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Geotechnical Investigation
Proposed Multi-Storey Building
2046 and 2050 Scott Street
Ottawa, Ontario

Prepared For

Scott Street Development Inc.

March 25, 2021

Report PG5222-1Revision 2

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Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Scott Street Development Inc. to prepare a geotechnical investigation report for the proposed high-rise building to be constructed at 2046 and 2050 Scott Street and 295, 297 to 299 and 301 Ashton Avenue, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current investigation was to:

❑ Determine the subsoil and groundwater conditions at this site by reviewing test holes completed by Paterson and by Others.

❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 Proposed Development

Based on the available information, it's our understanding that the proposed development will consist of a hi-rise building structure which is anticipated to be provided with 4 underground levels of parking and storage. It is understood that the first floor will be a mixed-use commercial and residential level, while the remaining storeys will be dedicated to residential use. The proposed development will also include associated at grade access lanes and landscaped areas. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on March 19 and 20, 2020, and consisted of five boreholes which were advanced to a maximum depth of 11.8 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5222-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track-mounted low-clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of augering and rock coring to the required depths at the selected locations, and sampling and testing the overburden.

Furthermore, fourteen boreholes from previous investigations carried out by others are also included to supplement our findings.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Rock samples were recovered using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

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

The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Groundwater monitoring wells were installed in the completed boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5222-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

4.0 Observations

4.1 Surface Conditions

The north portion of the subject site is currently occupied by two single storey commercial buildings with associated access lanes and parking areas, and the southern portion is occupied by three residential buildings. The ground surface across the subject site is relatively flat and at grade with Scott Street to the north and Ashton Street to the south. The subject site is bordered to the west by an existing apartment building, and o the east by a sporting facility.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of an approximate 1.3 to 2.5 m thick layer of fill underlying the topsoil or pavement surface. The fill was generally observed to consist of a silty sand to sand and crushed stone. The fill layer is further underlain by a deposit of glacial till consisting of a dense, brown silty sand with some gravel, cobbles and boulders.

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

Bedrock

Practical refusal to augering was encountered at depths ranging from 2.5 m to 3.8 m below the existing ground surface. Bedrock was cored at the borehole locations, and was observed to consist of grey limestone. Based on the RQDs of the recovered rock core, the bedrock can be classified as fair to good in quality. Additionally, based on the borehole coverage completed by others, the refusal on inferred bedrock was encountered between 2.3 m and 3.4 m throughout the site

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation. The overburden drift thickness is estimated to be between 1 to 3 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on March 26, 2020. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	63.31	6.31	57.00	March 26, 2020
BH 2	63.39	6.33	57.06	March 26, 2020
BH 3	63.03	5.32	57.71	March 26, 2020
BH 5	63.04	5.98	57.06	March 26, 2020

Note:
- The ground surface elevations at the borehole locations are referenced to a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 5 to 6 m below existing ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1. Furthermore, groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed high-rise building. It is expected that the proposed building will be founded on spread footings placed directly on a clean, surface sounded bedrock bearing surface.

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

Due to the depth of the proposed underground parking garage, a water suppression system is recommended to lessen the volume of water infiltration over the long term during post-construction.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building footprint.

Bedrock Removal

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

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

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

Excavation side slopes in sound bedrock can be excavated with almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

Bedrock Reinforcement and Stabilization

Due to the founding depth of the proposed building, bedrock stabilization may be required where the proposed foundation extends into the limestone bedrock.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

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

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

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Conventional Spread Footings

Footings can be designed a factored bearing resistance value at ULS of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5 if footings are founded upon a clean limestone bedrock bearing surface and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. The bedrock vertical face, along the excavation sides and within depressed areas such as the elevator pit, can be assessed by the geotechnical engineer to confirm the soundness of the bedrock at depth.

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

Settlement

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

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or flatter).

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. However, a higher site class (**Class A**) can be achieved. The higher site class will require a site specific shear wave velocity test to be completed in confirmation of the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Please refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

For the proposed development, all overburden soil will be removed from the building footprints, leaving the bedrock as the founding medium for the basement floor slab. It is anticipated that the basement area for the proposed buildings will be mostly parking and the recommended pavement structures noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the sub-floor fill under the lower basement floor (discussed further in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed buildings. 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 drained unit weight of 20 kN/m^3 (effective unit weight of 13 kN/m^3).

It is expected that the majority of the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective unit weight of 15.5 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for foundation walls which are poured against 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.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil and bedrock should be utilized, 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 material
γ = unit weight of fill of the applicable retained material (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

Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

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

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 servicing.

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

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

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress for sound rock of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between 50 and 80 MPa, which is stronger than most routine grouts.

A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed buildings are determined.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

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

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared. The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

5.8 Pavement Structure

Car only parking, access lanes and heavy truck parking areas are anticipated at this site. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Pavement Structure - Access Lanes

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building footprint will occupy the majority of the subject site. It is expected that insufficient room will be available for exterior backfill along the building perimeter and, therefore, the foundation wall will be blind poured against a drainage system placed on the bedrock face.

To manage and control groundwater infiltration to the building's storm sump pump(s) over the long term, the following foundation drainage and water suppression system is recommended to be installed on the exterior perimeter and surface of the building's foundation walls using the following methodology:

- ☐ Throughout the building excavation and bedrock removal process, the vertical bedrock should be hoe-rammed and grinded to provide a smooth and flat substrate surface approved for the placement of the waterproofing membrane. Shotcrete and/or lean concrete anchored into the bedrock with steel dowels and/or rock anchors may be required to fill in cavities and smooth out angular features and voids. This process and the requirement for shotcrete and/or lean concrete should be periodically reviewed by Paterson personnel during the excavation program.

- ☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the lower underground parking levels between the underside of footing elevation and up to the bedrock surface. The waterproofing membrane should consist of a 150 miL granular bentonite surface laminated to 20 miL thick HDPE membrane. The membrane should be installed in horizontal lifts and in accordance with the manufacturer's specifications in a shingle fashion with the HDPE side facing the applicator/the building to an adequately prepared substrate surface. The waterproofing membrane should extend horizontally a minimum of 600 mm below the building's perimeter footings as depicted on Figure 2 - Water Suppression System.

❑ As a secondary measure to capture any groundwater that may, in the unlikely event bypass the waterproofing membrane, it is recommended that a composite foundation drainage membrane, such as 6000 series membrane by DeltaDrain, G100N by MiraDrain or equivalent and approved other, be placed between the waterproofing membrane and exterior foundation wall surfaces. The composite foundation drainage membrane should extend from finished grade to the underside of footing level with the geotextile layer facing the waterproofing membrane.

❑ 150 mm diameter sleeves placed 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 the storm sump pit(s) within the lower basement area. It is recommended that the 150 mm diameter drainage sleeves be installed by carefully cutting an ‘X’ shaped incision through the composite foundation drainage. In areas where the drainage sleeves are to be installed, care must be taken to ensure that the underlying waterproofing membrane is **NOT** punctured during this process.

Figures 2 and 3 of Appendix 2 presents the proposed water suppression system. Further design information can be provided at the time of design as required.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. The spacing of the underfloor drainage system should be confirmed at the time of excavation when water infiltration can be better assessed. For design purposes, it’s suggested that one 150 mm in diameter perforated drainage pipe be placed in each bay.

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 any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement 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 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 Delta Drain 6000 or an approved equivalent, 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 required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with adequate 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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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

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

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

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

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

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

Temporary shoring may be required 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. Additional information can be provided when the above details are known.

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

Table 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
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.
Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

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

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. 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 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

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

A temporary Ministry of Environment, Conservation and Parks (MECP) Category 3 Permit to Take Water (PTTW) may be required if more than 400,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application 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, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

It is understood that 5 levels of underground parking are planned for the proposed building with the lower portion of the foundation walls having a groundwater infiltration control system in place. Due to the presence of a groundwater suppression system in place against the bedrock face, long-term groundwater lowering is anticipated to be negligible for the area. Therefore, no adverse effects to neighbouring properties are expected.

The buildings located within adjacent properties are anticipated to be founded within the glacial till or directly on the bedrock. Therefore, no adverse effects to the surrounding buildings or properties are expected with the minor dewatering of the groundwater from this development.

6.6 Winter Construction

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

The subsurface conditions 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 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.

The trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

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.
- Review proposed waterproofing and foundation drainage design and requirements for water suppression system.
- Review of waterproofing details for elevator shaft and building sump pits.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- 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 work 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 provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Scott Street Development Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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

LICENSED PROFESSIONAL ENGINEER
March 25, 2021
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution

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☐ Paterson Group

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 II - Environmental Site Assessment
2046 & 2050 Scott St. and 295-301 Ashton Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 19, 2020

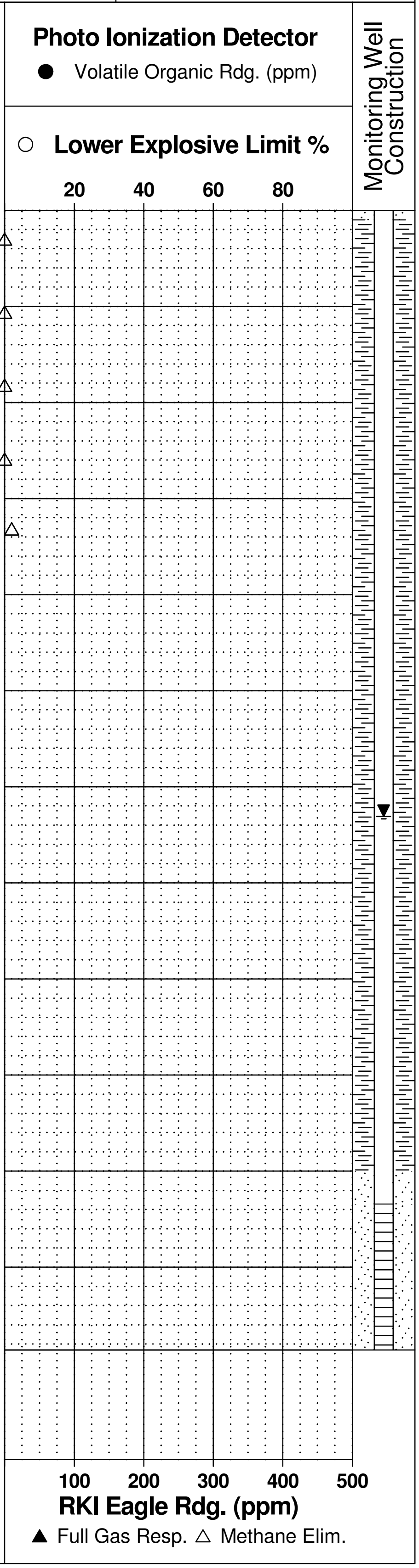
FILE NO. PE4892

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone, grave and cobbles	2.29	AU	1		
		SS	2	46	52
		SS	3	42	71
GLACIAL TILL: Dense, brown silty sand with gravel, some cobbles	3.81	SS	4	54	40
		SS	5	24	55
BEDROCK: Fair to good quality, grey limestone	11.86	RC	1	100	71
		RC	2	100	89
		RC	3	100	85
		RC	4	100	55
		RC	5	100	79
		RC	6	100	71
End of Borehole					

(GWL @ 6.31m - March 26, 2020)

DEPTH (m)	ELEV. (m)
0	63.31
1	62.31
2	61.31
3	60.31
4	59.31
5	58.31
6	57.31
7	56.31
8	55.31
9	54.31
10	53.31
11	52.31



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

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
2046 & 2050 Scott St. and 295-301 Ashton Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

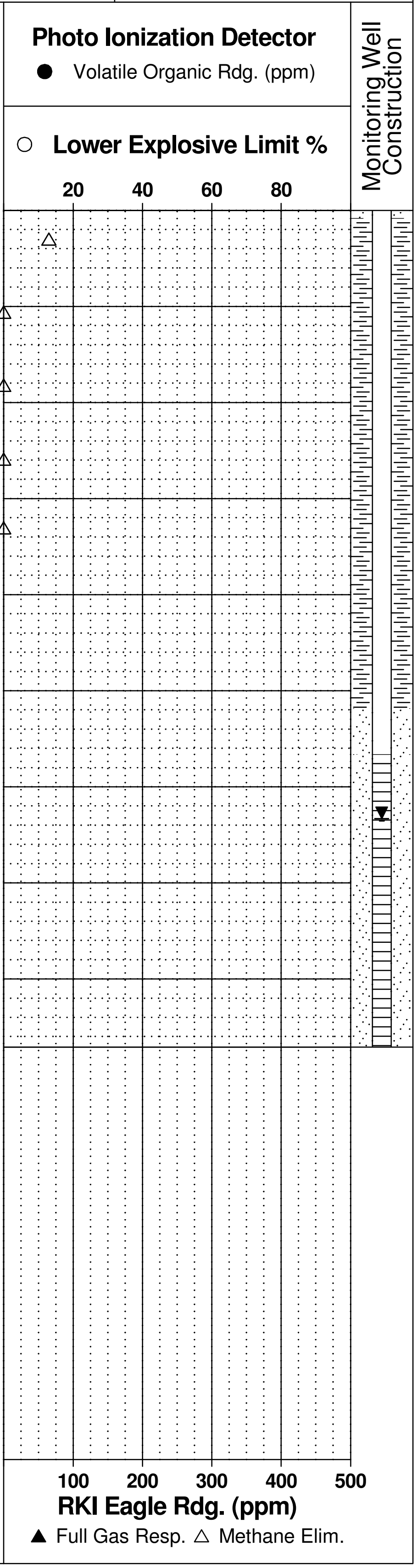
DATE March 19, 2020

FILE NO. PE4892

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.39
25mm Asphaltic concrete	AU	1				
FILL: Brown silty sand with crushed stone	SS	2	25	13	1	62.39
1.37						
Loose, brown **SILTY SAND**	SS	3	46	4		
2.13					2	61.39
GLACIAL TILL: Dense, grey-brown silty sand with gravel and cobbles	SS	4	38	50		
	SS	5	10	47	3	60.39
3.56						
BEDROCK: Poor to good quality, grey limestone	RC	1	100	34	4	59.39
	RC	2	100	6	5	58.39
					6	57.39
	RC	3	100	76	7	56.39
	RC	4	100	47	8	55.39
8.71						
End of Borehole						

(GWL @ 6.33m - March 26, 2020)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
2046 & 2050 Scott St. and 295-301 Ashton Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

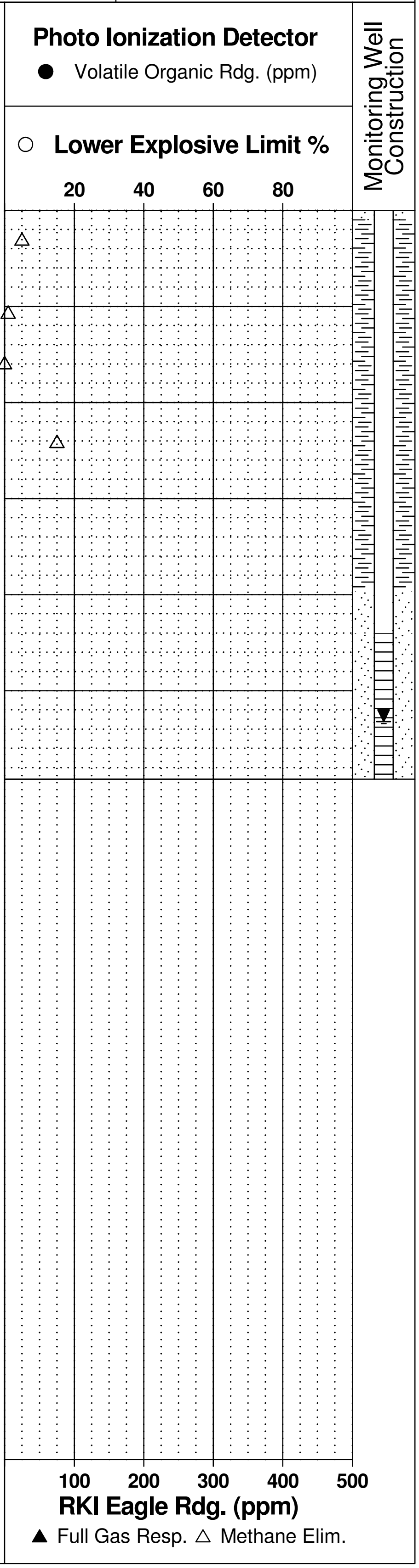
DATE March 20, 2020

FILE NO. PE4892

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Brown silty sand with crushed stone	2.29	SS	2	17	14
		SS	3	60	50+
GLACIAL TILL: Very dense, brown silty sand with gravel, trace clay	3.22	SS	4	56	50+
BEDROCK: Poor to good quality, grey limestone	5.92	RC	1	100	47
		RC	2	100	86
End of Borehole					
(GWL @ 5.32m - March 26, 2020)					

DEPTH (m)	ELEV. (m)
0	63.03
1	62.03
2	61.03
3	60.03
4	59.03
5	58.03



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
2046 & 2050 Scott St. and 295-301 Ashton Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

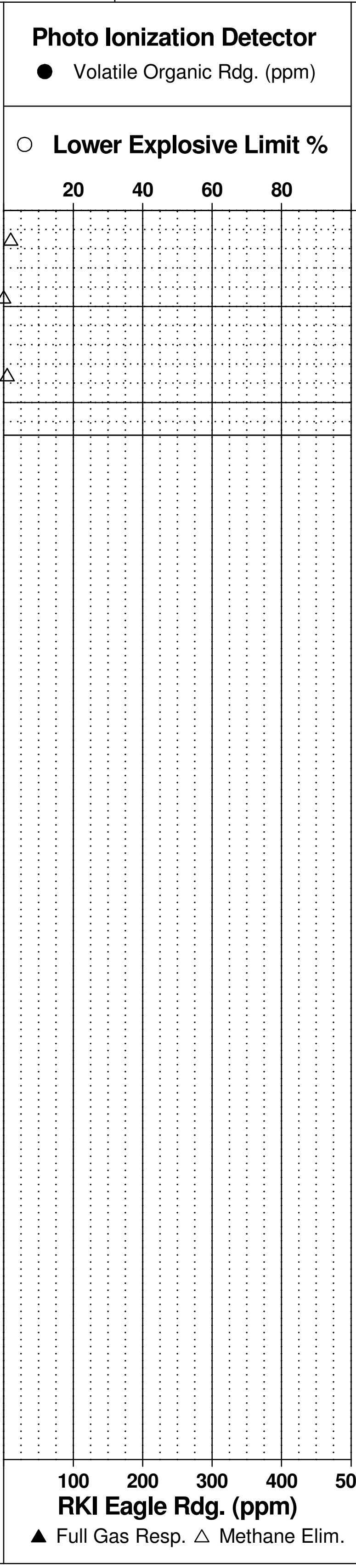
BORINGS BY CME-55 Low Clearance Drill

DATE March 19, 2020

FILE NO. PE4892

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.35
FILL: Brown silty sand with crushed stone, trace cobbles	AU	1				
	SS	2	9	50+	1	62.35
1.52						
GLACIAL TILL: Very dense, brown silty sand with gravel and clay	SS	3	33	50+	2	61.35
2.34						
End of Borehole Practical refusal to augering at 2.34m depth						



