Geotechnical Engineering

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# Geotechnical Report

Proposed Hi-Rise Buildings
2046/2050 Scott Street
Ottawa, Ontario

Prepared For

Scott Street Developments Inc.

April 29, 2020

Report: PG5222-1 Revision 1

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# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Scott Street Developments Inc. to review existing subsoil information and prepare a geotechnical report for the proposed hi-rise building to be constructed at 2046 and 2050 Scott Street, 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 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 available information, the proposed development consists of a hi-rise building with 2 to 3 underground parking and storage levels. The development will also include associated at grade asphaltic parking areas, access lanes and landscaped areas. It is further anticipated that the site will be municipally serviced.

# 3.0 Available Information

Previous geotechnical field investigations were completed by other firms for the subject site. The investigation consisted of advancing a total of 14 boreholes across the subject site to a maximum depth of 9.1 m below existing grade. Locations of the test holes completed by others on site are shown on Drawing PG5222-1 - Test Hole Location Plan attached in Appendix 2.

## Surface Conditions

The site is currently occupied by 2 single storey commercial buildings with associated access lanes and parking areas. The ground surface across the subject site is relatively flat and at grade with Scott Street. It should be noted that the subject site is bordered from the west and south by residential buildings, to the east by a sporting facility and to the north by Scott Street.

## Subsurface Profile

Based on the borehole coverage completed by others and our experience with the surrounding sites, the subsurface profile at the test hole locations consists of a pavement structure and/or fill consisting of an assortment of silty sand, gravel, clayey sand and boulders. Fill containing brick fragments and other deleterious materials was noted at various test hole locations. A native layer of grey silty sand was encountered overlaying a limestone bedrock with interbedded dolostone. The refusal on bedrock was encountered between 2.3 m and 3.4 m throughout the site. Specific details of the soil profile at each test hole location are presented on the borehole logs provided by others in Appendix 1.

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.

## Groundwater

Based on Paterson's review of the available groundwater measurements, the long-term groundwater is expected to be at a depth ranging between 5 to 6 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

# 4.0 Discussion

## 4.1 Geotechnical Assessment

### Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed hi-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 may be required to complete the underground level. Hoe ramming is an option where only small quantities of bedrock need to be removed. 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 further discussed in the following sections.

## 4.2 Site Grading and Preparation

### Stripping Depth

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

### Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or 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 of 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 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.

### Vibration Considerations

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

The following construction equipments 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.

### Fill Placement

Fill placed for grading beneath the building area 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 fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material’s standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. 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 paved areas should be compacted to at least 95% of its SPMDD.

## 4.3 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 founded on **a clean limestone bedrock** bearing surface and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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 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 weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

## 4.4 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. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

## 4.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and a rigid pavement structure designed by a structural engineer will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab fill consists 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 at least 98% of its SPMDD.

## 4.6 Basement Wall

It is expected that the basement walls will be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.01 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 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.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

## Static Conditions

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
$\gamma$ = 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 Conditions

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

The seismic earth force ($\Delta P_{AE}$) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c$ = $(1.45-a_{max}/g)a_{max}$
$\gamma$ = 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.

## 4.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in limestone and dolostone bedrock is based upon two possible failure modes. The rock anchor can fail by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock 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 individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should 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) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

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.

Regardless of whether an anchor is a passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the base, which will provide the capacity, and an free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops 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 sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

### Grout to Rock Bond

The unconfined compressive strength of limestone at the subject site ranges between 65 and 125 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, should be provided. A minimum grout strength of 40 MPa is recommended.

### Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 64** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

### Recommended Grouted Rock Anchor Lengths

Parameters used to calculate grouted rock anchor lengths are provided in Table 1.

**Table 1 - 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 and Dolostone<br>Hoek and Brown parameters | 64<br>m=0.575 and s=0.00293 |
| Unconfined compressive strength - Limestone | 65 MPa |
| Effective unit weight - Bedrock | 15 kN/m$^3$ |
| Apex angle of failure cone | 60$^o$ |
| Apex of failure cone | mid-point of fixed anchor length |

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 2. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

**Table 2 - 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 | 1.9 | 0.8 | 2.7 | 500 |
| | 2.6 | 1 | 3.6 | 750 |
| | 3.2 | 1.2 | 4.4 | 1000 |
| | 4.5 | 2 | 6.5 | 1500 |
| 125 | 1.6 | 0.6 | 2.2 | 500 |
| | 2 | 1 | 3 | 750 |
| | 2.2 | 1.3 | 3.5 | 1000 |
| | 3 | 1.8 | 4.8 | 1500 |

**Other Considerations**

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom to top of the anchor holes.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on test procedures 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.

