Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

patersongroup

Geotechnical Investigation Report

Proposed Multi-Storey Building
1166 Bank Street
Ottawa, Ontario

Prepared For
Ambassador Realty Inc.

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

July 25, 2023

Report: PG6191-1

Revision 1

Table of Contents

1.0 Introduction 1
2.0 Proposed Development 1
3.0 Method of Investigation 2
3.1 Field Investigation 2
3.2 Field Survey 3
3.3 Laboratory Testing 3
3.4 Analytical Testing 3
4.0 Observations 4
4.1 Surface Conditions 4
4.2 Subsurface Profile 4
4.3 Groundwater 5
5.0 Discussion 6
5.1 Geotechnical Assessment 6
5.2 Site Grading and Preparation 6
5.3 Foundation Design 7
5.4 Design for Earthquakes 8
5.5 Basement Slab 8
5.6 Basement Wall 8
5.7 Pavement Structures 10
6.0 Design and Construction Precautions 12
6.1 Foundation Drainage and Backfill 12
6.2 Protection of Footings Against Frost Action 12
6.3 Excavation Side Slopes 13
6.4 Pipe Bedding and Backfill 15
6.5 Groundwater Control 15
6.6 Winter Construction 16
6.7 Corrosion Potential and Sulphate 17
7.0 Recommendations 18
8.0 Statement of Limitations 19

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Soil Profile and Test Data Sheets by Others Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG6191-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ambassador Realty Inc, to undertake a geotechnical investigation for the proposed multi-storey building to be located at 1166 Bank Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the geotechnical investigation were to:

❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.

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

The following report has been prepared specifically and solely for the aforementioned project which is described herein. 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.

2.0 Proposed Development

It is understood that the proposed development will consist of a multi-storey building with 1 underground parking level, which will occupy most of the site footprint. At finished grades, the proposed building will generally be surrounded by walkways and landscaped areas. It is also expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on May 3, 2022. A total of 3 boreholes (BH 1-22 to BH 3-22) were drilled to a maximum depth of 7.6 m below the existing ground surface The borehole locations were selected in a manner to provide general coverage along the areas of environmental concerns at 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.

A previous geotechnical investigation was also conducted at the site, by others, which consisted of 4 boreholes (MW 21-01 to MW 21-04) advanced to a maximum depth of 7.6 m.

The borehole locations are presented on Drawing PG6191-1-Test Hole Location Plan appended to this report.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, 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. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 1-22. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

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

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevation at each borehole location were surveyed by Paterson with respect to a geodetic datum.

The location of the boreholes, and the ground surface elevation at each borehole location, are presented on Drawing PG6191-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one (1) 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 analyzed to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are discussed in Section 6.7 and are provided in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a single-storey commercial building, which is located on the western half of the site. The remainder of the site consists of an asphalt-surfaced parking lot with landscaped margins. It is understood that the site has been previously used as a retail fuel outlet before 1976.

The site is bordered to the south by Grove Avenue, to the east by Bank Street, to the north by a commercial property, and to the west by a residential property. The ground surface across the site is relatively level at approximate geodetic elevation 62 to 63 m.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the borehole locations generally consists of paved asphaltic concrete to a depth of approximately 50 mm, which is underlain by fill and a silty sand/sandy silt deposit. The fill generally extends to approximate depths of 2.4 to 3 m below the existing ground surface, and consists of loose to compact, brown silty sand with some clay and gravel.

A compact to very dense silty sand to sandy silt deposit, with occasional cobbles and boulders, was encountered underlying the fill, and extending to the bottom depths of the boreholes. An approximate 0.7 to 0.8 m thick layer of hard silty clay was also observed within silty sand/sandy silt deposit at boreholes BH 1-22 and BH 3-22.

Practical refusal to the DCPT was encountered in borehole BH 1-22 at an approximate depth of 9.2 m below the existing ground surface.

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

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and shale of the Verulam formation, with an overburden thickness ranging from approximately 5 to 10 m.