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
2046 & 2050 Scott St. and 295-301 Ashton Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

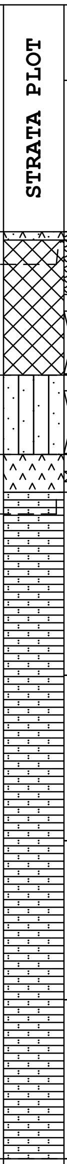
DATE March 19, 2020

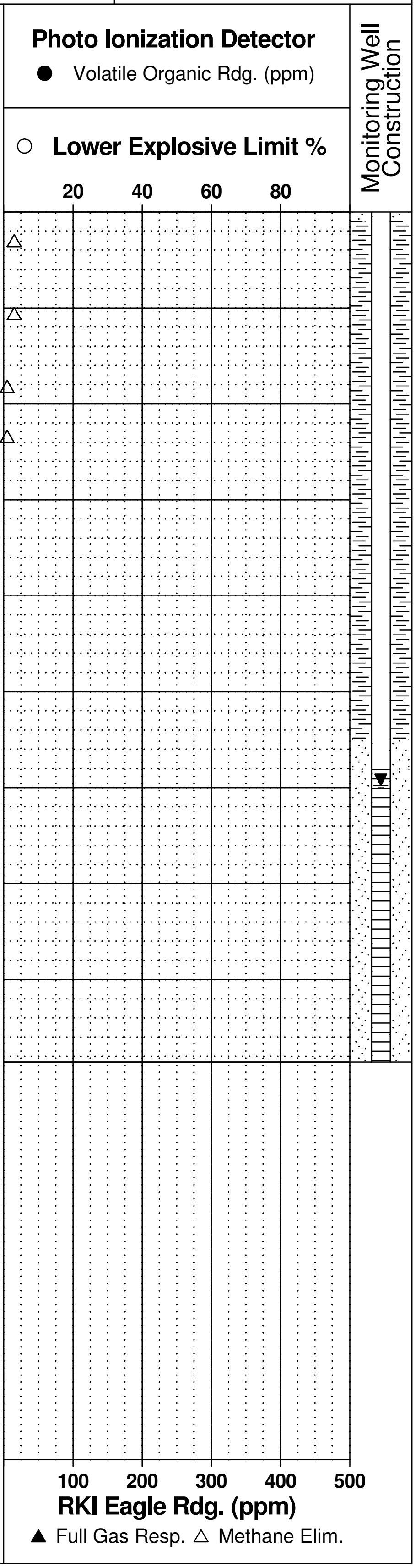
FILE NO. PE4892

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.08	AU	1		
FILL: Brown silty sand with crushed stone, trace gravel 1.37	SS	2	54	8
Compact, brown **SILTY SAND** 2.13	SS	3	62	28
GLACIAL TILL: Dense, brown silty sand with gravel and clay 2.49	SS	4	25	50+
BEDROCK: Fair to good quality, grey limestone	RC	1	86	54
	RC	2	100	64
	RC	3	100	78
	RC	4	100	82
8.86 End of Borehole				
(GWL @ 5.98m - March 26, 2020)				

DEPTH (m)	ELEV. (m)
0	63.04
1	62.04
2	61.04
3	60.04
4	59.04
5	58.04
6	57.04
7	56.04
8	55.04





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)

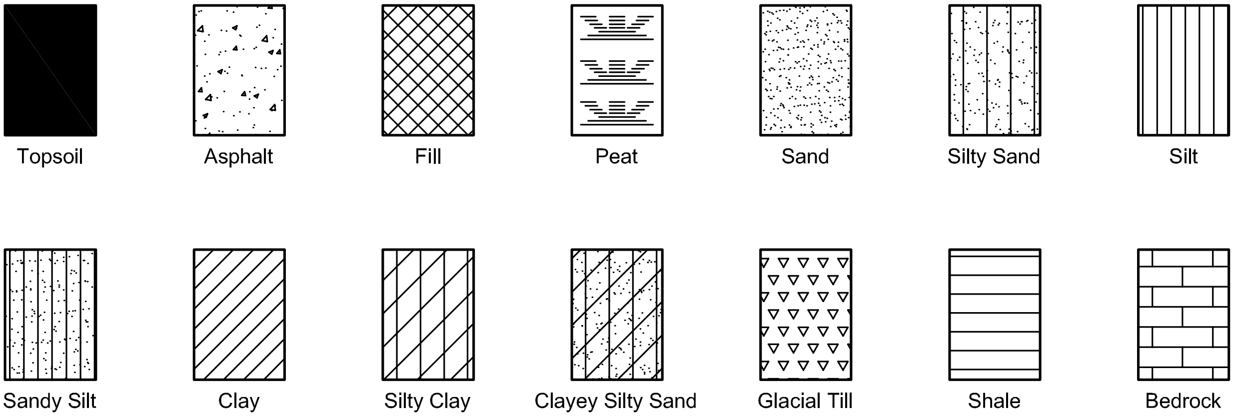
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

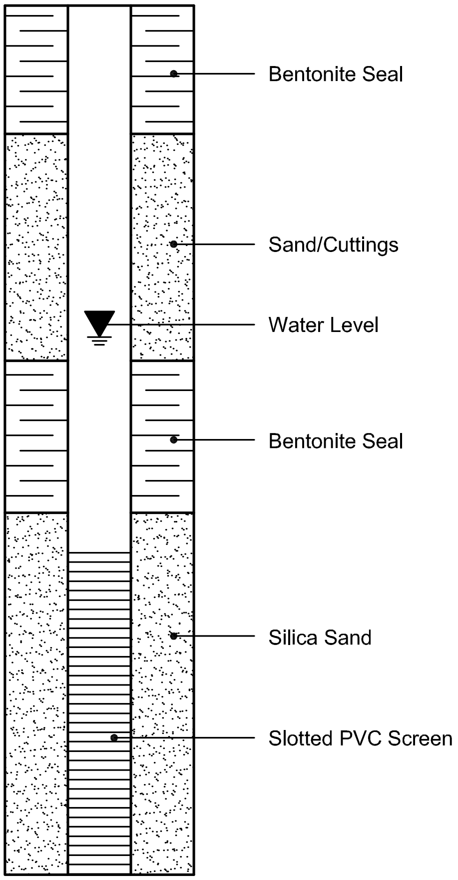
SYMBOLS AND TERMS (continued)

STRATA PLOT

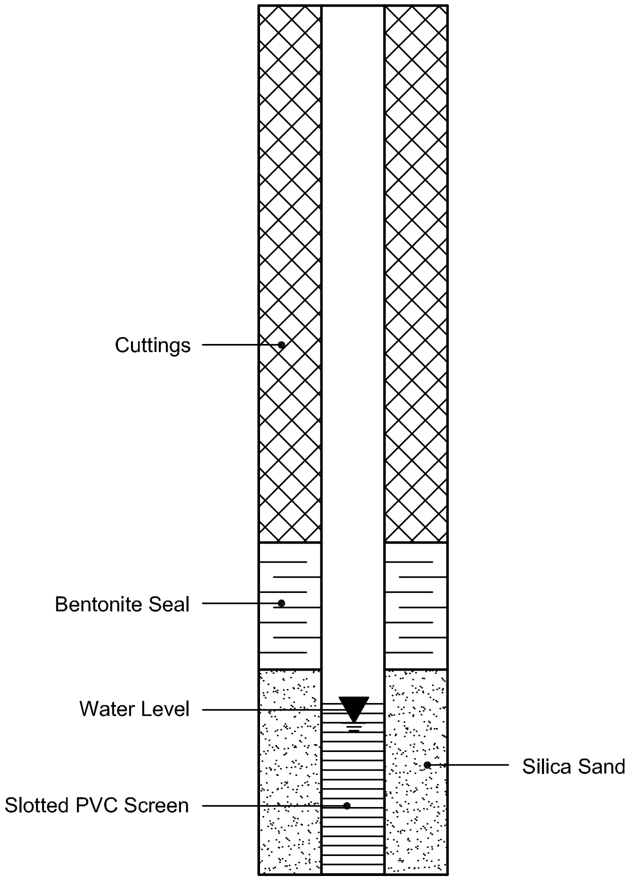


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

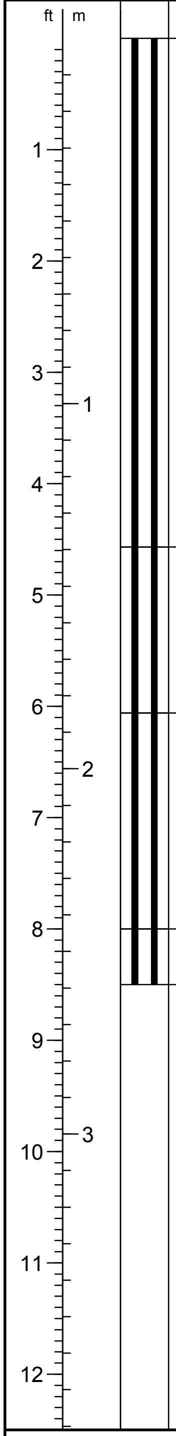


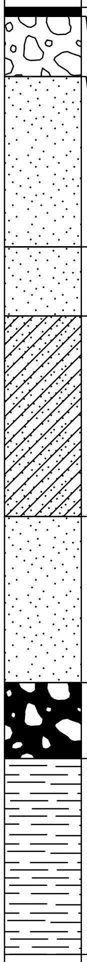
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-01