## 4.8 Pavement Design

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.

# 5.0 Design and Construction Precautions

## 5.1 Foundation Drainage and Backfill

### Foundation Drainage

It is expected that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be poured against a drainage system placed against the shoring face.

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

### 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, we suggest a 150 mm in diameter perforated pipe with a geotextile sock be placed in each bay.

### 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.

## 5.2 Protection of Footings Against Frost Action

The parking garage is expected to not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may required to be insulated against the deleterious effect of 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. More details regarding foundation insulation can be provided, if requested.

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.

## 5.3 Excavation Side Slopes

### Temporary Side Slopes

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

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

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

## Temporary Shoring

Temporary shoring 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 5 - Soil Parameters for Shoring System Design**

| Parameters | Values |
|---|---|
| Active Earth Pressure Coefficient ($K_a$) | 0.33 |
| Passive Earth Pressure Coefficient ($K_p$) | 3 |
| At-Rest Earth Pressure Coefficient ($K_o$) | 0.5 |
| Unit Weight (γ), kN/m$^3$ | 20 |
| Submerged Unit Weight (γ), kN/m$^3$ | 13 |

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

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

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

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

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

### Soldier Pile and Lagging System

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

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

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

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

## 5.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the material's SPMDD.

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

## 5.5 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.

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.

## 5.6 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.

# 6.0 Recommendations

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

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

☐ Sampling and testing of the concrete and fill materials.

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

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed foundation drainage design and requirements.

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

☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.

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

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

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

# 7.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

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

**Paterson Group Inc.**

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

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
April 29, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

**Report Distribution**

☐ Scott Street Developments Inc.
☐ Paterson Group

# APPENDIX 1

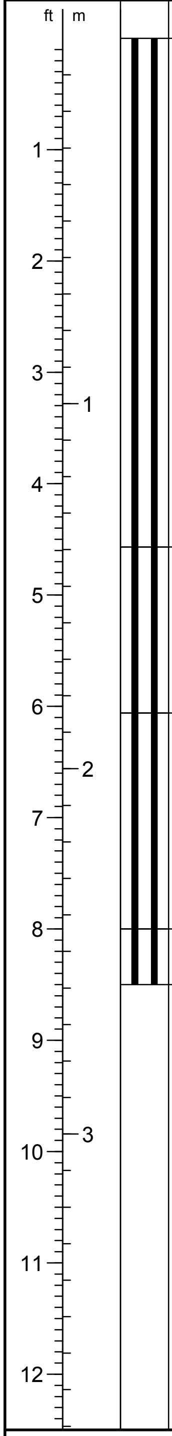
## BOREHOLE LOGS - BY OTHERS

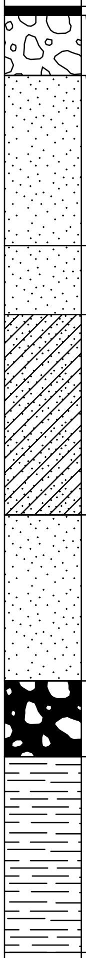
# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

## 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 | BH18-01 |
| | | | | Thin layer of asphalt | |
| | | | | FILL<br>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<br>5 | 0<br>0 | Native<br>Clay, firm, grey, moist | |
| | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-01<br>2.59 mBGS | |

