**patersongroup**

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

# Geotechnical Investigation

Proposed Multi-Storey Building
1050 Bank Street
Ottawa, Ontario

Prepared For

Beaumont & Fine Investments

August 20, 2018

Report PG4506-1

**Paterson Group Inc.**
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

# Table of Contents

# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Beaumont & Fine Investments to undertake a geotechnical investigation for a proposed multi-storey building to be located at 1050 Bank Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- Determine the subsurface soil and groundwater conditions by means of boreholes.

- Provide geotechnical recommendations for the design of the proposed building including construction considerations which may affect the design.

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

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

# 2.0 Proposed Development

Specific details of the development were not available at the time of issuance of this report. However, it is understood that the proposed development is to consist of a 4 storey mixed-use (residential and commercial) building. It is further anticipated that the proposed building will have one level of underground parking which will occupy the entire building footprint. The proposed development is understood to include associated at-grade paved access lanes and landscaped areas.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the current investigation was carried out from May 1 to 4, 2018. At that time, 3 boreholes (BH 1 to BH 3) were drilled to a maximum depth of 14.6 m below the existing ground surface. The test hole locations were selected in a manner to provide general coverage of the subject site.

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

The borehole locations are presented on Drawing PG4506-1 - Test Hole Location Plan appended to this report. It should be noted that the current geotechnical investigation and recommendations apply only to 1050 Bank Street.

### Sampling and In Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to the laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

**Groundwater**

Monitoring wells, consisting of 50 mm diameter rigid PVC standpipes, were installed in each of the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current field program.

**Sample Storage**

All samples will be stored in the laboratory for a period of one month after issuance of this report unless we are otherwise directed.

## 3.2 Field Survey

The test hole locations for the current geotechnical investigation were selected, determined in the field and surveyed by Paterson personnel. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located across Bank Street from the subject property. An arbitrary elevation of 100.00 m was assigned to the TBM.

The ground surface elevation and location of each borehole, as well as the location of the TBM is presented on the attached Drawing PG4506-1 - Test Hole Location Plan.

## 3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

## 3.4 Analytical Testing

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

# 4.0 Observations

## 4.1 Surface Conditions

The subject site is currently occupied by an existing 1 storey commercial building in the west portion of the site. The remainder of the site consists of an asphalt surfaced parking lot with landscaped margins. Site drainage is provided by several centrally located catch basins. The site is bordered to the north by Aylmer Avenue, to the east by Bank Street, to the south by existing commercial properties and to the west by existing residential properties. The ground surface across the site is relatively level.

## 4.2 Subsurface Profile

The subsurface profile at the borehole locations consists of a pavement structure underlain by a fill layer to approximately 1.4 to 2.4 m depth. The fill was generally observed to consist of a loose, brown sand with some silt, gravel, and asphalt.

The fill was underlain by a compact to very dense sand deposit. Within the sand deposit, an interbedded layer of compact to very dense silty sand to sandy silt was encountered at depths of 8.4 and 7.8 m in boreholes BH 1 and BH 2, respectively. The silty sand to sandy silt layer had an approximate thickness of 2.5 to 3 m. The sand deposit extended to the bottom of the boreholes at depths of 14.6 m, with the exception of borehole BH 3 which encountered practical refusal to augering at an approximate 13.7 m depth. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam formation.

## 4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed in the boreholes upon completion of the sampling program. The groundwater level readings at each borehole location are presented in Table 1.

**Table 1 - Summary of Groundwater Level Readings**

| Borehole Number | Ground Elevation (m) | Groundwater Levels | | Recording Date |
|---|---|---|---|---|
| | | Depth (m) | Elevation (m) | |
| BH 1 | 99.34 | 11.39 | 87.95 | May 15, 2018 |
| BH 2 | 99.34 | 11.75 | 87.59 | May 15, 2018 |
| BH 3 | 99.31 | 11.73 | 87.58 | May 15, 2018 |

**Note:** Ground surface elevations at borehole locations were referenced to a TBM, consisting of the top spindle of a fire hydrant located across Bank Street from the subject property, which was assigned an elevation of 100.00 m.

Based on these observations, it is estimated that the long-term groundwater table an be expected at approximately 11 to 12 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is anticipated that the proposed building will be founded on conventional spread footings placed on a bearing surface consisting of undisturbed, compact to dense sand.