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441011.59E, 5027169.68N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A





LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	STRATIGRAPHIC DESCRIPTION	INSTALLATION
				GROUND SURFACE	
				Thin layer of asphalt	BH18-01
				FILL gravel	
				Poor recovery, sand with gravel, dark brown	
X		0	0	Sand, loose, light brown	
				Clayey sand with some red brick, grey	
		0	0	Sand, loose, light brown	
				Boulder	
		0	0	Native Clay, firm, grey, moist	
		5	0		
				BOREHOLE TERMINATED Total Depth of BH18-01 2.59 mBGS	

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-02

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441013.55E, 5027167.02N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
ft | m							GROUND SURFACE	BH18-02
							Thin layer of ashphalt	
							FILL Gravel	
1				0	0		Sand with gravel, dark brown	
2								
3 1				0	0			
4							Boulder	
5								
6		X		0	0		Sand, loose, light brown	
2								
7							Wet	
8				0	0		Native Sandy silt, grey	
9							BOREHOLE TERMINATED Total Depth of BH18-02 2.59 mBGS	
3 10								
11								
12								

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma
Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-03

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441015.8E, 5027164.22N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
ft m							GROUND SURFACE	BH18-03
							Asphalt	
				0	0		Sand with gravel, loose, brown to grey	
				0	0		Sand with some gravel, loose, brown	
				0	0		Sand, loose, brown	
							Sand, loose, brown	
				0	0		Native Silty sand, firm, grey	
				0	0		Sandy silt, very firm, grey	
							BOREHOLE TERMINATED Total Depth of BH18-03 2.90 mBGS	

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-04

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441018.74E, 5027148.51N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A

DEPTH BGS (ft / m)	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	BH18-04
1, 2, 3 ft				5	1		Asphalt FILL Gravel Sand with Boulder (poor recovery), dark brown	
4, 5 ft		X		80	1		Clayey sand, grey Sand with some gravel, dark brown Sand, loose, light brown	
6, 7 ft / 2 m				0	1		Sand lenses, grey to brown Silty sand, grey Damp	
8, 9, 10, 11 ft / 3 m				0	1		Native Sandy silt with boulders, damp, grey	
12 ft							BOREHOLE TERMINATED Total Depth of BH18-04 3.51 mBGS	

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

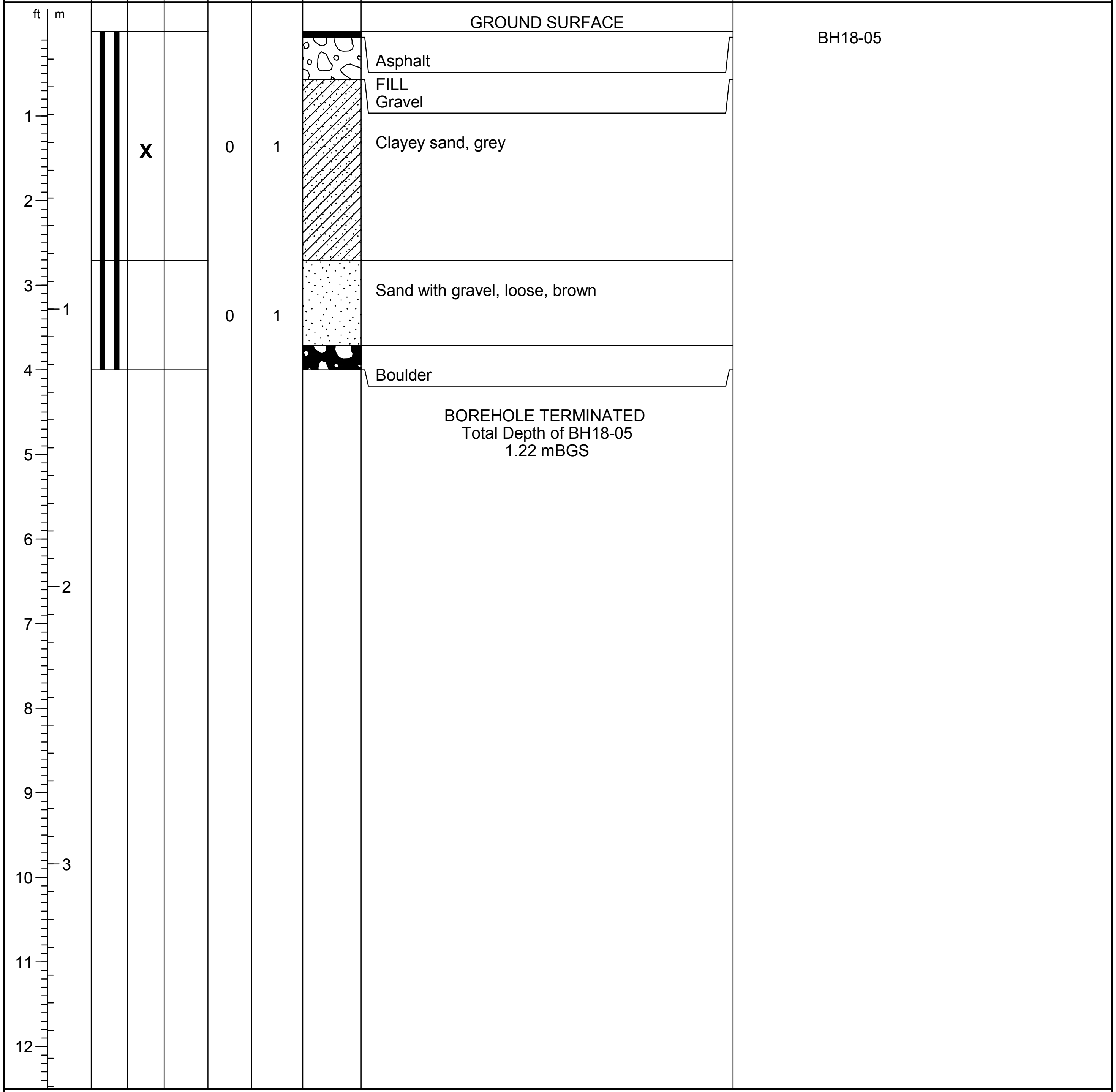
Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-05

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441017.48E, 5027149.07N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A



DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	BH18-05
							Asphalt	
							FILL Gravel	
		X		0	1		Clayey sand, grey	
				0	1		Sand with gravel, loose, brown	
							Boulder	
							BOREHOLE TERMINATED Total Depth of BH18-05 1.22 mBGS	

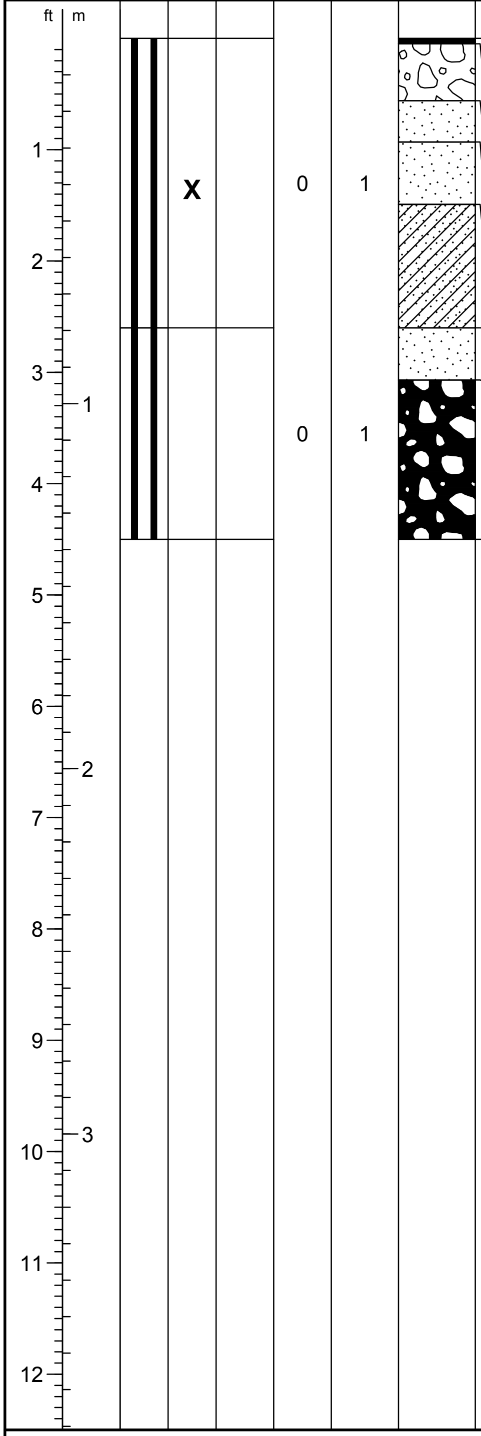
Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-06

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441010.47E, 5027145.43N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A



DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	BH18-06
							Asphalt	
							FILL Gravel with sand and ash	
		X		0	1		Sand with some gravel, loose, dark brown	
							Sand, loose, light brown	
							Clayey sand, black staining, brick present, grey	
							Sand, compact, brown	
				0	1		Boulder (Refusal)	