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

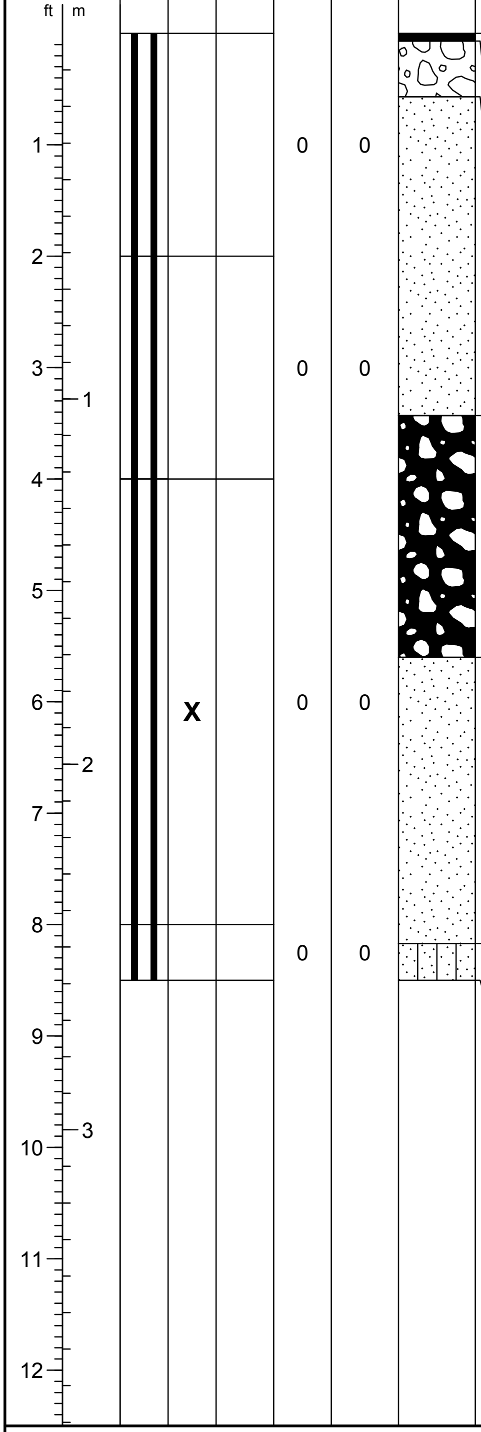
Geofirma Engineering Ltd

# 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 |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | GROUND SURFACE | BH18-02 |
| | | | | | | | Thin layer of ashphalt | |
| | | | | | | | FILL<br>Gravel | |
| 1 | | | | 0 | 0 | | Sand with gravel, dark brown | |
| 3 | | | | 0 | 0 | | | |
| | | | | | | | Boulder | |
| 6 | | X | | 0 | 0 | | Sand, loose, light brown | |
| | | | | | | | Wet | |
| | | | | 0 | 0 | | Native<br>Sandy silt, grey | |
| | | | | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-02<br>2.59 mBGS | |

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

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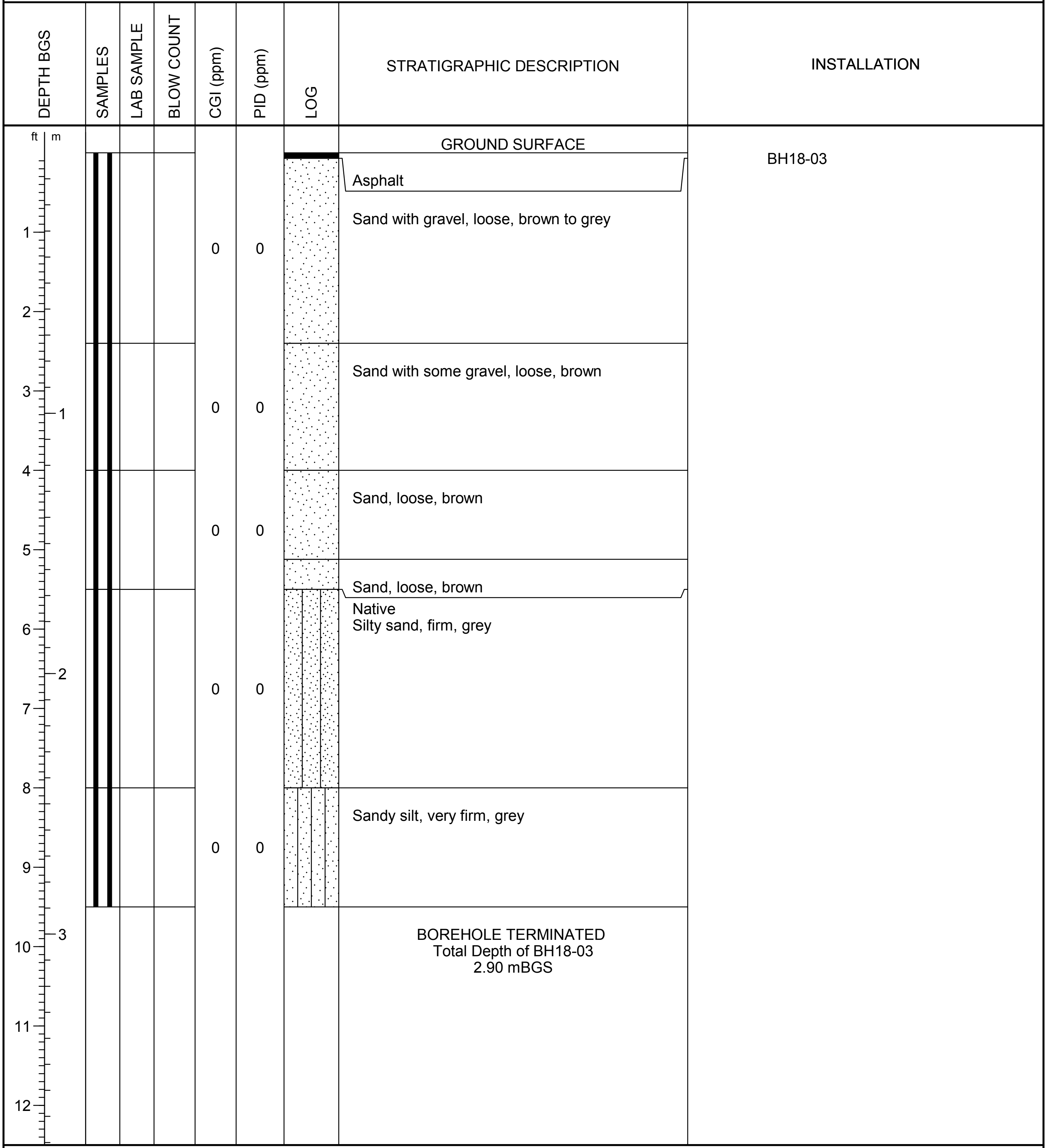
# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