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

## 5.2 Site Grading and Preparation

### Stripping Depth

Asphalt, topsoil and deleterious fill, such as those containing organic materials, should be stripped from within the building footprint, paved areas, pipe bedding and other settlement sensitive structures.

It is anticipated that the existing fill layer, free of deleterious material and significant amounts of organics, can be left in place below the proposed paved access lanes. However, it is recommended that the existing fill layer be proof-rolled using heavy vibratory equipment and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with approved granular fill, such as Granular A or B Type II material.

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

### 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).

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

## 5.3 Foundation Design

### Conventional Shallow Footings

Footings placed on an undisturbed, compact to dense sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS. Proof rolling should be carried out for any soils at footing level which are noted to be in a loose state. Any soft or loose areas noted during proof rolling should be removed and replaced with an engineered fill pad. Footings placed over an engineered fill pad can be designed using the abovenoted bearing resistance values.

Footings designed using the above-noted bearing resistance values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

### Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a compact sand and engineered fill bearing medium above the water table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil or engineered fill of the same or higher capacity as the soil.

## 5.4 Design for Earthquakes

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

Due to the relative density of the sand and the depth of the long term groundwater level, soils underlying the site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

## 5.5 Basement Slab Construction

With the removal of all topsoil and/or fill, containing significant amounts of organic or deleterious materials within the footprint of the proposed buildings, the native soil will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compacted to at least 98% of the material's SPMDD.

It is also recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone below the basement floor slab.

It may be possible that the garage level will consist of a flexible asphaltic pavement structure which will follow the recommendations stipulated in Subsection 5.7 of this report.

## 5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m$^3$. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m$^3$, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

### Lateral Earth Pressures

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

$K_o$ = at-rest earth pressure coefficient of the applicable retained soil, 0.5
$\gamma$ = 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 wall height should be incorporated into the diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be calculated with the seismic loading case.

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

### Seismic Earth Pressures

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

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

$a_c$ = $(1.45 - a_{max}/g)a_{max}$
$\gamma$ = 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. The vertical seismic coefficient is assumed to be zero.

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

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

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

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

## 5.7 Pavement Structure

### Minimum Pavement Structure

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

**Table 2 - Recommended Pavement Structure - Car Only Parking Areas**

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

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

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

### Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

### Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 composite drainage blanket, such as Miradrain G100N or Delta Drain 6000.

### Concrete Sidewalks Adjacent to Building(s)

To avoid differential settlements within the proposed sidewalks adjacent to the proposed buildings, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks to consist of free draining, non-frost susceptible material such as Granular A or Granular B Type II, instead of site excavated material which, in most cases, is considered frost susceptible. The granular material should be placed in maximum 300 mm thick loose lifts and compacted to 95% of the material's SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the building's perimeter drainage pipe.

## 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. Soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation should be provided.

## 6.3 Excavation 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.

**Temporary Side Slopes**

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**

The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. It is expected that a shoring contractor will be responsible for the design and installation of the shoring system based on information provided by the contractor’s engineers, including the geotechnical consultant.

For preliminary design purposes, the temporary system may consist of a 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. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 4.

| Table 4 - Soil Parameters | |
|---|---|
| **Parameters** | **Values** |
| Active Earth Pressure Coefficient ($K_a$) | 0.33 |
| Passive Earth Pressure Coefficient ($K_p$) | 3 |
| At-Rest Earth Pressure Coefficient ($K_o$) | 0.5 |
| Unit Weight (γ), $kN/m^3$ | 20 |
| Submerged Unit Weight(γ), $kN/m^3$ | 13 |

### 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.

### Damage to Adjacent Structures

There is a possibility of disturbance to adjacent subsoil which may cause damage to adjacent structures due to the installation of the temporary shoring system. The sole responsibility of any damage caused by the installation of the temporary shoring system will be with the shoring contractor. The geotechnical engineering consultant will not be liable for any damages arising from the temporary shoring installation and will be indemnified by the owner and shoring contractor.

## 6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential for 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 low through the sides of the excavation and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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 the Environment and Climate Change (MOECC) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

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

## 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 the 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.

## 6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive corrosive environment.

# 7.0 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.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

# 8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation of this nature is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the 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 Beaumont & Fine Investments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

**Paterson Group Inc.**

LICENSED PROFESSIONAL ENGINEER
Aug. 20-2018
N. F. S. CHRISTIE
100200855
PROVINCE OF ONTARIO

Nathan F. S. Christie, P.Eng.

