BEDROCK - LIMESTONE	Groundwater depth is 2.387 m.b.t.c. (5/16/2012)
						End of Borehole at 5.61 m	



⊟ Coring
m.b.t.c. Meter Below Top of Casing
m.b.g.s. Meter Below Grade Surface

* - Total Hydrocarbon Vapour Concentration
NR - No Recovery
ND - Not Detectable
RKI Eagle

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
969 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

BORINGS BY Portable Drill

DATE 2013 March 27

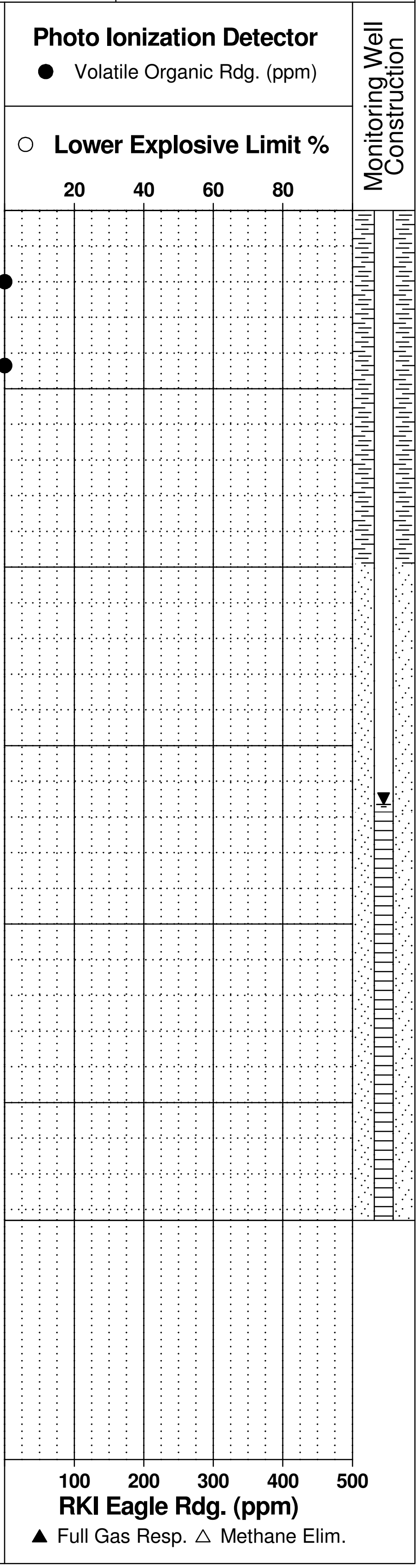
FILE NO. PE2934

HOLE NO. BH 1-13

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.26
Concrete 0.10						
FILL: Crushed stone 0.20						
FILL: Brown silty sand with gravel, trace clay 0.60	SS	1	62			
FILL: Brown silty clay with sand 1.14	SS	2	48		1	98.26
					2	97.26
	RC	1	100	86		
BEDROCK: Grey limestone interbedded with shale					3	96.26
					4	95.26
	RC	2	59	26		
5.66					5	94.26

End of Borehole

(GWL @ 3.33m-April 3, 2013)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
969 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