## 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



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: BH18-04

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

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

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# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

## Borehole Number: BH18-05

| | |
|---|---|
| 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: 441017.48E, 5027149.07N (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 |

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

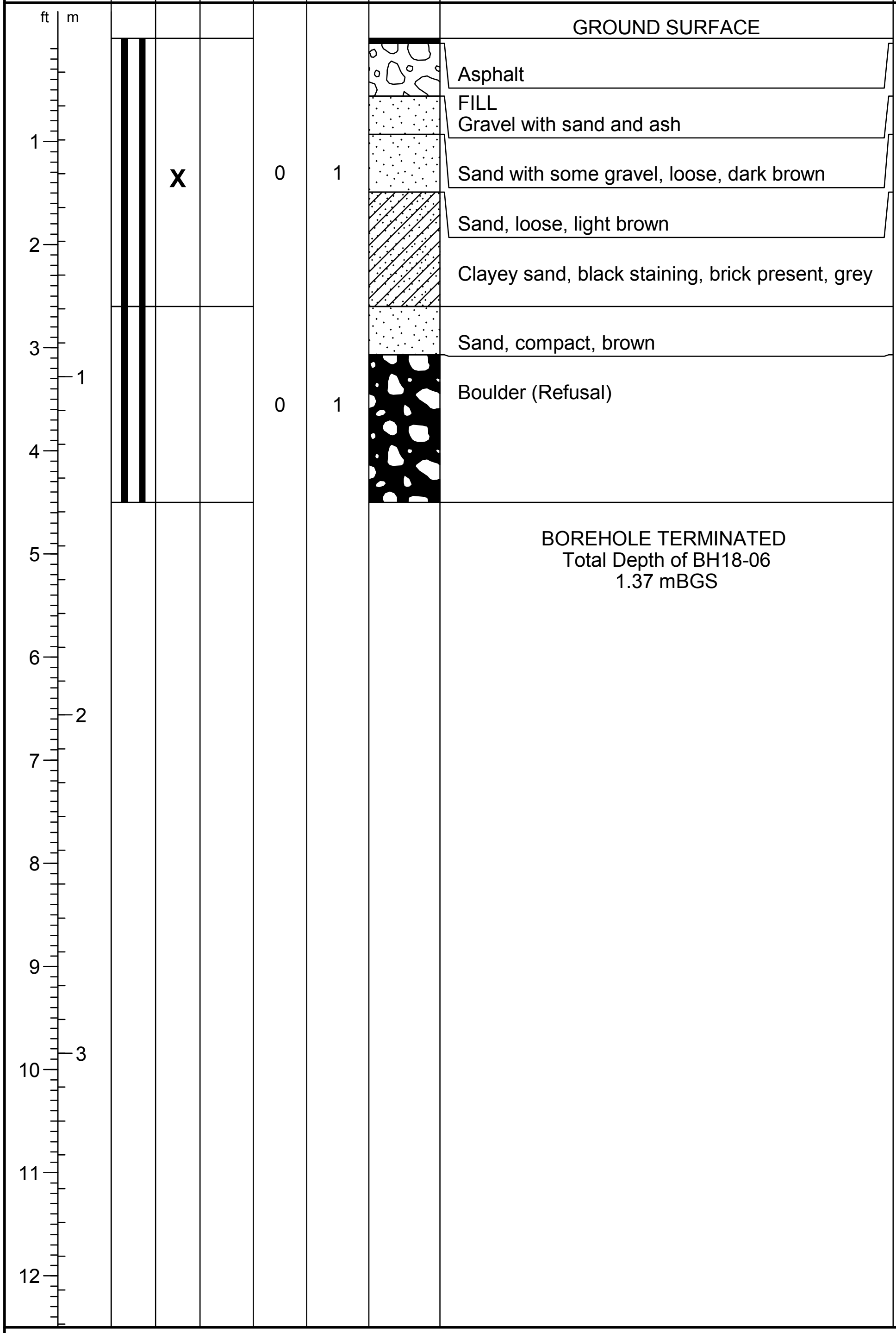
Prepared By: TEW
Reviewed By: TKG
Doc: 18-214-1_BH LOGS.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