4.3 Groundwater

Groundwater levels were measured on May 11, 2022, within the installed monitoring wells. The measured groundwater levels are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-22	62.80	5.49	57.31	May 11, 2022
BH 2-22	62.88	5.48	57.40	May 11, 2022
BH 3-22	62.33	4.93	57.40	May 11, 2022

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate depths of 5 m to 6 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed building. It is recommended that foundation support for the proposed building consist of conventional spread footings bearing on the undisturbed, compact to dense silty sand/sandy silt or undisturbed, hard silty clay.

As part of an environmental site remediation, should contaminated soil need to be removed below the founding level of the proposed building, grades should be re-instated using compacted engineered fill from the undisturbed, compact to dense silty sand/sandy silt or undisturbed, hard silty clay up to the underside of footing or basement slab elevation. Refer to the Phase II – Environmental Site Assessment (Paterson Group Report PE5590-2) for additional information.

Due to the presence of the silty clay layer, the site will be subjected to grade raise restrictions. The permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 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.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

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

Bearing Resistance Values

Footings placed on undisturbed, compact to dense silty sand/sandy silt or undisturbed, hard silty clay, or on compacted engineered fill which is placed directly over these materials, can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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, prior to placement of concrete for footings.

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

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

Permissible Grade Raise Restrictions

Due to the presence of the silty clay deposit at the site, a permissible grade raise restriction of **1 m** above the existing ground surface is recommended for the subject site.

If higher than permissible grade raises are required, preloading, with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlement.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. Soils underlying the subject 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

With the removal of all topsoil and/or fill, containing significant amounts of organic or deleterious materials, within the footprint of the proposed building, the native soil subgrade is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Section 5.7 will be applicable.

However, for 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.

In consideration of the groundwater conditions encountered at the time of the geotechnical investigation, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone layer under the lowest level floor slab.

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.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be incorporated 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 (ΔP_{AE}).

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

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

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

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

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

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

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

5.7 Pavement Structures

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structure presented in Table 3 should be used for access roads and heavy loading parking areas, should they be required.

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 3 - Recommended Pavement Structure – Access Lanes and Heavy Loading 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 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
Subgrade – OPSS Granular B Type I or II material placed over in situ soil or engineered 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 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is anticipated that majority of the building foundation walls will be placed in close proximity to the site boundaries, with the exception of the western foundation wall. Where the foundation walls are located in proximity to the site boundaries, it is expected that the foundation walls will be blind-poured against a composite drainage board which is fastened directly against the temporary shoring system.

It is recommended that the composite drainage board, such as Delta Drain 6000 or an approved equivalent, extend from 300 mm below the exterior finished grade to the founding elevation. It is further recommended that 150 mm diameter sleeves at 3 m centers be cast in the foundation wall at the footing interface to allow for the infiltration of water to flow to an interior perimeter sub-floor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Where a 2-sided foundation wall pour is feasible, it is recommended that the composite drainage board be fastened to the exterior of the foundation wall, and used in conjunction with a perimeter foundation drain. The perimeter foundation drain should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level at the exterior of the foundation wall. The pipe should have a positive outlet, such as a gravity connection to a catch basin.

Underslab Drainage

Underslab drainage is recommended to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 to 10 m centres underlying the lowest level floor slab. The spacing of the underslab drainage pipes should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

Other exterior unheated footings are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent combination of soil cover and foundation insulation.

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. Based on the proximity of the proposed building to the site boundaries, it is anticipated that a temporary shoring system will be required for the majority of the excavation.

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

Temporary shoring may be required to support the overburden soil where insufficient room is available for open cut methods. The shoring requirements, designed by a structural engineer specializing in those works, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and approval of these temporary systems will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid damage to adjacent structures, and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

For design purposes, the temporary shoring system may generally consist of a soldier pile and lagging system, however, consideration should be given to using a secant pile wall where the proposed excavation will be located in proximity to adjacent structures (such as the northern site boundary), in order to minimize impacts to such structures.