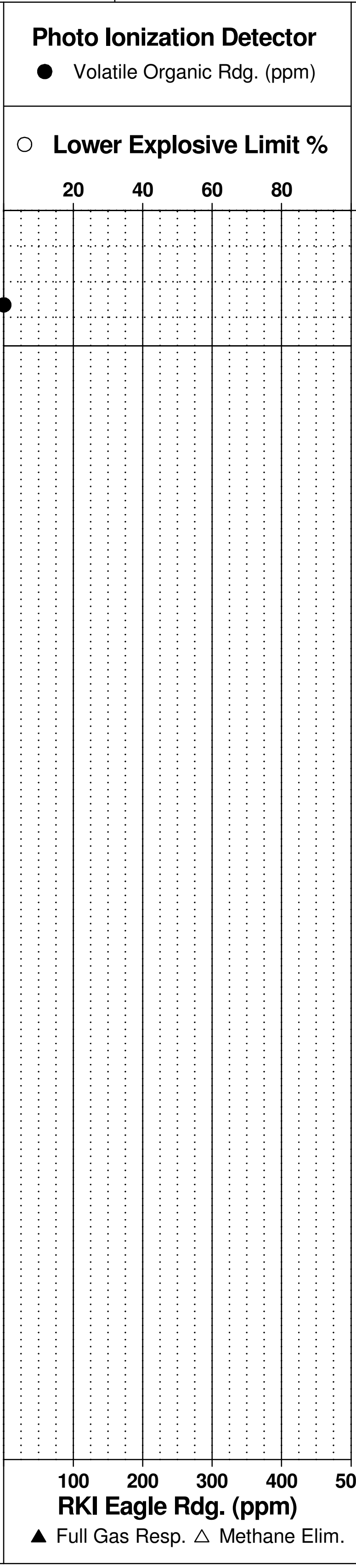
BORINGS BY Portable Drill

DATE 2013 March 27

FILE NO. PE2934

HOLE NO. BH 2-13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Concrete 0.10							
FILL: Crushed stone 0.30							
FILL: Brown sand, trace gravel and brick 0.76		SS	1	67			
End of Borehole							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
979 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2016 August 24

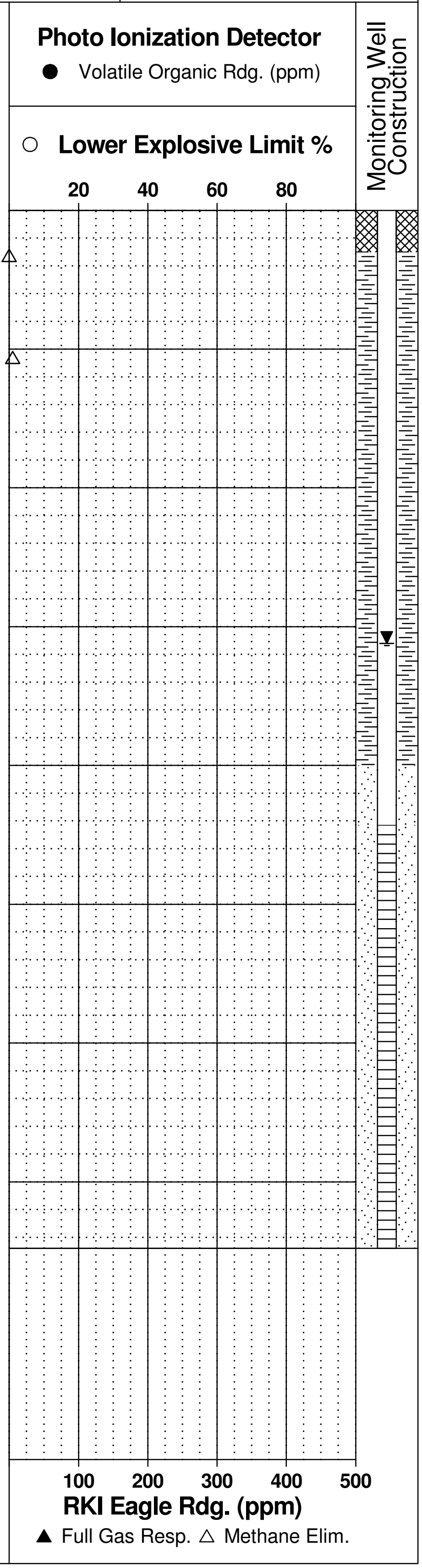
FILE NO. PE3837

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.91
Asphaltic concrete 0.05						
FILL: Crushed stone with silt 0.60	AU	1				
FILL: Brown sandy silt with clay, trace gravel and cobbles 1.35	SS	2*	87	38	1	97.91
BEDROCK: Grey limestone	RC	1	100	81	2	96.91
					3	95.91
	RC	2	100	91	4	94.91
	RC	3	100	92	5	93.91
					6	92.91
7.48	RC	4	100	85	7	91.91
End of Borehole						

(GWL @ 3.12m - Oct. 28, 2019)

* Sample SS2 submitted for analytical testing



SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
979 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2016 August 24