## Borehole Number: BH18-06

| | |
|---|---|
| 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: 441010.47E, 5027145.43N (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 |



| DEPTH BGS | SAMPLES | LAB SAMPLE | BLOW COUNT | CGI (ppm) | PID (ppm) | LOG | STRATIGRAPHIC DESCRIPTION | INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | GROUND SURFACE | BH18-06 |
| | | | | | | | Asphalt | |
| | | | | | | | FILL<br>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

| DEPTH BGS<br>ft / m | SAMPLES | LAB SAMPLE | BLOW COUNT | CGI (ppm) | PID (ppm) | LOG | STRATIGRAPHIC DESCRIPTION | INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | GROUND SURFACE | BH18-07 |
| | | | | | | | Asphalt | |
| 1 | | | | 0 | 0 | | FILL<br>Gravel with sand and ash | |
| | | | | | | | Clayey sand, brown grey | |
| 2 | | | | | | | Clayey sand with black staining, brown grey | |
| 3 / 1 | | | | 0 | 1 | | Sand with boulders, brown | |
| 4, 5 | | | | 0 | 1 | | Sand, fine grained, loose, light brown grey | |
| 6, 7 / 2 | | | | 0 | 1 | | Native<br>Sandy silt, damp, grey | |
| | | | | | | | Damp | |
| 8, 9, 10 / 3, 11, 12 | | | | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-07<br>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 |

| DEPTH BGS | | SAMPLES | LAB SAMPLE | BLOW COUNT | CGI (ppm) | PID (ppm) | LOG | STRATIGRAPHIC DESCRIPTION | INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|
| ft | m | | | | | | | GROUND SURFACE | BH18-08 |
| | | | | | | | | Asphalt | |
| | | | | | | | | FILL<br>Gravel | |
| 1 | | | | | 5 | 1 | | Sand with some gravel, loose, brown | |
| 2<br>3 | 1 | | | | 5 | 1 | | Clayey sand, with black staining, brown | |
| | | | | | | | | Boulder | |
| 4<br>5<br>6 | | | | | 0 | 1 | | Sand lenses, light brown and grey | |
| 7 | 2 | | | | 0 | 1 | | Silty sand, fine grained, grey | |
| 8<br>9<br>10 | 3 | | X | | 40 | 1 | | Sandly silt, firm, grey<br>Damp | |
| 11<br>12 | | | | | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-08<br>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 |
| 1 | | | | | 0 | 1 | | Concrete<br>FILL<br>Gravel<br>Sand, loose, brown | |
| 2<br>3 | 1 | | X | | 15 | 11 | | Clayey sand, black staining, black, brown<br>Clayey sand, brown | |
| 4<br>5<br>6 | 2 | | | | 0 | 0 | | Gravel<br>Sand, loose, brown<br>Sand, loose, grey brown<br>Sand, loose, light brown | |
| 7 | | | | | 0 | 0 | | Native<br>Sandy silt, firm, grey | |
| 8 | | | X | | 25 | 2 | | Wet | |
| 9<br>10 | 3 | | | | 20 | 0 | | | |
| 11<br>12 | | | | | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-09<br>3.05 mBGS | |

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

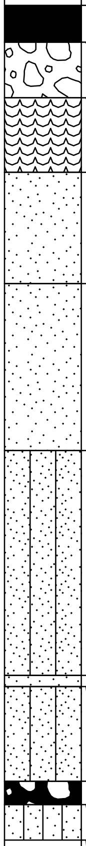
Geofirma Engineering Ltd

# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

## Borehole Number: BH18-10

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



| DEPTH BGS | SAMPLES | LAB SAMPLE | BLOW COUNT | CGI (ppm) | PID (ppm) | LOG | STRATIGRAPHIC DESCRIPTION | INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| ft \| m | | | | | | | GROUND SURFACE | BH18-10 |
| | | | | | | | Concrete | |
| | | | | | | | FILL<br>Gravel | |
| | | | | 0 | 0 | | Organic material, black | |
| | | | | | | | Sand with some gravel, loose | |
| | | | | 0 | 0 | | Sand, loose, grey brown | |
| | | | | 0 | 0 | | Sand, compact, grey | |
| | | | | | | | Sand lenses, light brown | |
| | | | | 0 | 0 | | Silty sand, brown grey | |
| | | | | | | | Boulder | |
| | | | | | | | Native<br>Sandy silt, grey | |
| | | | | | | | BOREHOLE TERMINATED<br>Total Depth of BH18-10<br>2.29 mBGS | |