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. 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 by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is used.

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

Table 4 - Soil Parameters

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

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

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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 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.

Potential Impacts to Adjacent Structures

Based on the observed groundwater level, the proposed building construction is not anticipated to extend below the groundwater level. Therefore, it should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures in the vicinity of the proposed building.

6.6 Winter Construction

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

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

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. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and 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 a 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:

- Review of the temporary shoring system design, once available.
- 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.
- Observation of the installation of all foundation drainage and underslab drainage structures.
- 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 Ambassador Realty Inc. 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.

Kevin A. Pickard, P.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Ambassador Realty Inc.
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1166 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 3, 2022

FILE NO. PG6191

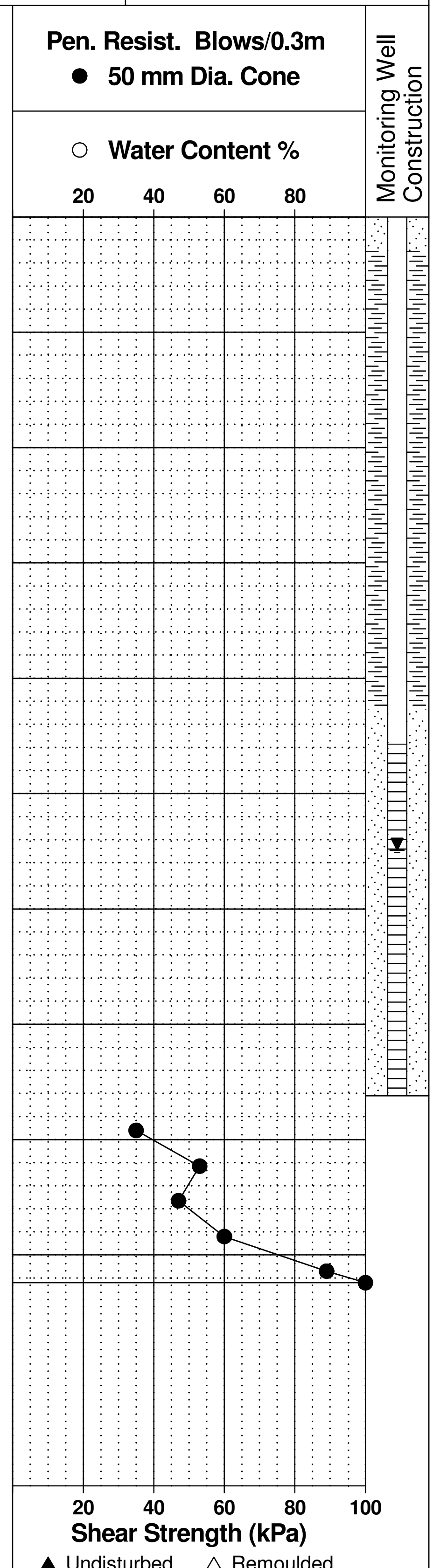
HOLE NO. BH 1-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.20	AU	1		
FILL: Brown silty sand with gravel, some crushed stone		SS	2	17	14
- some clay by 1.4m depth	2.44	SS	3	42	7
Compact, brown **SILTY SAND**	2.97	SS	4	50	14
Dark grey **SILTY CLAY** with sand and gravel, occasional cobbles	3.66	SS	5	83	24
Dense, brown **SILTY SAND to SANDY SILT** with gravel, occasional cobbles, trace clay	4.50	SS	6	100	50
Dense to very dense, light brown to brown **SILTY SAND**		SS	7	75	45
		SS	8	50	45
		SS	9	82	50+
- with gravel by 6.7m depth	7.62	SS	10	67	63
Dynamic Cone Penetration Test commenced at 7.62m depth.	9.24				
End of Borehole					

Practical DCPT refusal at 9.24m depth.