BOREHOLE TERMINATED
Total Depth of BH18-06
1.37 mBGS

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

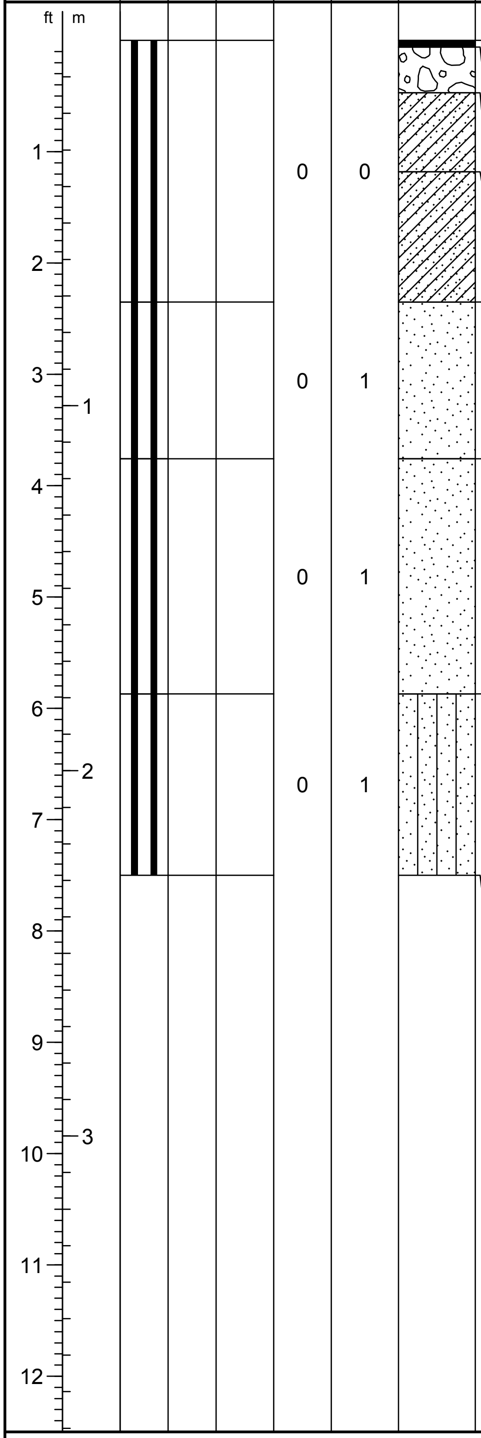
Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-07

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441011.03E, 5027144.17N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A



CGI (ppm)	PID (ppm)	STRATIGRAPHIC DESCRIPTION	INSTALLATION
		GROUND SURFACE	BH18-07
		Asphalt	
		FILL Gravel with sand and ash	
0	0	Clayey sand, brown grey	
		Clayey sand with black staining, brown grey	
0	1	Sand with boulders, brown	
0	1	Sand, fine grained, loose, light brown grey	
0	1	Native Sandy silt, damp, grey	
		Damp	
		BOREHOLE TERMINATED Total Depth of BH18-07 2.29 mBGS	

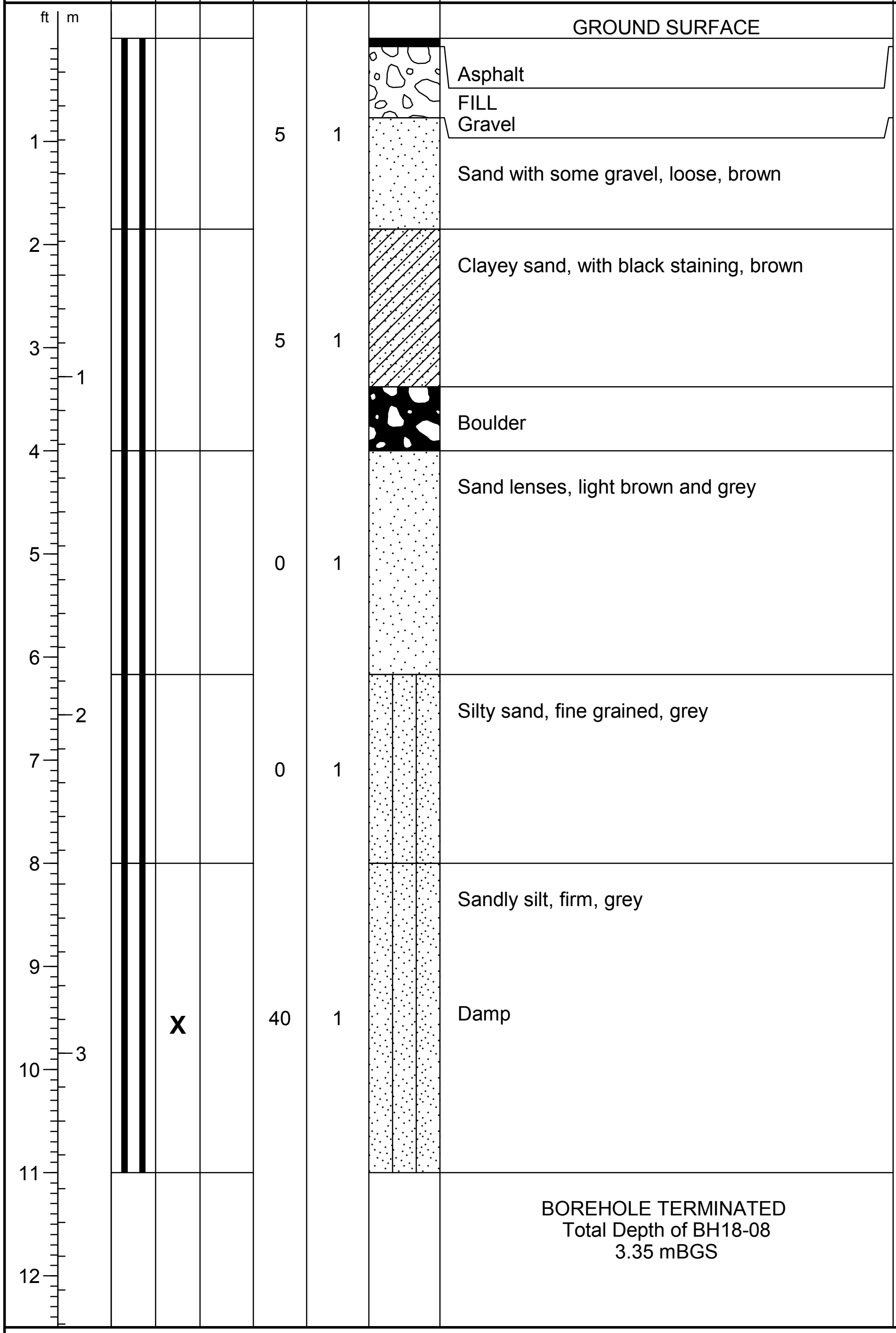
Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-08

Project Number: 18-214-1	MOE Well ID: N/A
Client: Bob Peter's Garage Inc.	Date Completed: 6-Apr-18
Site Location: 2046 Scott St., Ottawa, Ontario	Supervisor: GDB
Coordinates: 441016.08E, 5027157.91N (MTM Zone 18)	Logged By: TKG
Drilling Method: Direct Push	Ground Surface Elevation: N/A
Drilling Rig: Geoprobe 540	Date of Water Level Measurement: N/A



SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	STRATIGRAPHIC DESCRIPTION	INSTALLATION
					GROUND SURFACE	BH18-08
			5	1	Asphalt FILL Gravel Sand with some gravel, loose, brown	
			5	1	Clayey sand, with black staining, brown Boulder	
			0	1	Sand lenses, light brown and grey	
			0	1	Silty sand, fine grained, grey	
	X		40	1	Sandly silt, firm, grey Damp	
					BOREHOLE TERMINATED Total Depth of BH18-08 3.35 mBGS	

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-09

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441009.49E, 5027163.93N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A

DEPTH BGS ft	DEPTH BGS m	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
								GROUND SURFACE	BH18-09
								Concrete	
								FILL Gravel	
1					0	1		Sand, loose, brown	
2								Clayey sand, black staining, black, brown	
3			X		15	11		Clayey sand, brown	
	1								
4								Gravel	
								Sand, loose, brown	
5								Sand, loose, grey brown	
					0	0			
6									
	2							Sand, loose, light brown	
7								Native Sandy silt, firm, grey	
					0	0			
8									
			X		25	2		Wet	
9									
					20	0			
10	3								
								BOREHOLE TERMINATED Total Depth of BH18-09 3.05 mBGS	
11									
12									