Scott S. Dennis, P.Eng.

**Report Distribution**

- ❑ Beaumont & Fine Investments (3 copies)
- ❑ Paterson Group (1 copy)

# APPENDIX 1

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TESTING RESULTS**

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1050 Bank Street
Ottawa, Ontario

**DATUM** TBM - Top spindle of fire hydrant located in front of subject site. Assumed elevation = 100.00m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** May 1, 2018

**FILE NO.** PG4506

**HOLE NO.** BH 1

| SOIL DESCRIPTION | Strata depth (m) | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 99.34 |
| 50mm Asphaltic concrete over crushed stone with silt and sand | 0.60 | AU | 1 | | | | |
| **FILL:** Brown sand, some silt, gravel and asphalt | | SS | 2 | 33 | 8 | 1 | 98.34 |
| | 2.44 | SS | 3 | 50 | 4 | 2 | 97.34 |
| Compact to dense, brown **SAND,** trace silt | | SS | 4 | 50 | 14 | | |
| | | SS | 5 | 92 | 12 | 3 | 96.34 |
| | | SS | 6 | 92 | 22 | 4 | 95.34 |
| | | SS | 7 | 92 | 54 | 5 | 94.34 |
| | | SS | 8 | 92 | 45 | 6 | 93.34 |
| | | SS | 9 | 83 | 45 | | |
| | | SS | 10 | 75 | 41 | 7 | 92.34 |
| | 8.38 | SS | 11 | 83 | 29 | 8 | 91.34 |
| Dense, brown **SILTY FINE SAND,** trace clay and gravel | 9.14 | SS | 12 | 100 | 40 | 9 | 90.34 |
| Very dense, brown **SANDY SILT,** some clay, gravel and cobbles | | SS | 13 | 89 | 50+ | | |
| | 10.67 | SS | 14 | 75 | 50+ | 10 | 89.34 |
| Very dense, brown **SAND,** some gravel | | SS | 15 | 75 | 61 | 11 | 88.34 |
| | | SS | 16 | 82 | 50+ | | |
| | | SS | 17 | 100 | 50+ | 12 | 87.34 |
| | | | | | | 13 | 86.34 |
| | 14.63 | | | | | 14 | 85.34 |
| End of Borehole | | | | | | | |
| (GWL @ 11.39m-May 15, 2018) | | | | | | | |

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Monitoring Well Construction

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Development - 1050 Bank Street**
**Ottawa, Ontario**

**DATUM** TBM - Top spindle of fire hydrant located in front of subject site. Assumed elevation = 100.00m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** May 2, 2018

**FILE NO.** PG4506

**HOLE NO.** BH 2

| SOIL DESCRIPTION | Layer base depth (m) |
|---|---|
| GROUND SURFACE | |
| 50mm Asphaltic concrete over crushed stone with silt and sand | 0.46 |
| **FILL:** Brown sand, some gravel, asphalt, trace silt | 1.37 |
| Very loose, brown **FINE SAND,** trace silt, gravel, cobbles | 2.13 |
| Compact to dense, brown **SAND**<br>- some gravel and cobbles, trace clay by 6.1m depth | 7.77 |
| Compact to very dense, brown **SILTY SAND,** some gravel, trace clay, cobbles and boulders | 10.67 |
| Dense to very dense, brown **SAND,** some gravel | 14.63 |
| End of Borehole<br>(GWL @ 11.75m-May 15, 2018) | |

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| SS | 2 | 92 | 4 |
| SS | 3 | 67 | 3 |
| SS | 4 | 100 | 10 |
| SS | 5 | 92 | 21 |
| SS | 6 | 75 | 29 |
| SS | 7 | 75 | 27 |
| SS | 8 | 83 | 25 |
| SS | 9 | 75 | 40 |
| SS | 10 | 58 | 38 |
| SS | 11 | 83 | 27 |
| SS | 12 | 67 | 19 |
| SS | 13 | 88 | 50+ |
| SS | 14 | 50 | 50+ |
| SS | 15 | 83 | 36 |
| SS | 16 | 80 | 50+ |
| SS | 17 | 67 | 50+ |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 99.34 |
| 1 | 98.34 |
| 2 | 97.34 |
| 3 | 96.34 |
| 4 | 95.34 |
| 5 | 94.34 |
| 6 | 93.34 |
| 7 | 92.34 |
| 8 | 91.34 |
| 9 | 90.34 |
| 10 | 89.34 |
| 11 | 88.34 |
| 12 | 87.34 |
| 13 | 86.34 |
| 14 | 85.34 |