(GWL @ 5.49m - May 11, 2022)

DEPTH (m)	ELEV. (m)
0	62.80
1	61.80
2	60.80
3	59.80
4	58.80
5	57.80
6	56.80
7	55.80
8	54.80
9	53.80



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 1166 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 3, 2022

FILE NO. PG6191

HOLE NO. BH 2-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone with gravel	0.20	AU	1		
FILL: Brown silty sand with gravel - some clay by 0.8m depth	2.44	SS	2	8	7
		SS	3	8	8
Compact, reddish brown **SILTY SAND**	2.97	SS	4	58	12
Compact, dark brown to brown **SILTY SAND to SANDY SILT,** some gravel, trace clay	4.27	SS	5	75	17
		SS	6	100	50+
Very dense, light brown to brown **SILTY SAND**	6.10	SS	7	75	62
		SS	8	83	52
End of Borehole (GWL @ 5.48m - May 11, 2022)					

DEPTH (m)	ELEV. (m)
0	62.88
1	61.88
2	60.88
3	59.88
4	58.88
5	57.88

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 Multi-Storey Building - 1166 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 3, 2022

FILE NO. PG6191

HOLE NO. BH 3-22

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.33
Asphaltic concrete	0.05						
FILL: Crushed stone with gravel	0.20	AU	1				
FILL: Brown silty sand with gravel, trace clay, occasional cobbles		SS	2	75	13	1	61.33
		SS	3	67	10	2	60.33
	2.97	SS	4		27		
Compact, brown **SILTY SAND** with gravel, occasional cobbles		SS	5	100	22	3	59.33
	4.34	SS	6	58	29	4	58.33
Hard, dark grey **SILTY CLAY** with sand and gravel	5.18	SS	7	92	32	5	57.33
Very dense, brown **SILTY SAND** with gravel, occasional cobbles and boulders		SS	8	83	58	6	56.33
		SS	9	100	50+		
	7.47	SS	10	36	50+	7	55.33
End of Borehole							
(GWL @ 4.93m - May 11, 2022)							

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

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

CONSOLIDATION TEST

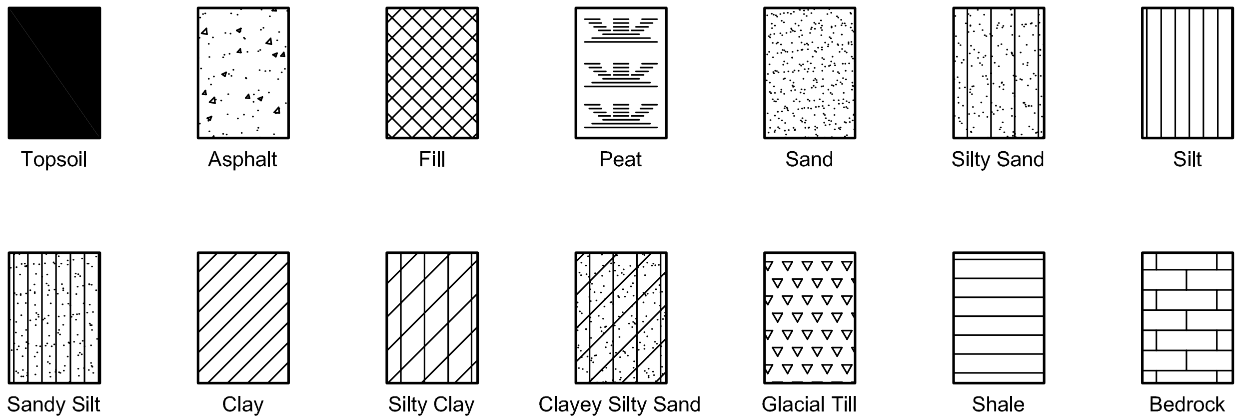
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