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

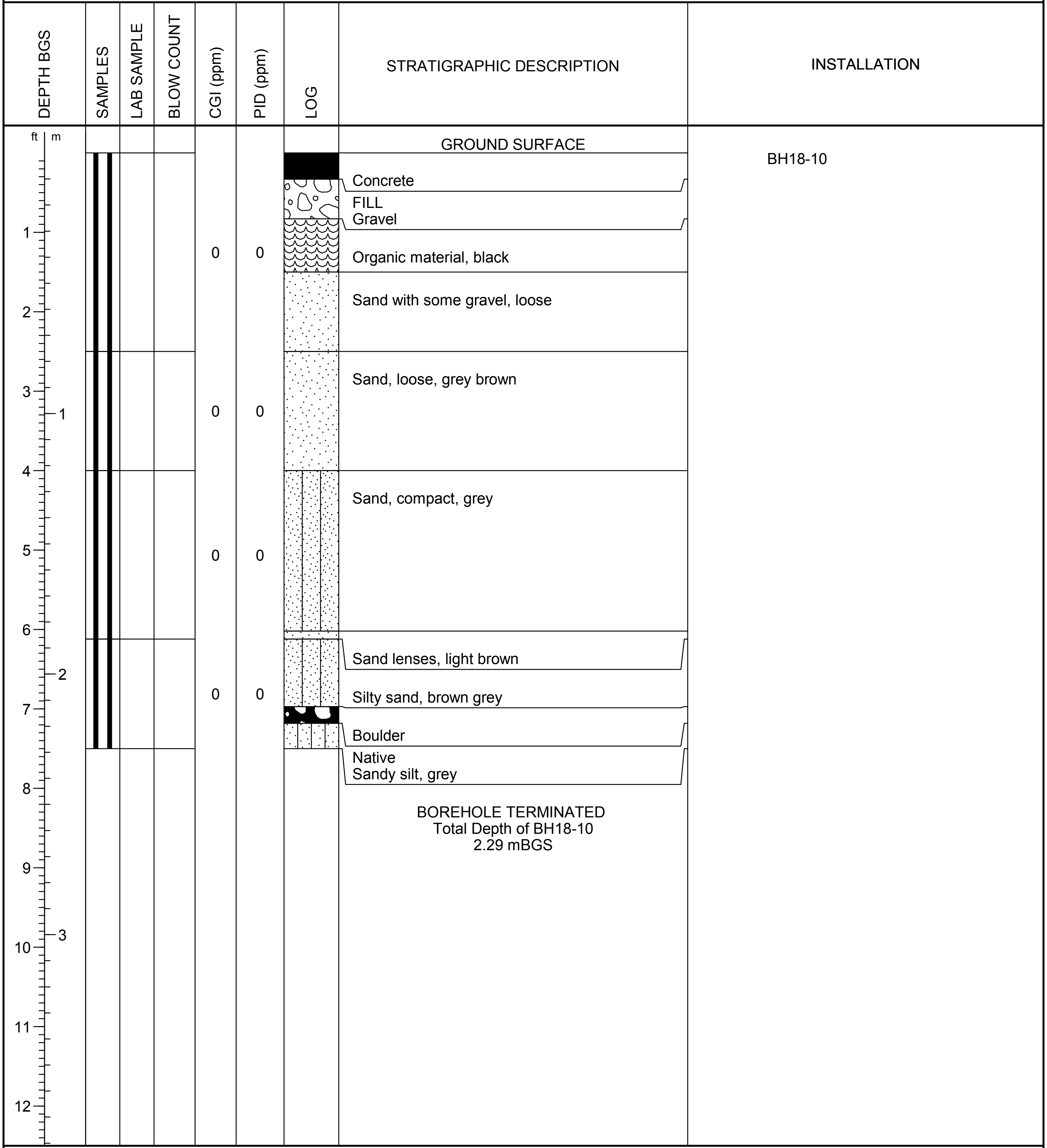
Geofirma
Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-10

Project Number: 18-214-1
Client: Bob Peter's Garage Inc.
Site Location: 2046 Scott St., Ottawa, Ontario
Coordinates: 441005.56E, 5027154.4N (MTM Zone 18)
Drilling Method: Direct Push
Drilling Rig: Geoprobe 540

MOE Well ID: N/A
Date Completed: 6-Apr-18
Supervisor: GDB
Logged By: TKG
Ground Surface Elevation: N/A
Date of Water Level Measurement: N/A



CGI (ppm)	PID (ppm)
0	0
0	0
0	0
0	0

STRATIGRAPHIC DESCRIPTION

GROUND SURFACE

Concrete

FILL
Gravel

Organic material, black

Sand with some gravel, loose

Sand, loose, grey brown

Sand, compact, grey

Sand lenses, light brown

Silty sand, brown grey

Boulder

Native
Sandy silt, grey

BOREHOLE TERMINATED
Total Depth of BH18-10
2.29 mBGS

INSTALLATION

BH18-10

Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

Log of Borehole: MW-1

Project #: 232288.001 **Logged By:** RL

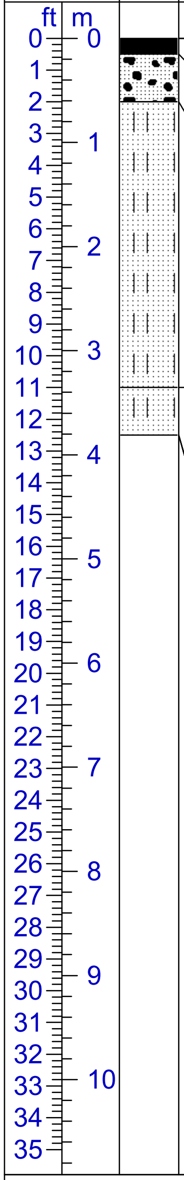
Project: Phase II Environmental Site Assessment

Client: 347313 Canada Inc.

Location: 2050 Scott Street, Ottawa, Ontario

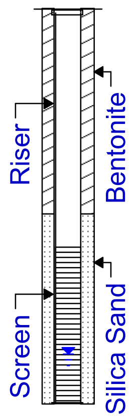
Drill Date: November 22, 2018

SUBSURFACE PROFILE



Description	Measured Depth (m)
Ground Surface	0.00
Asphalt	
Sandy Gravel Brown, moist, no PHC odour or staining.	0.61
Silty Sand Brown, gravel throughout, no PHC odour or staining.	3.35
Silty Sand Brown, gravel throughout, turning wet, no PHC odour or staining. Refusal on Bedrock.	3.81
End of Borehole	

Monitoring Well Details



Water level measured at 3.32 mbgs on December 11, 2018.

SAMPLE

Recovery (%)	Sample ID	Soil Vapour Concentration (ppm) PID	Laboratory Analysis
60	SS1	0	
60	SS2	0	pH
70	SS3	0	Grain Size
70	SS4	0	pH
60	SS5	0	PHCs, VOCs, PAHs

Soil vapour concentrations measured using a photoionization detector (PID).

Contractor: Strata Drilling Group

Drilling Method: Geo-probe

Well Casing Size: 5.08 cm

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW-2

Project #: 232288.001 **Logged By:** RL

Project: Phase II Environmental Site Assessment

Client: 347313 Canada Inc.

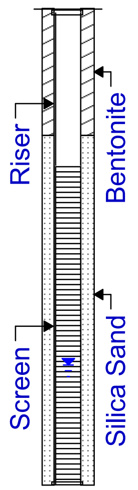
Location: 2050 Scott Street, Ottawa, Ontario

Drill Date: November 22, 2018

SUBSURFACE PROFILE

Description	Measured Depth (m)
Ground Surface	0.00
Asphalt	
Sandy Gravel Brown, moist, no PHC odour or staining.	0.61
Silty Sand Brown, gravel throughout, moist, no PHC odour or staining.	2.74
Silty Sand Brown, gravel throughout, wet, PHC odour, no staining. Refusal on Bedrock.	4.57
End of Borehole	

Monitoring Well Details



Water level measured at 3.42 mbgs on December

SAMPLE

Recovery (%)	Sample ID	Soil Vapour Concentration (ppm) PID	Laboratory Analysis
50	SS1	0	
50	SS2	0	
60	SS3	0	
60	SS4	78	
70	SS5	118	PHCs, VOCs, PAHs, TCLP
70	SS6	62	

Soil vapour concentrations measured using a photoionization detector (PID).

Contractor: Strata Drilling Group

Drilling Method: Geo-probe

Well Casing Size: 5.08 cm

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW-3

Project #: 232288.001 **Logged By:** RL

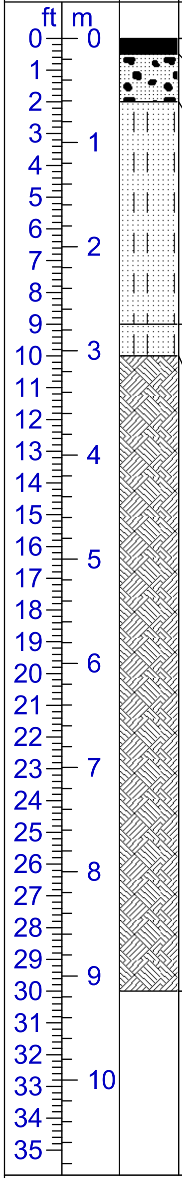
Project: Phase II Environmental Site Assessment

Client: 347313 Canada Inc.

Location: 2050 Scott Street, Ottawa, Ontario

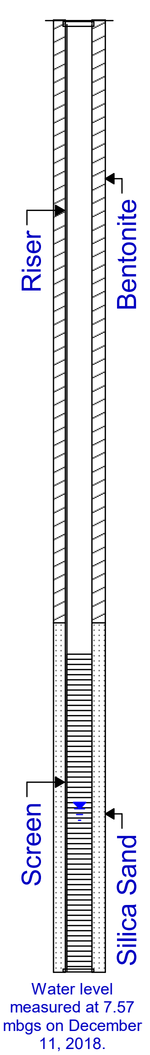
Drill Date: November 22, 2018

SUBSURFACE PROFILE



Description	Measured Depth (m)
Ground Surface	0.00
Asphalt	
Sandy Gravel Brown, moist, no PHC odour or staining.	0.61
Silty Sand Brown, gravel throughout, moist, no PHC odour or staining.	2.74
Silty Sand Brown, gravel throughout, turning wet, no PHC odour or staining.	3.05
Bedrock Down hole hammer to 30'. Refusal on Bedrock.	9.14
End of Borehole	

Soil vapour concentrations measured using a photoionization detector (PID).



Water level measured at 7.57 mbgs on December 11, 2018.