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

Monitoring Well Construction

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

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1050 Bank Street
Ottawa, Ontario

**DATUM** TBM - Top spindle of fire hydrant located in front of subject site. Assumed elevation = 100.00m.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** May 4, 2018

**FILE NO.** PG4506

**HOLE NO.** BH 3

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 99.31 |
| 50mm Asphaltic concrete over crushed stone with silt and sand | 0.46 | AU | 1 | | | | |
| **FILL:** Brown sandy silt, some gravel and asphalt | 1.98 | SS | 2 | 54 | 9 | 1 | 98.31 |
| | | SS | 3 | 75 | 5 | 2 | 97.31 |
| Compact to very dense, brown **SAND,** trace silt | 9.75 | SS | 4 | 58 | 23 | 3 | 96.31 |
| | | SS | 5 | 75 | 34 | 4 | 95.31 |
| | | SS | 6 | 58 | 47 | 5 | 94.31 |
| | | SS | 7 | 58 | 40 | 6 | 93.31 |
| | | SS | 8 | 58 | 41 | 7 | 92.31 |
| | | SS | 9 | 83 | 54 | 8 | 91.31 |
| | | SS | 10 | 75 | 56 | 9 | 90.31 |
| | | SS | 11 | 58 | 75 | | |
| Very dense, brown **SAND,** some silt, clay and gravel | 10.67 | SS | 12 | 67 | 53 | 10 | 89.31 |
| Dense, brown **SAND** | 13.72 | SS | 13 | 75 | 43 | 11 | 88.31 |
| | | SS | 14 | 83 | 47 | 12 | 87.31 |
| | | | | | | 13 | 86.31 |

End of Borehole

Practical refusal to augering at 13.72m depth

(GWL @ 11.73m-May 15, 2018)

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

Monitoring Well Construction

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

# 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<br>These grain size descriptions are not used below 0.075 mm grain size |
| D10 | - | Grain size at which 10% of the soil is finer (effective grain size) |
| D60 | - | Grain size at which 60% of the soil is finer |
| Cc | - | Concavity coefficient = $(D30)^2 / (D10 \times D60)$ |
| Cu | - | Uniformity coefficient = $D60 / D10$ |

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

### CONSOLIDATION TEST

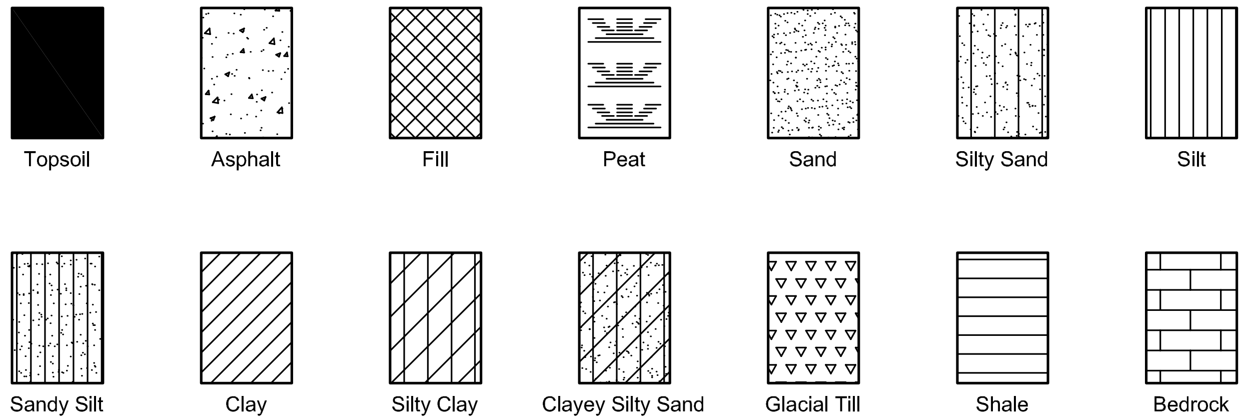
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above $p'_c$) |
| OC Ratio | | Overconsolidaton ratio = $p'_c / p'_o$ |
| Void Ratio | | Initial sample void ratio = volume of voids / volume of solids |
| Wo | - | Initial water content (at start of consolidation test) |