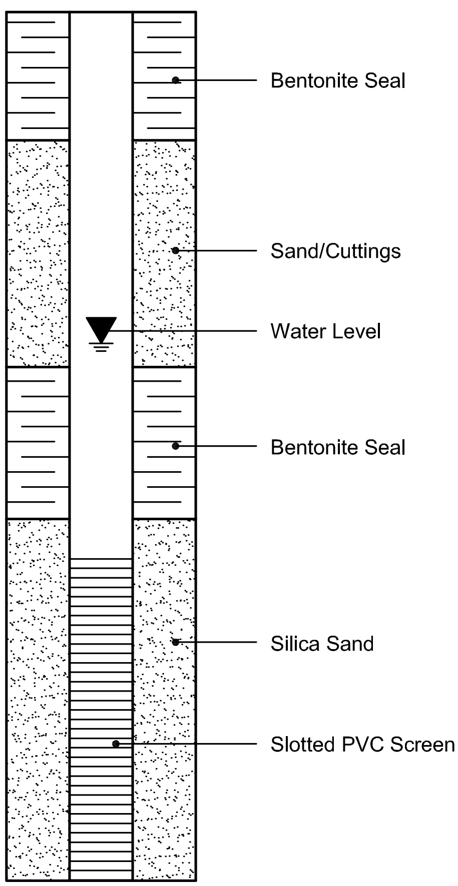
SYMBOLS AND TERMS (continued)

STRATA PLOT

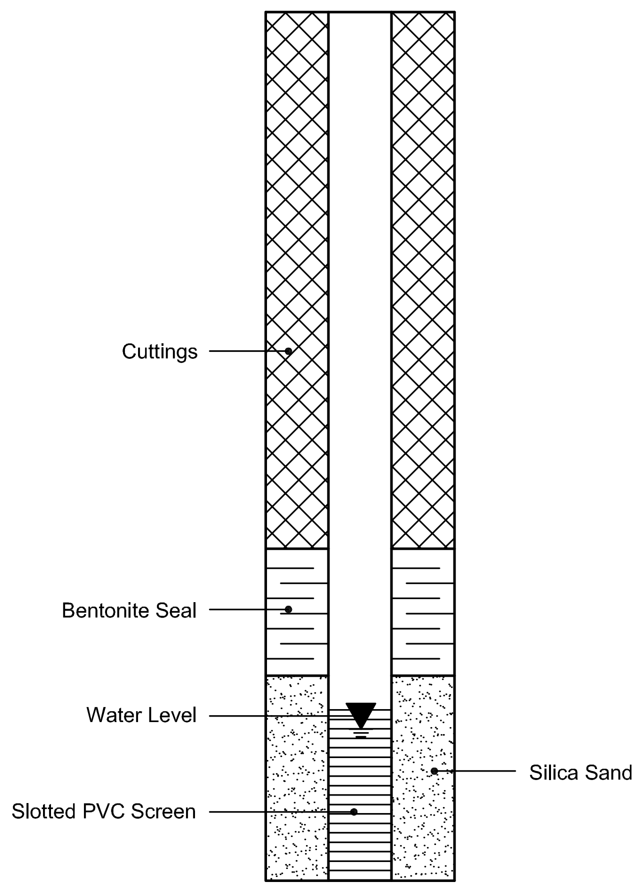


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 21458827

RECORD OF MONITORING WELL: 21-01

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 29, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ (ND = Not Detected) — 20, 40, 60, 80
HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected) — 20, 40, 60, 80
HYDRAULIC CONDUCTIVITY, k, cm/s — 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}
WATER CONTENT PERCENT — Wp ⊢—○W—⊣ Wl — 20, 40, 60, 80
ADDITIONAL LAB. TESTING