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

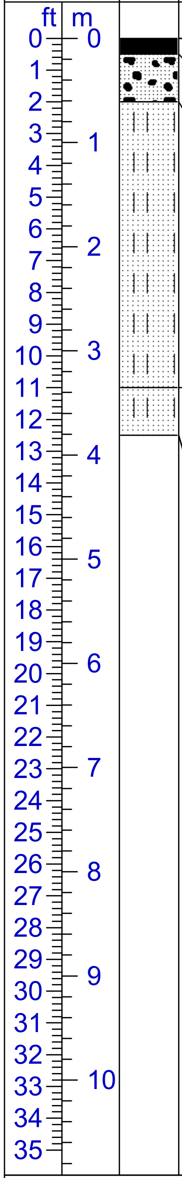
***Project:*** Phase II Environmental Site Assessment

***Client:*** 347313 Canada Inc.

***Location:*** 2050 Scott Street, Ottawa, Ontario

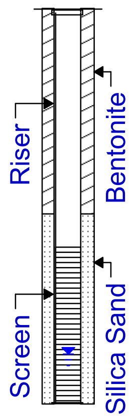
***Drill Date:*** November 22, 2018

## SUBSURFACE PROFILE



| Description | Measured Depth (m) |
|---|---|
| Ground Surface | 0.00 |
| ***Asphalt*** | |
| ***Sandy Gravel***<br>Brown, moist, no PHC odour or staining. | 0.61 |
| ***Silty Sand***<br>Brown, gravel throughout, no PHC odour or staining. | 3.35 |
| ***Silty Sand***<br>Brown, gravel throughout, turning wet, no PHC odour or staining.<br>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.

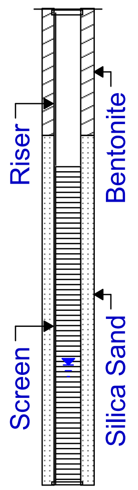
**Location:** 2050 Scott Street, Ottawa, Ontario

**Drill Date:** November 22, 2018

## SUBSURFACE PROFILE

| Description | Measured Depth (m) |
|---|---|
| Ground Surface | 0.00 |
| ***Asphalt*** | |
| ***Sandy Gravel***<br>Brown, moist, no PHC odour or staining. | 0.61 |
| ***Silty Sand***<br>Brown, gravel throughout, moist, no PHC odour or staining. | 2.74 |
| ***Silty Sand***<br>Brown, gravel throughout, wet, PHC odour, no staining.<br>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

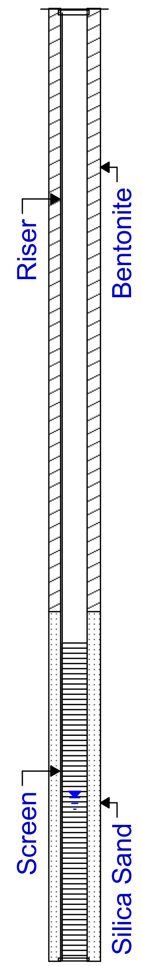
**Drill Date:** November 22, 2018

## SUBSURFACE PROFILE

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

Depth scale: 0–35 ft; 0–10 m.

Monitoring Well Details



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 |

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-4

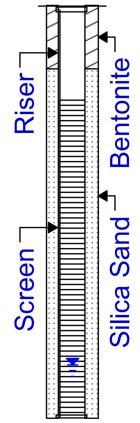
**Project #:** 232288.001
**Logged By:** RL
**Project:** Phase II Environmental Site Assessment
**Client:** 347313 Canada Inc.
**Location:** 2050 Scott Street, Ottawa, Ontario
**Drill Date:** December 3, 2018

## SUBSURFACE PROFILE

| Description | Measured Depth (m) |
|---|---|
| Ground Surface | 0.00 |
| ***Concrete*** | |
| ***Sandy Gravel***<br>Brown, moist, no PHC odour or staining. | 0.61 |
| ***Sand***<br>Brown, moist, no PHC odour or staining.<br>Turning wet and PHC odour @ 13'.<br>Refusal on Bedrock. | 3.96 |
| End of Borehole | |

Depth: ft 0–35; m 0–10

Monitoring Well Details



Water level measured at 3.45 mbgs on December 11, 2018.