### PERMEABILITY TEST

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

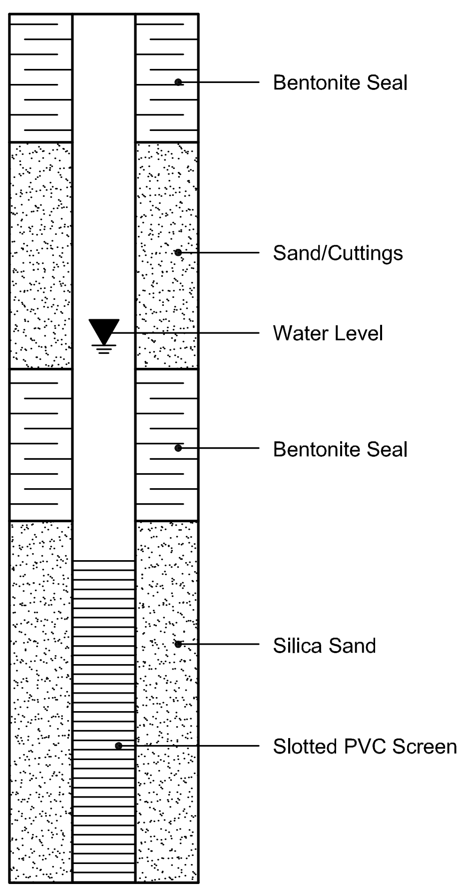
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

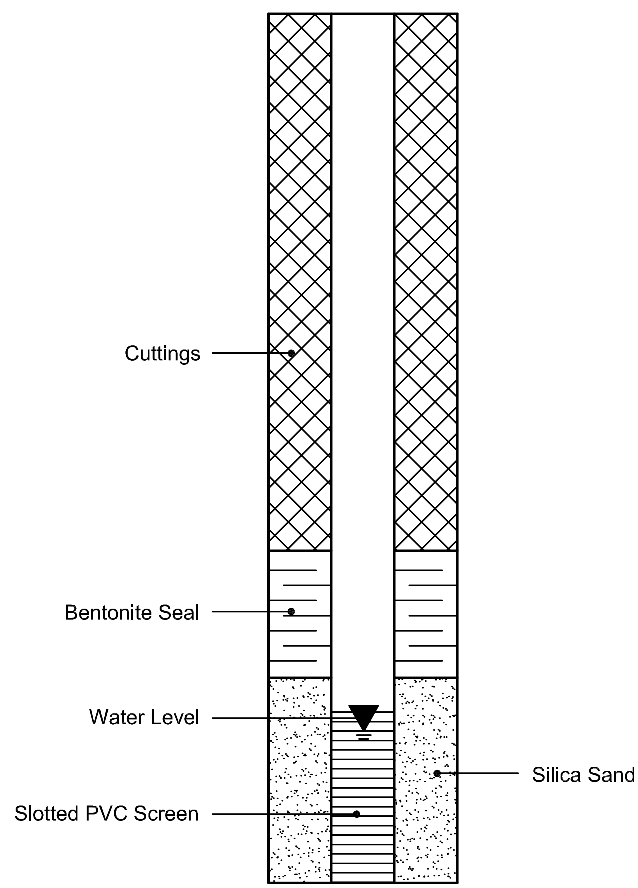



## MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION




PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

**Order #: 1826552**

Certificate of Analysis
**Client: Paterson Group Consulting Engineers**
**Client PO: 24194**