SAMPLE

Recovery (%)	Sample ID	Soil Vapour Concentration (ppm) PID	Laboratory Analysis
70	SS1	0	
70	SS2	0	
60	SS3	0	
60	SS4	0	PHCs, VOCs, PAHs

Contractor: Strata Drilling Group

Drilling Method: Geo-probe

Well Casing Size: 5.08 cm

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: MW-4

Project #: 232288.001 **Logged By:** RL

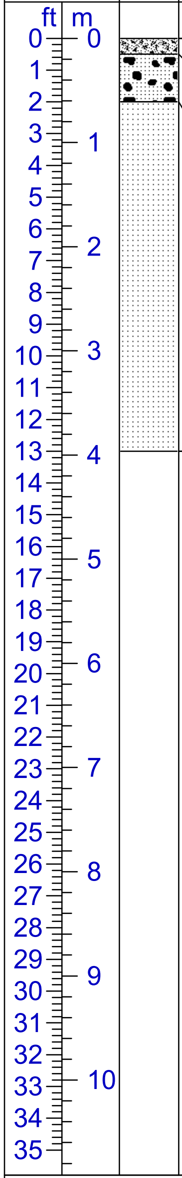
Project: Phase II Environmental Site Assessment

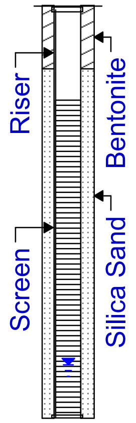
Client: 347313 Canada Inc.

Location: 2050 Scott Street, Ottawa, Ontario

Drill Date: December 3, 2018

SUBSURFACE PROFILE			SAMPLE			
Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm) PID	Laboratory Analysis
Ground Surface	0.00					
Concrete						
Sandy Gravel Brown, moist, no PHC odour or staining.	0.61		30	SS1	0	
Sand Brown, moist, no PHC odour or staining.			30	SS2	0	
			40	SS3	0	
			40	SS4	48	
Turning wet and PHC odour @ 13'. Refusal on Bedrock.	3.96		40	SS5	268	PHCs, VOCs, PAHs
End of Borehole		Water level measured at 3.45 mbgs on December 11, 2018.				





Soil vapour concentrations measured using a photoionization detector (PID).

Contractor: Strata Drilling Group

Drilling Method: Geo-probe

Well Casing Size: 5.08 cm

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - WATER SUPPRESSION SYSTEM

FIGURE 3 - FOUNDATION DRAINAGE SYSTEM

DRAWING PG5222-1 - TEST HOLE LOCATION PLAN

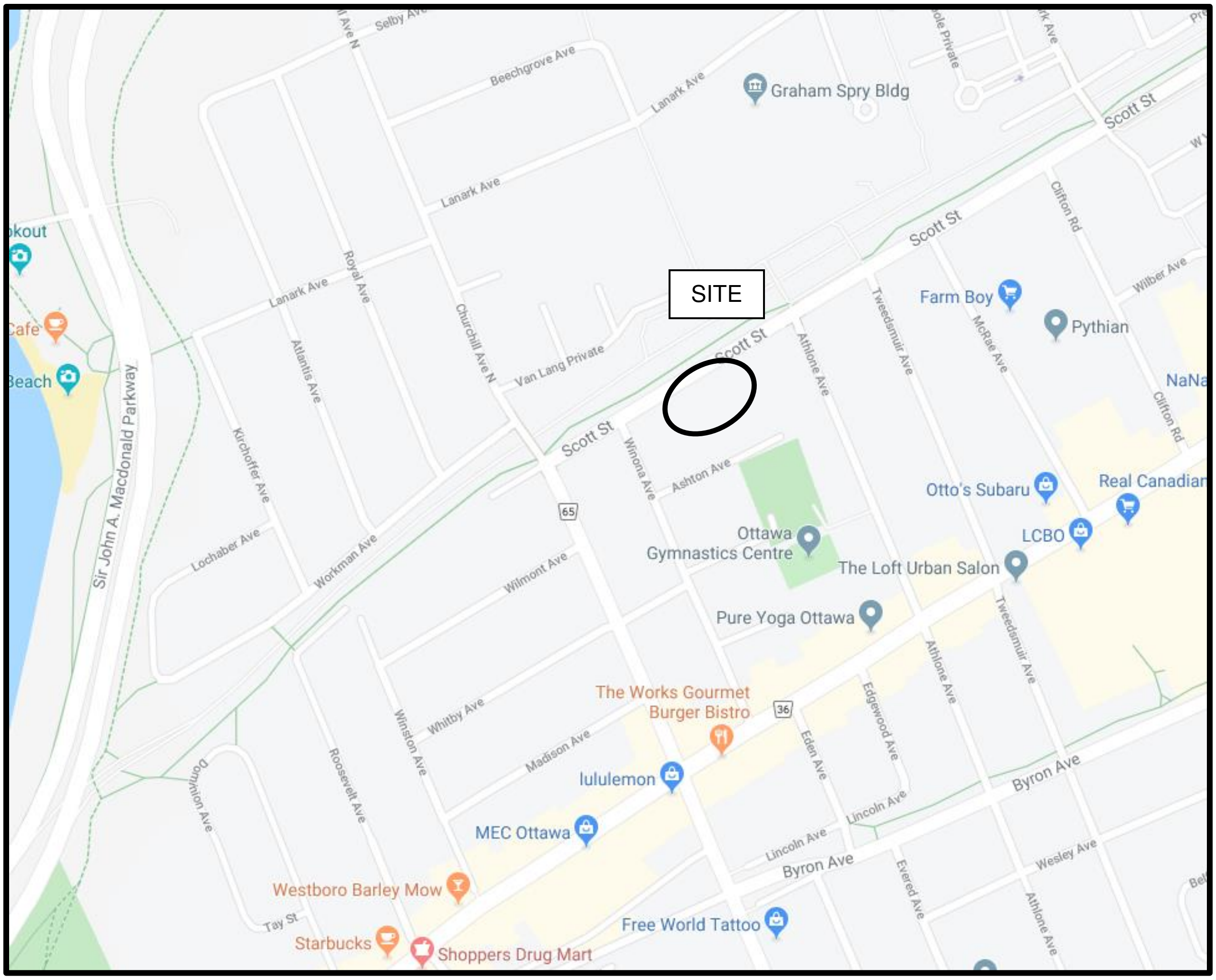
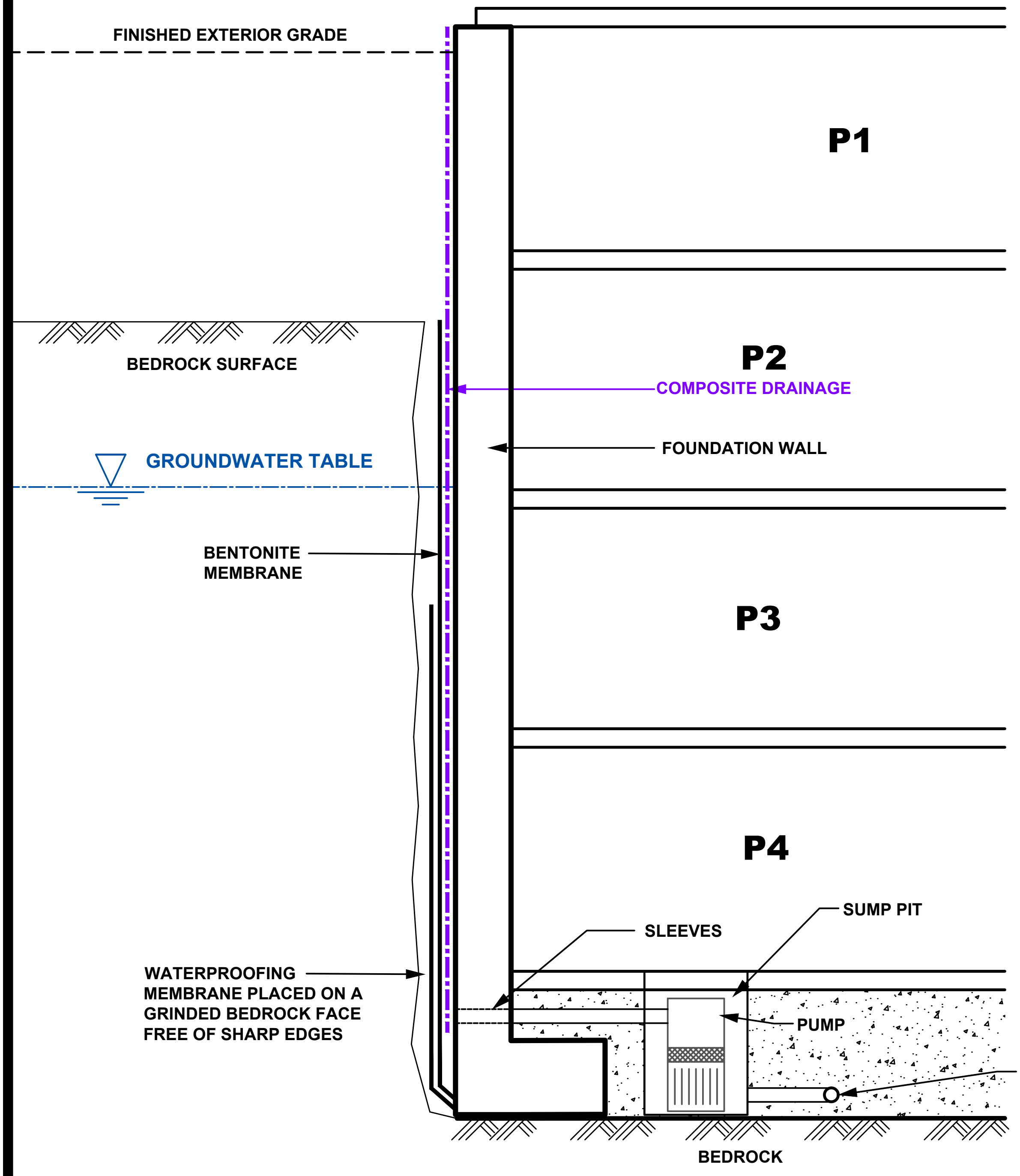