## SAMPLE

| Recovery (%) | Sample ID | Soil Vapour Concentration (ppm) PID | Laboratory Analysis |
|---|---|---|---|
| 30 | SS1 | 0 | |
| 30 | SS2 | 0 | |
| 40 | SS3 | 0 | |
| 40 | SS4 | 48 | |
| 40 | SS5 | 268 | PHCs, VOCs, PAHs |

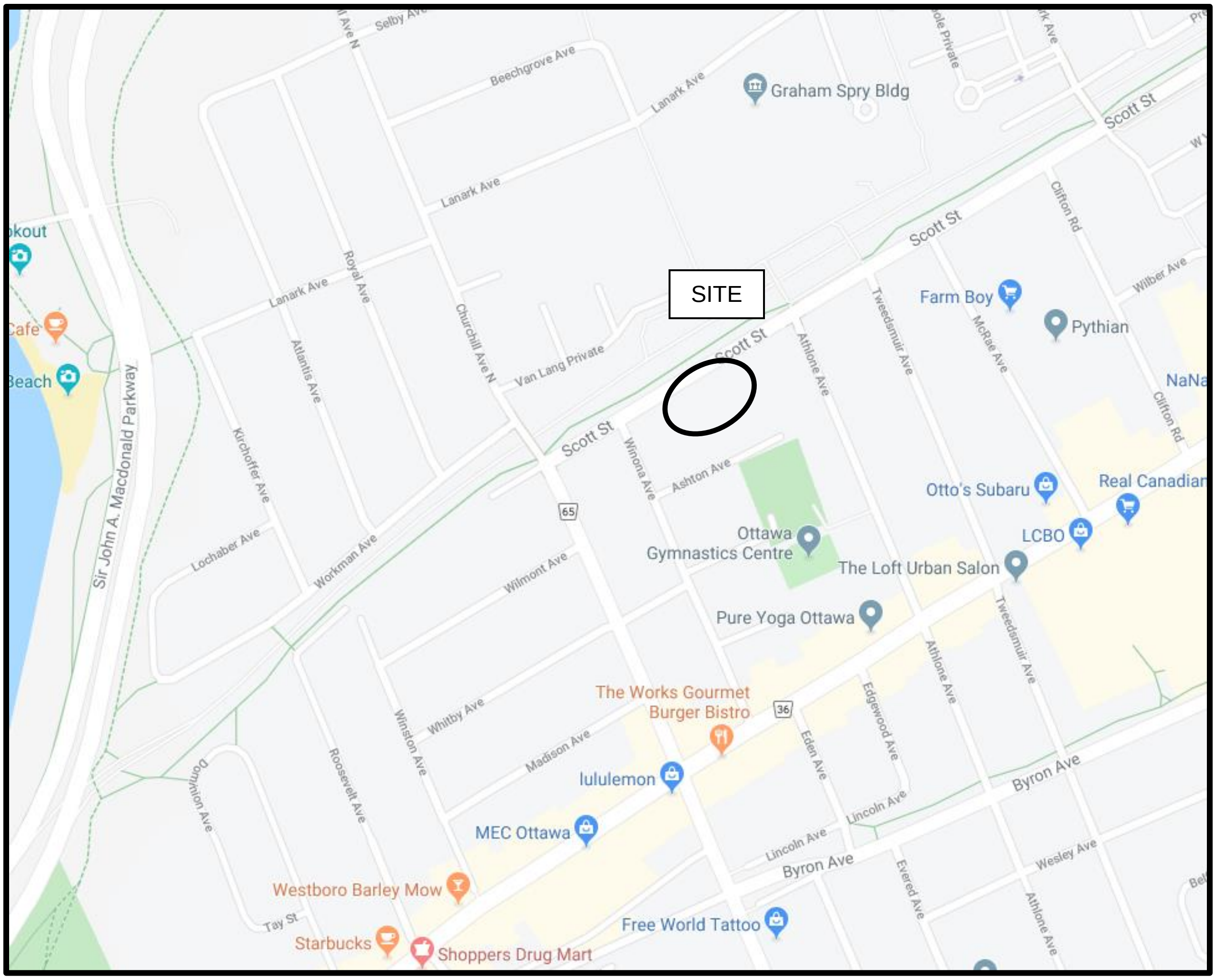
Soil vapour concentrations measured using a photoionization detector (PID).