FILE NO. PE3837

HOLE NO. BH 2

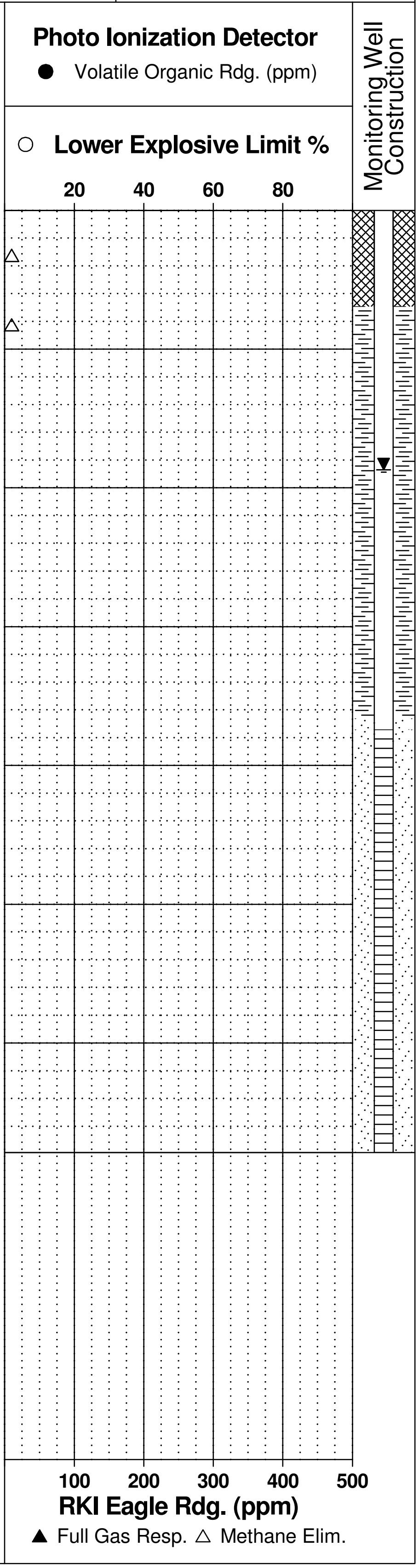
SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with silt	0.89	AU	1*		
		SS	2	60	50+
BEDROCK: Grey limestone	6.79	RC	1	91	84
		RC	2	98	93
		RC	3	99	95
		RC	4	100	100
		RC	5	100	100

DEPTH (m)	ELEV. (m)
0	99.43
1	98.43
2	97.43
3	96.43
4	95.43
5	94.43
6	93.43

End of Borehole

(GWL @ 1.87m - Oct. 28, 2019)

* Sample AU1 submitted for analytical testing



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
979 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

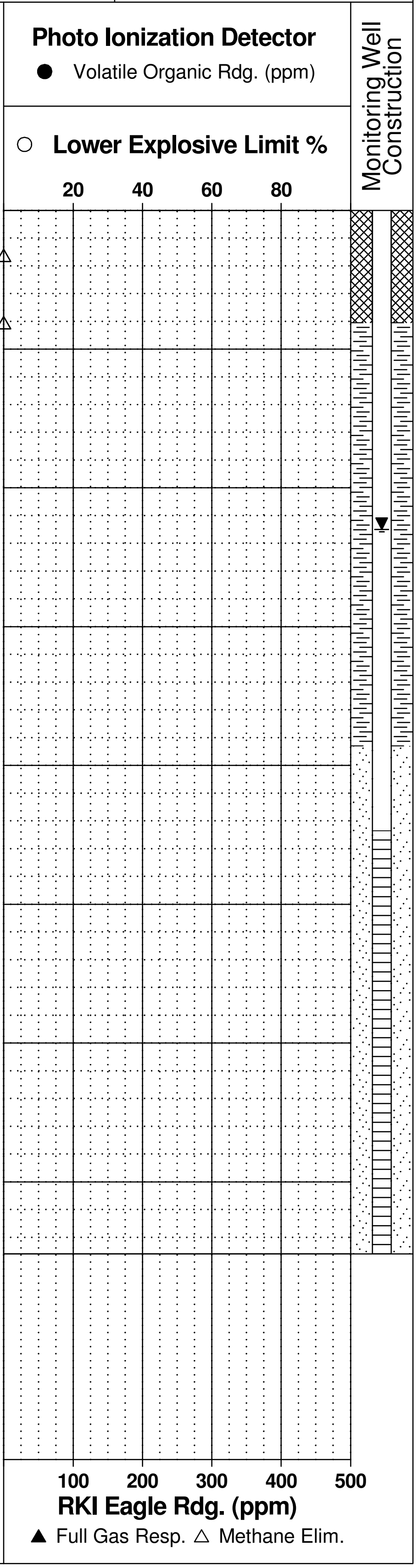
DATE 2016 August 24

FILE NO. PE3837

HOLE NO. BH 3

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with silt		AU	1		
	0.81	SS	2	50	50+
BEDROCK: Grey limestone		RC	1	80	48
		RC	2	97	86
		RC	3	97	33
		RC	4	100	93
	7.52	RC	5	100	90
End of Borehole					
(GWL @ 2.30m - Oct. 28, 2019)					