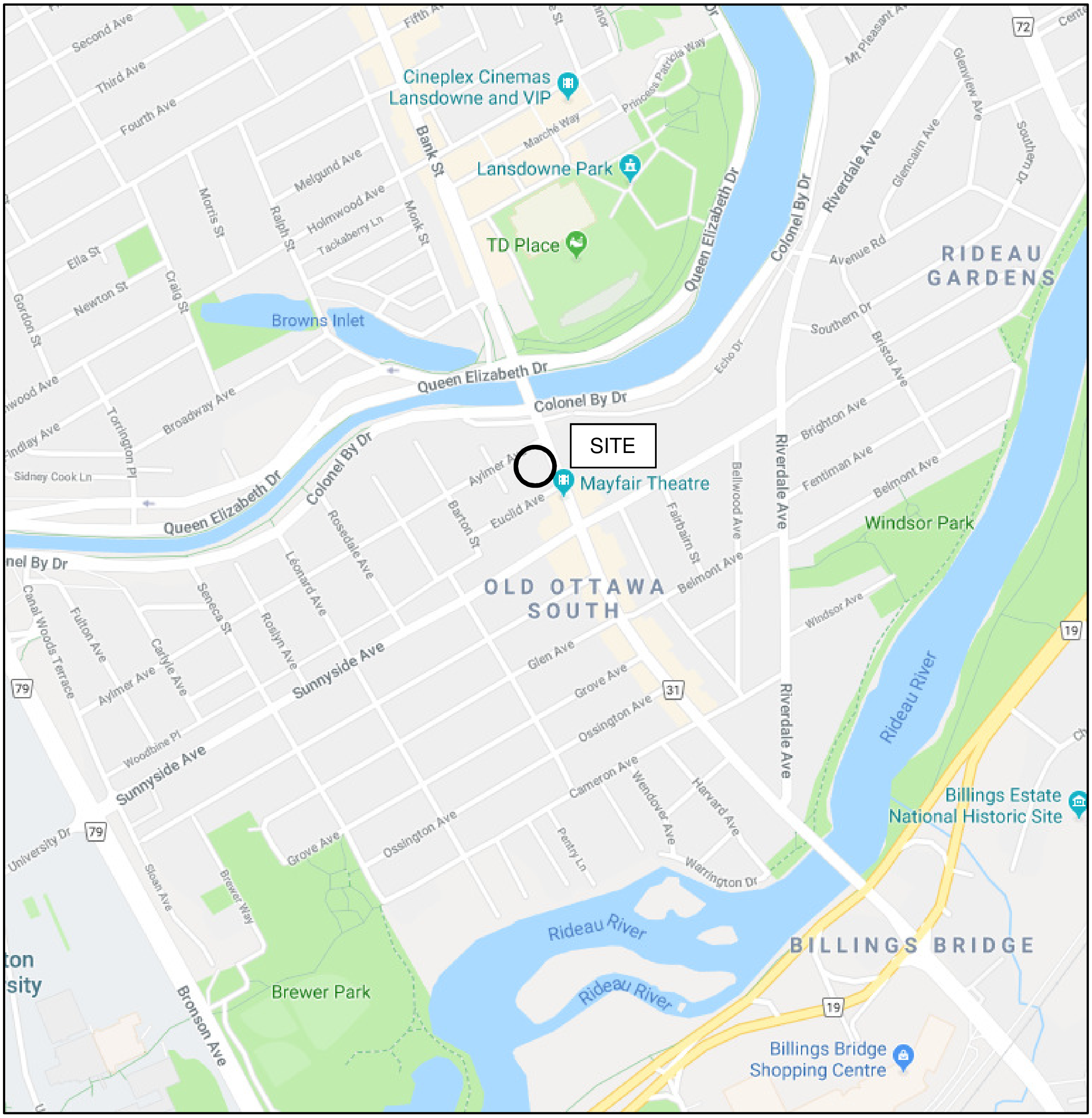
Report Date: 04-Jul-2018
Order Date: 28-Jun-2018
**Project Description: PG4506**

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH1-SS4 | - | - | - |
| | **Sample Date:** | 05/01/2018 09:00 | - | - | - |
| | **Sample ID:** | 1826552-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 89.8 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.70 | - | - | - |
| Resistivity | 0.10 Ohm.m | 22.1 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 202 [1] | - | - | - |
| Sulphate | 5 ug/g dry | 53 [1] | - | - | - |