FIGURE 1

KEY PLAN



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SCOTT STREET DEVELOPMENT INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
2046 & 2050 SCOTT STREET
OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM

Date: 03/2021

Report No.: PG5222-1

Scale: N.T.S

Drawing No.: FIGURE 2

Drawn by: NFRV

Checked by: JV

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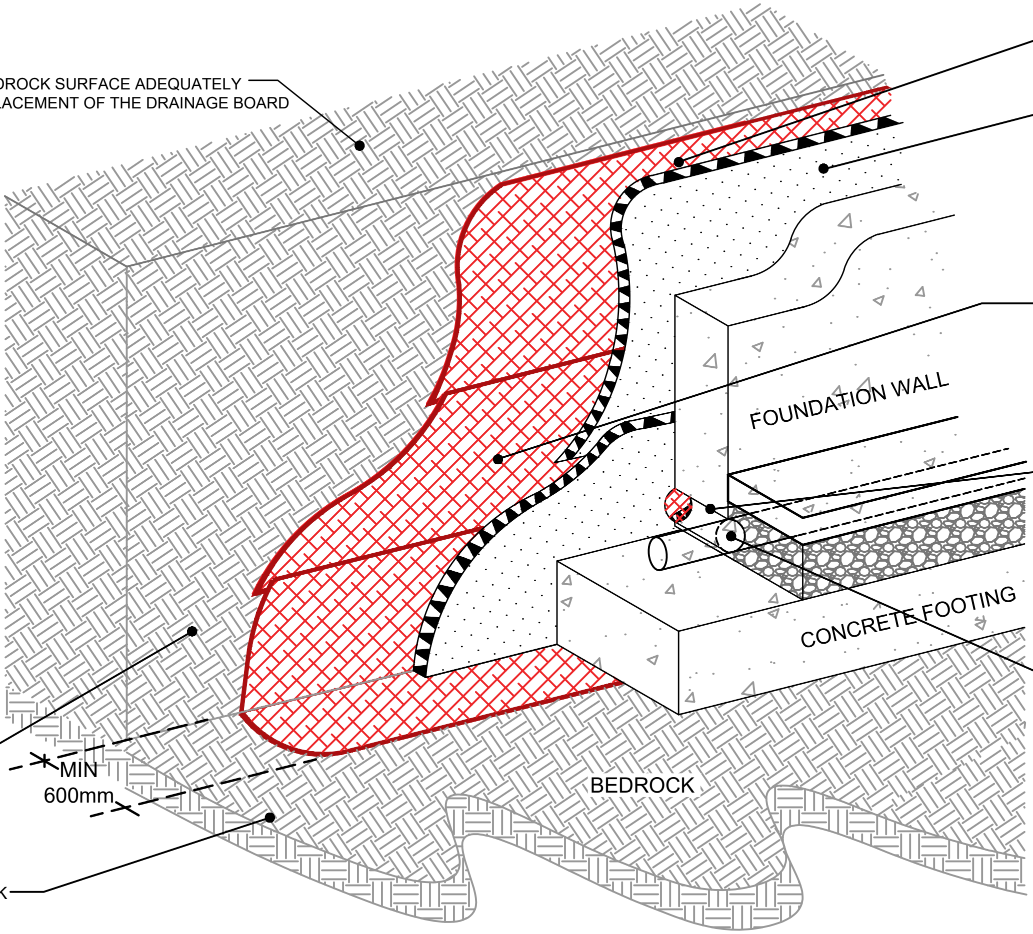

SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE DRAINAGE BOARD

GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE BEDROCK

COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

15miL THICK EXPANDABLE, GRANULAR BENTONITE AND HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. THE LAYER SHOULD BE DOUBLED FOR THE FOURTH UNDERGROUND PARKING LEVEL.

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT

SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM .

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www.patersongroup.ca

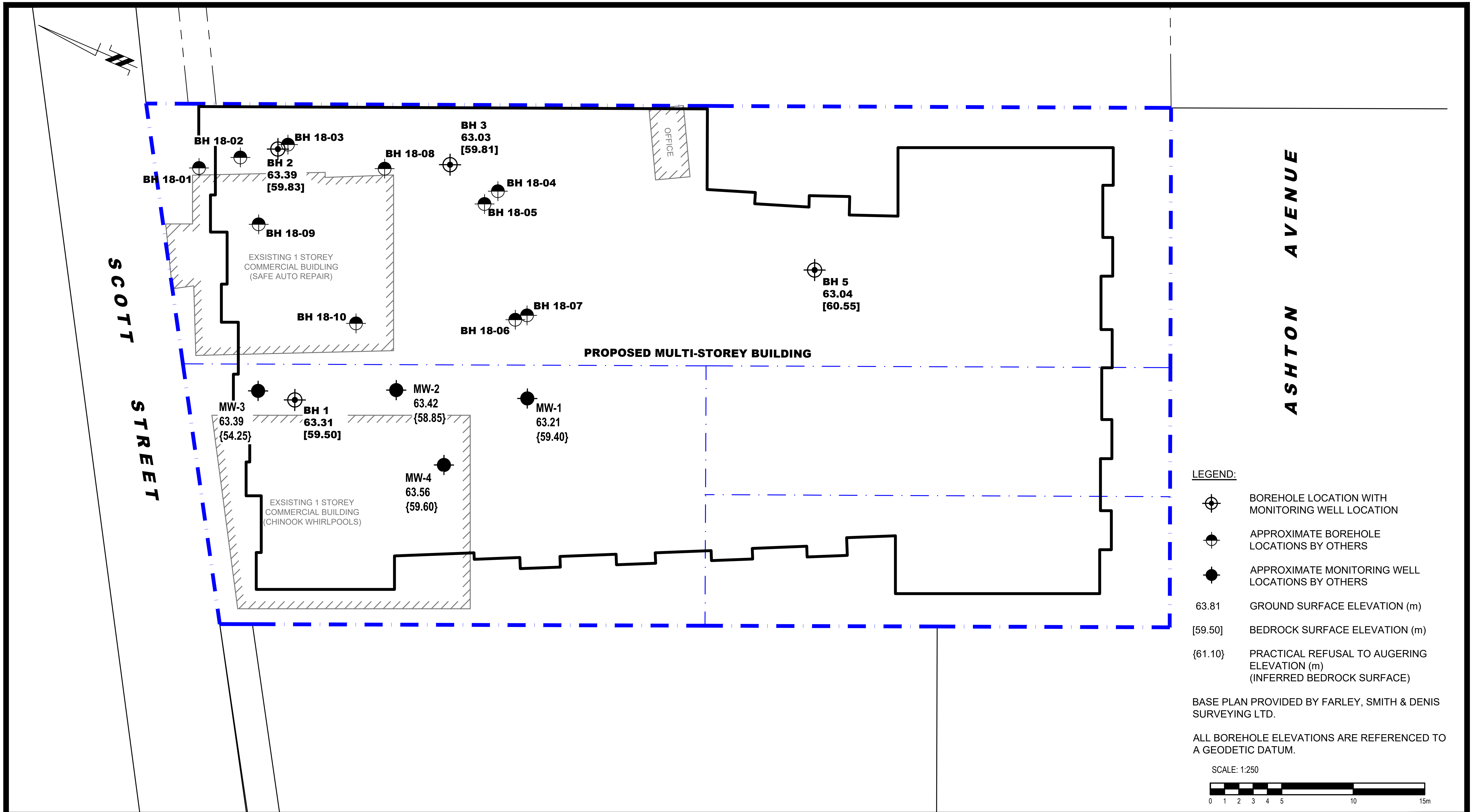
SCOTT STREET DEVELOPMENT INC.
PROPOSED MULTI STOREY BUILDING
2046 / 2050 SCOTT STREET
OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM

Scale: N.T.S.
Date: 03/2021
Drawn by: NFRV
Report No.: PG5222-1
Checked by: JV
Drawing No.: FIGURE 3
Approved by: DJG
Revision No.:

p:\autocad drawings\geotechnical\pg52xx\pg5222\pg5222 figure 3 - foundation drainage.dwg



LEGEND:

⊕	BOREHOLE LOCATION WITH MONITORING WELL LOCATION
⊕	APPROXIMATE BOREHOLE LOCATIONS BY OTHERS
●	APPROXIMATE MONITORING WELL LOCATIONS BY OTHERS
63.81	GROUND SURFACE ELEVATION (m)
[59.50]	BEDROCK SURFACE ELEVATION (m)
{61.10}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m) (INFERRED BEDROCK SURFACE)

BASE PLAN PROVIDED BY FARLEY, SMITH & DENIS SURVEYING LTD.

ALL BOREHOLE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

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NO.	REVISIONS	DATE	INITIAL
1	REVISED SITE PLAN	17/03/2021	JV

SCOTT STREET DEVELOPMENT INC.

GEOTECHNICAL INVESTIGATION - PROPOSED MULTI STOREY BUILDING
2046 / 2050 SCOTT STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 01/2020
Drawn by: NFRV	Report No.: PG5222-1
Checked by: JV	Dwg. No.: PG5222-1
Approved by: DJG	Revision No.: 1