DEPTH (m)	ELEV. (m)
0	99.72
1	98.72
2	97.72
3	96.72
4	95.72
5	94.72
6	93.72
7	92.72



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
979 Wellington Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northwest corner of Garland Street and Wellington Street West. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2016 August 24

FILE NO. PE3837

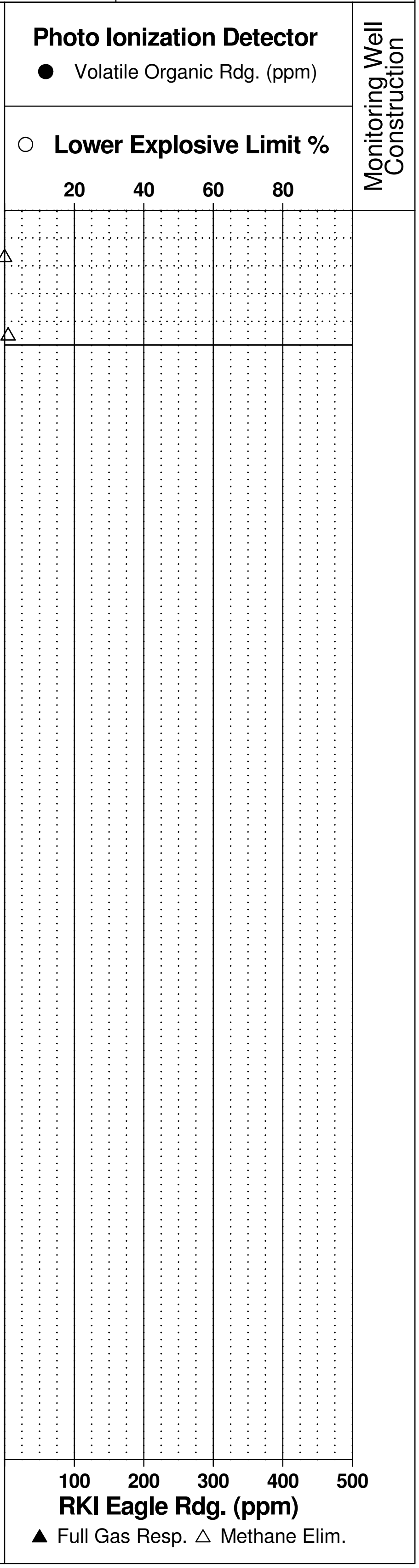
HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0	99.96	
Asphaltic concrete 0.05								
FILL: Crushed stone with silt 0.81		AU	1					
FILL: Grey-brown sand, trace concrete 0.97		SS	2*	75	50+			

End of Borehole

Practical refusal to augering at 0.97m depth

* Sample SS2 submitted for analytical testing



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

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
36 and 40 Armstrong Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed elevation = 100.00m.

FILE NO. **PE4752**

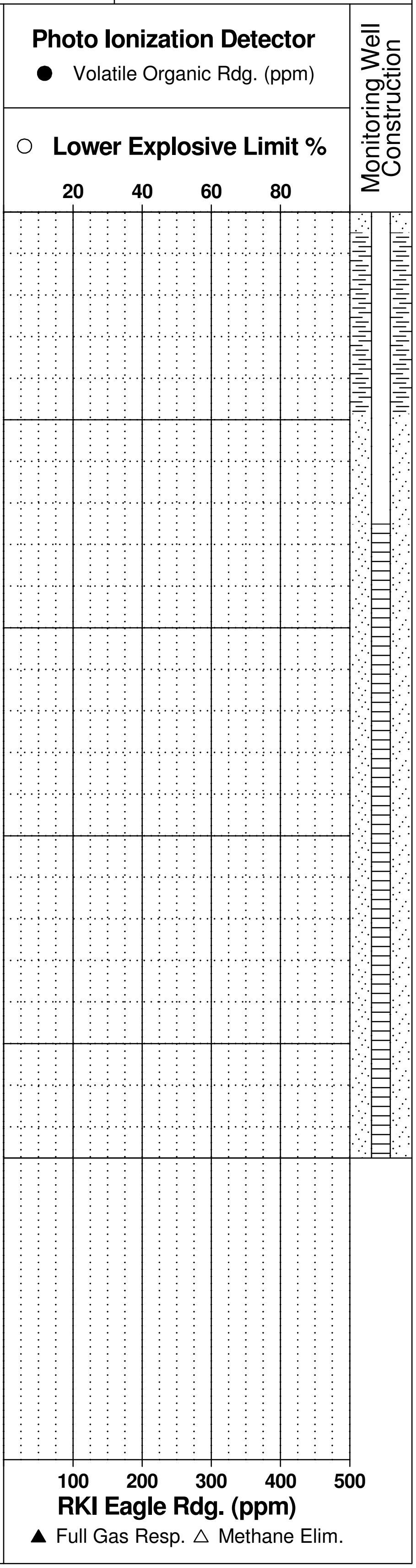
REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 2020 March 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.45
Asphaltic concrete 0.10							
FILL: Brown silty sand with gravel 0.36		AU	1				
		RC	1	100	73	1	99.45
						2	98.45
BEDROCK: Good to excellent quality, grey limestone		RC	2	100	96		
						3	97.45
		RC	3	100	70		
						4	96.45
4.55							
End of Borehole							