# APPENDIX 2

**FIGURE 1 - KEY PLAN**

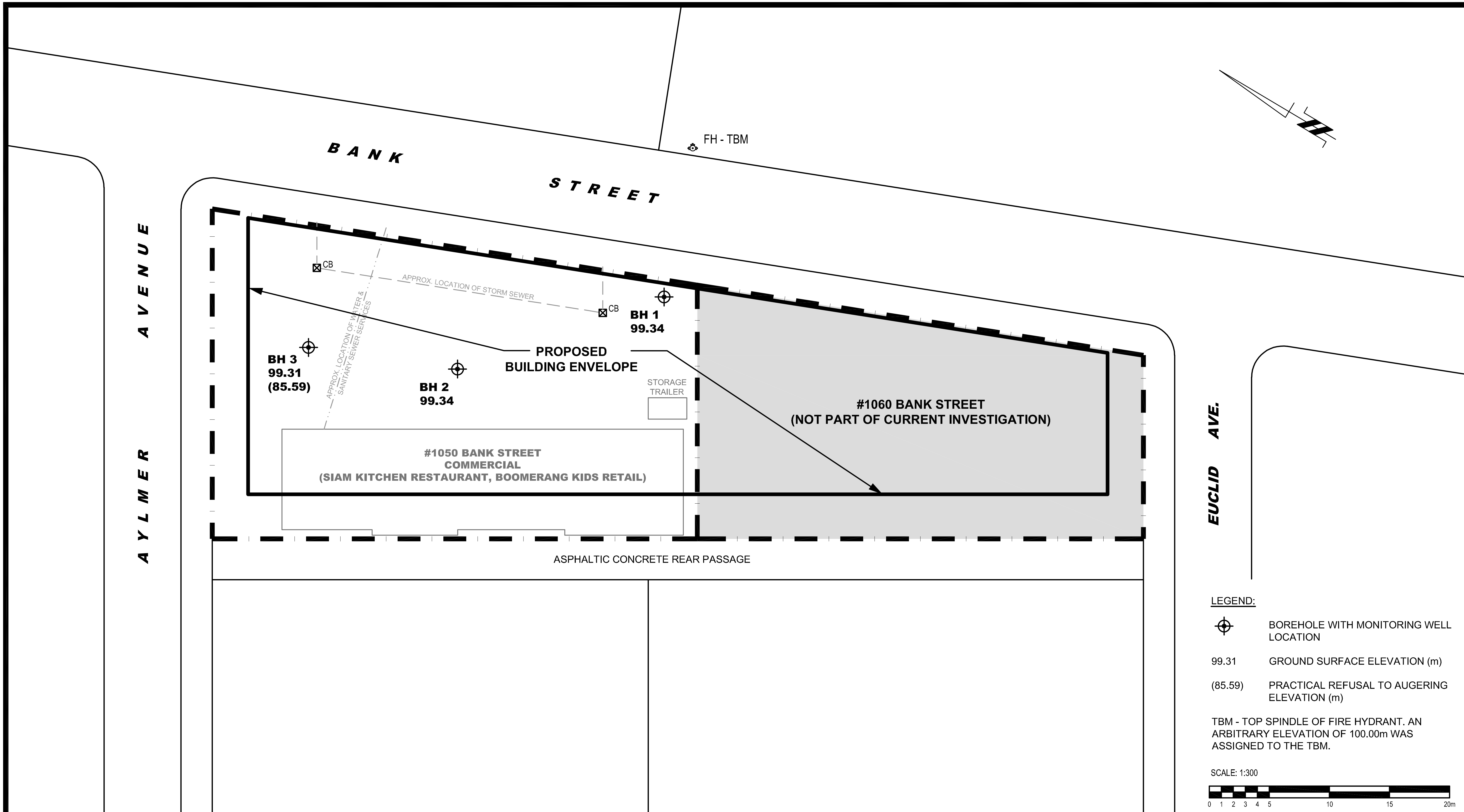
**DRAWING PG4506-1 - TEST HOLE LOCATION PLAN**




# 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 |
|---|---|---|---|
| 0 | | | |

BEAUMONT AND FINE INVESTMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT - 1050 BANK STREET

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Scale: 1:300 | Date: 06/2018 |
| Drawn by: MPG | Report No.: PG4506-1 |
| Checked by: NC | Dwg. No.: **PG4506-1** |
| Approved by: DJG | Revision No.: 0 |

p:\autocad drawings\geotechnical\pg45xx\pg4506-1 thlp.dwg