**Contractor:** Strata Drilling Group
**Grade Elevation:** NA
**Drilling Method:** Geo-probe
**Top of Casing Elevation:** NA
**Well Casing Size:** 5.08 cm
**Sheet:** 1 of 1

# APPENDIX 2

FIGURE 1 - KEY PLAN

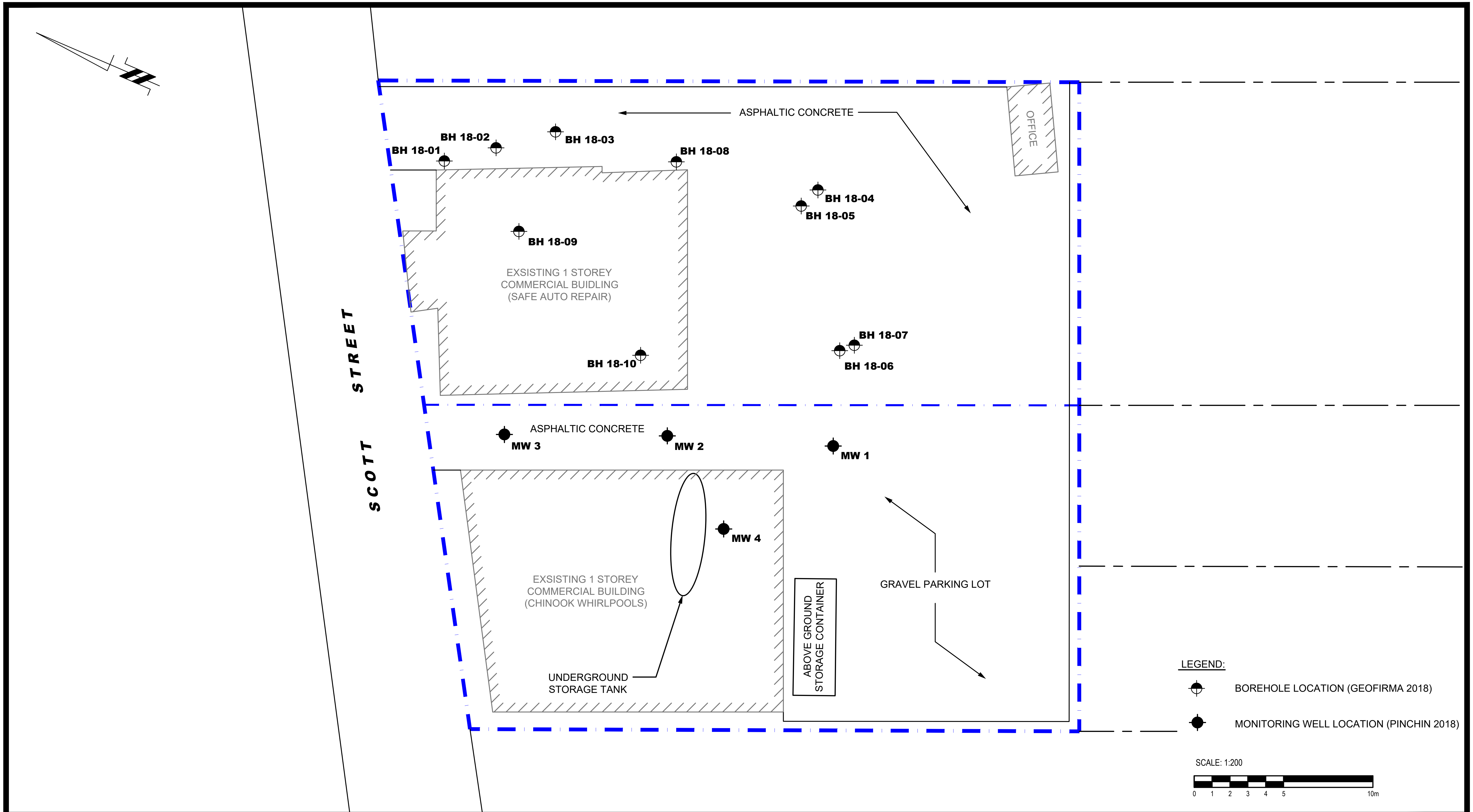
PG5222-1 - TEST HOLE LOCATION PLAN



# FIGURE 1

## KEY PLAN

**patersongroup**



**patersongroup**
**consulting engineers**

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

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|

Scott Street Development Inc.
**GEOTECHNICAL DESKTOP REVIEW**
**2046 / 2050 SCOTT STREET**

OTTAWA, ONTARIO

Title:
# TEST HOLE LOCATION PLAN

Scale: 1:200
Date: 01/2020
Drawn by: NFRV
Report No.: PG5222-1
Checked by: JV
Dwg. No.: **PG5222-1**
Approved by: DJG
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