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

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
36 and 40 Armstrong Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 March 9

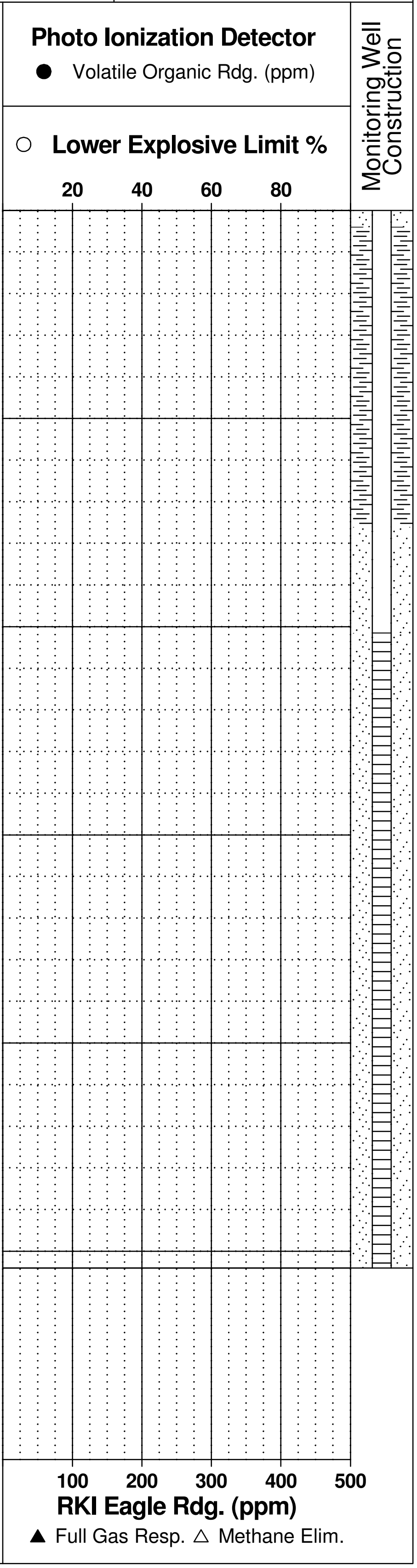
FILE NO. PE4752

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.89
BEDROCK: Excellent to fair quality, grey limestone	5.08
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	67	50+
RC	1	100	100
RC	2	96	81
RC	3	100	100
RC	4	100	64

DEPTH (m)	ELEV. (m)
0	100.81
1	99.81
2	98.81
3	97.81
4	96.81
5	95.81



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

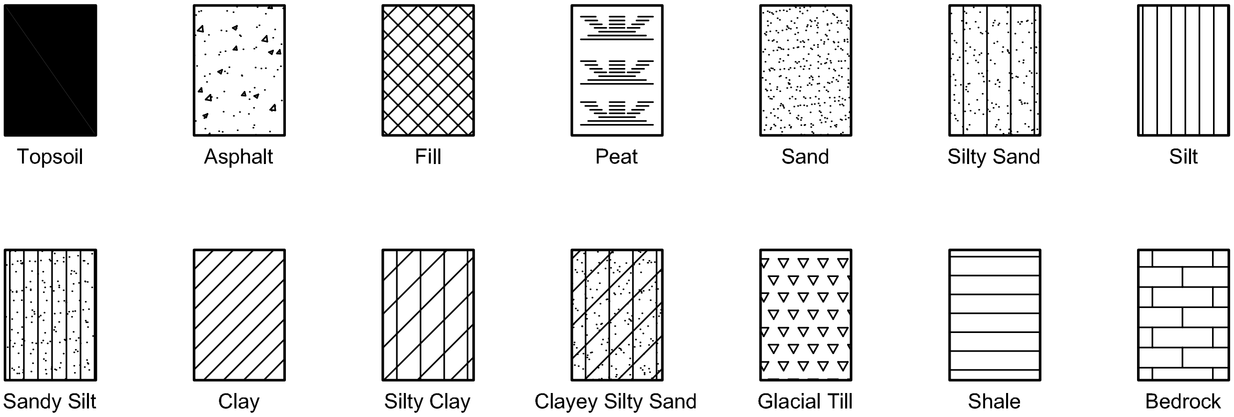
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

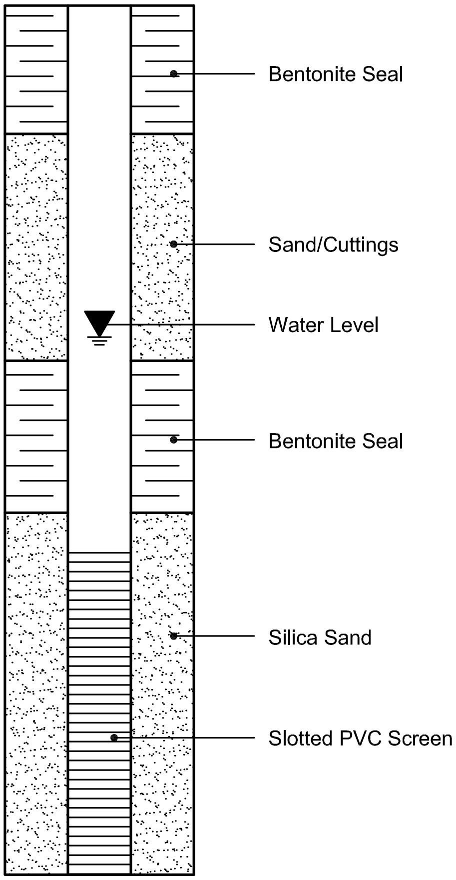
SYMBOLS AND TERMS (continued)

STRATA PLOT

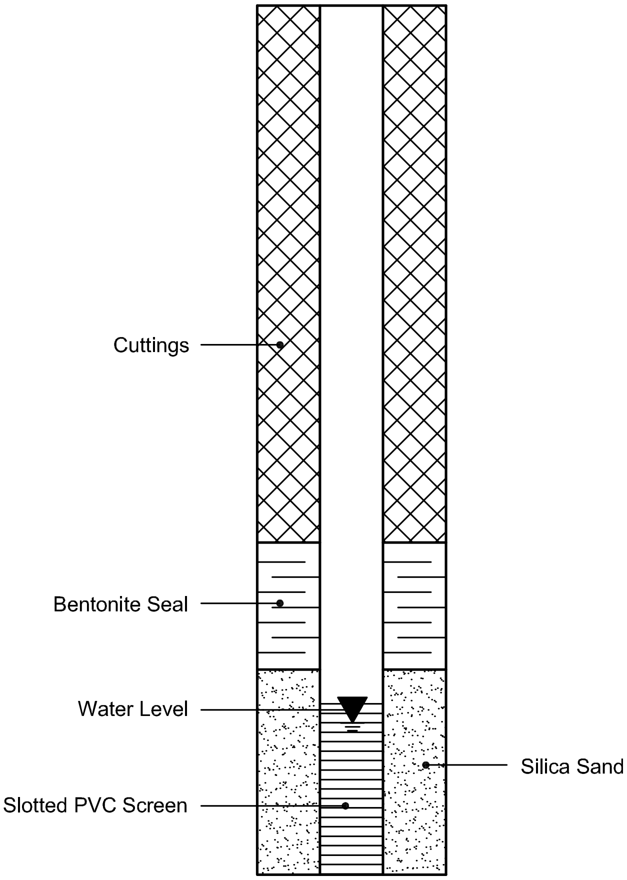


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 20218

Report Date: 02-Sep-2016
Order Date: 29-Aug-2016
Project Description: PG3849

	Client ID:	BH1-SS2	-	-	-
	Sample Date:	24-Aug-16	-	-	-
	Sample ID:	1636069-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.93	-	-	-
Resistivity	0.10 Ohm.m	20.2	-	-	-
Anions					
Chloride	5 ug/g dry	162	-	-	-
Sulphate	5 ug/g dry	54	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5494-1 - TEST HOLE LOCATION PLAN

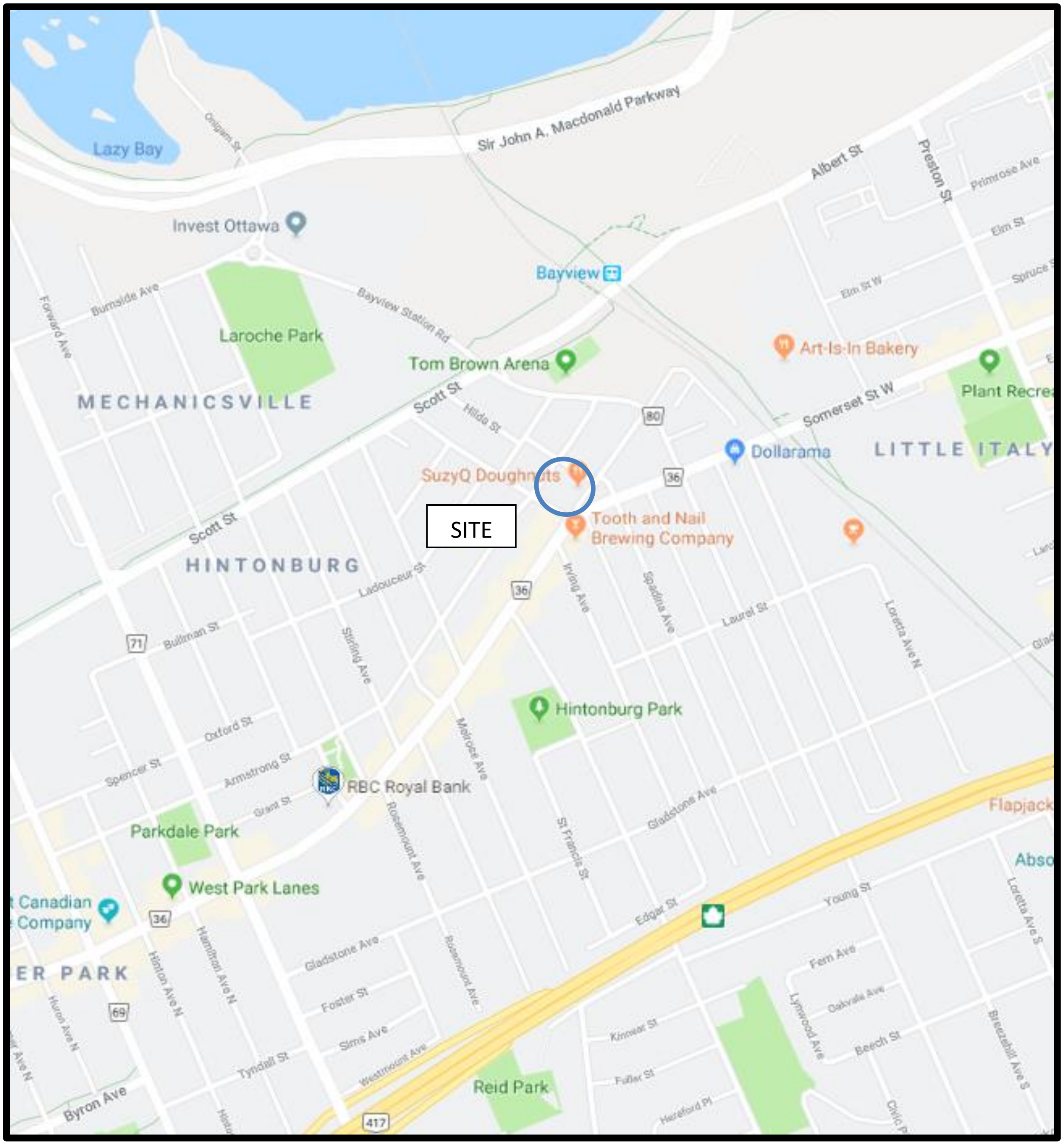
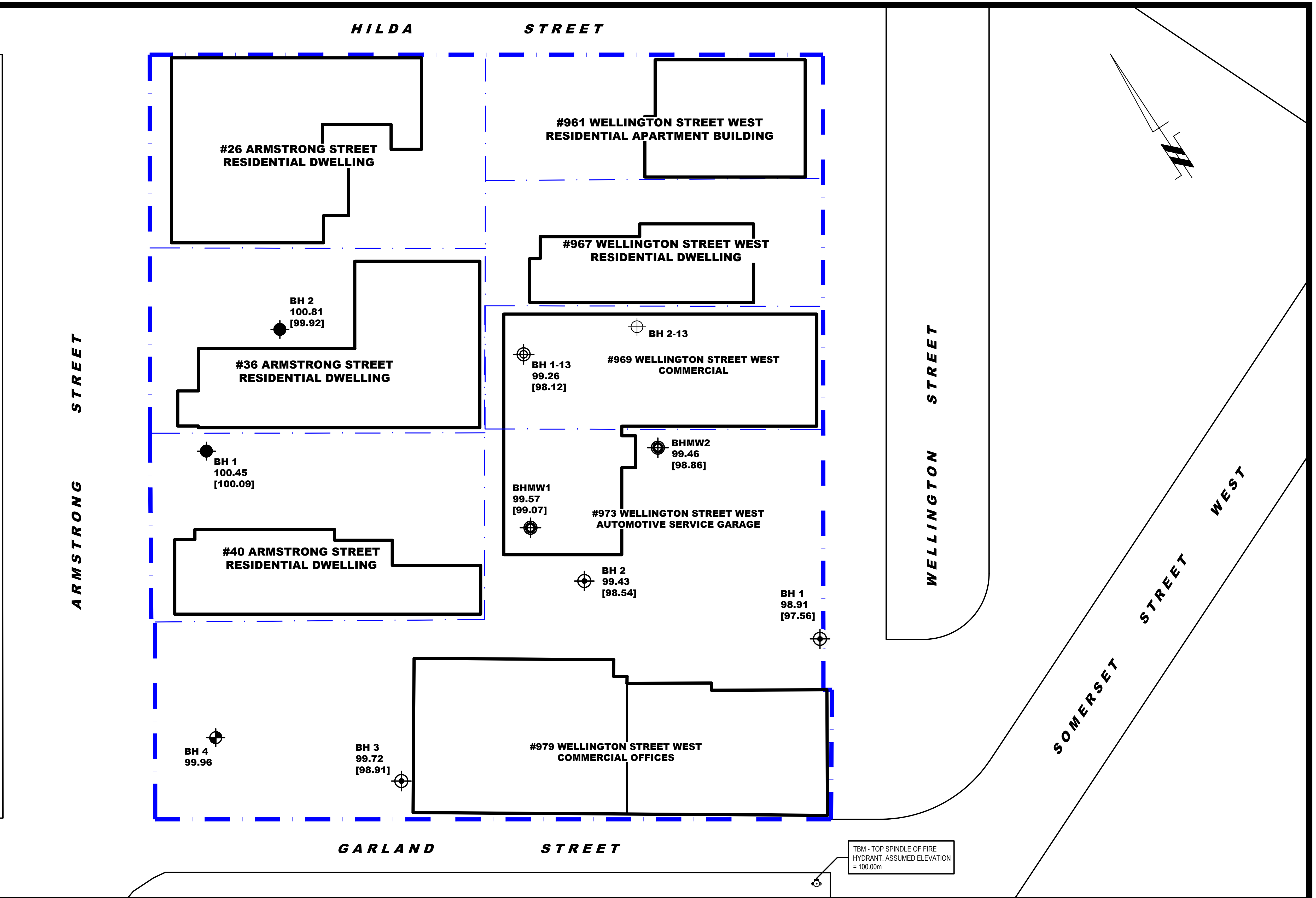


FIGURE 1

KEY PLAN



LEGEND:

● BOREHOLE LOCATION, PATERSON GROUP REPORT PE4752, 2020

⊕ BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE2934, 2013

⊕ BOREHOLE LOCATION, PATERSON GROUP REPORT PE2934, 2013

⊕ BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE3837, 2016

⊕ BOREHOLE LOCATION, PATERSON GROUP REPORT PE3837, 2016

⊕ MONITORING WELL LOCATION BY OTHERS

99.72 GROUND SURFACE ELEVATION (m)

[98.91] BEDROCK SURFACE ELEVATION (m)

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

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NO.	REVISIONS	DATE	INITIAL
0			

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Title:

TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 08/2020
Drawn by: NFRV
Report No.: PG5494-1
Checked by: JV
Dwg. No.: PG5494-1
Approved by: DJG
Revision No.: 0

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