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	HEADSPACE	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	100.59					
ASPHALTIC CONCRETE	0.00					Flush Mount Casing
GRAVEL, medium to coarse	0.15	1	SS	-	□ ND ⊕	
Sandy SILT, trace gravel, fine; light brown	100.07 / 0.52					
SILTY SAND; brown, contains cobbles; loose	0.61	2	SS	-	□ ND ⊕	
SILTY SAND, trace gravel; brown; moist, loose	99.07 / 1.52	3	SS	-	□ ND ⊕	
Sandy SILT, fine; light brown; moist, very loose	98.46 / 2.13	4	SS	-	□ ⊕ ND	Bentonite Seal
SILTY CLAY; brown; moist, soft to firm	97.85 / 2.74	5	SS	-	□ ⊕	
SILTY CLAY; brown with orange mottling; moist, firm	96.78 / 3.81	6	SS	-	□ ⊕	
SAND, medium; light brown; moist to wet, loose	96.02 / 4.57	7	SS	-	□ ND ⊕	Silica Sand
		8	SS	-	□ ND ⊕	32 mm Diam. PVC #10 Slot Screen
		9	SS	-	□ ND ⊕	
BEDROCK	94.07 / 6.52					
	6.64					

W.L. in Screen at Elev. 94.79 m in April, 2021

DRAFT

DEPTH SCALE 1 : 50

GOLDER

LOGGED: BD

CHECKED:

MIS-BHS 001 1156 BANK OTTAWA.GPJ GAL-MIS.GDT 4-28-21 JEM

PROJECT: 21458827

RECORD OF MONITORING WELL: 21-02

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 30, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	BLOWS/0.30m	HEADSPACE COMBUSTIBLE / ORGANIC VAPOUR CONCENTRATIONS [PPM]	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE	100.43				
		ASPHALTIC CONCRETE	0.00				Flush Mount Casing
	Power Auger, 200 mm Diam. (Hollow Stem)	SAND, some silt, trace gravel, fine; light brown; moist, loose	0.15 / 99.82	1	-	ND	
1		No Recovery	0.61 / 99.21	2	-		
		Sandy SILT, trace gravel, fine; brown; moist, soft	1.22 / 98.60	3	-	ND	
2		SILTY SAND, trace gravel, fine; light brown; moist, loose	1.83 / 97.38	4	-	ND	Bentonite Seal
				5	-		
3		Sandy SILT, trace gravel, fine; brown; moist, soft	96.77	6	-	ND	
		SAND, some silt; brown; moist, loose	3.66 / 96.53	7	>50		
4		No Recovery	3.90 / 94.94	8	-		Silica Sand
5				9	30		
		SAND, some silt; brown; moist, loose	5.49 / 94.03	10	35		
6				11	15	ND	32 mm Diam. PVC #10 Slot Screen
7	Rotary Drill, NQ Core	No Recovery	6.40 / 92.81				
			7.62				
8							WL in Screen at Elev. 94.70 m in April, 2021
9							
10							

DRAFT

MIS-BHS 001 1166 BANK OTTAWA.GPJ GAL-MIS.GDT 4-28-21 JEM

DEPTH SCALE
1 : 50

GOLDER

LOGGED: BD
CHECKED:

PROJECT: 21458827

RECORD OF MONITORING WELL: 21-03

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 29, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Power Auger, 200 mm Diam. (Hollow Stem)

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM]	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	100.00					
ASPHALTIC CONCRETE	0.00					Flush Mount Casing
Sandy SILT, trace gravel, coarse; brown; moist, loose	0.15	1	SS	7	ND	
SILTY SAND, trace gravel, fine; brown; moist, loose	99.39 / 0.61	2	SS	9	ND	
		3	SS	6	ND	Bentonite Seal
SILTY SAND; brown, contains cobbles; moist, loose	98.17 / 1.83	4	SS	16	ND	
No Recovery	97.56 / 2.44	5	SS	21	ND	
SILTY SAND; brown, contains cobbles; moist, loose	96.95 / 3.05	6	SS	61	ND	Silica Sand
No Recovery	96.34 / 3.66					
SILTY SAND; brown; moist, compact	95.73 / 4.27	8	SS	38	ND	
SAND, medium; brown; moist, loose	95.12 / 4.88	9	SS	23	ND	32 mm Diam. PVC #10 Slot Screen
SAND, medium; brown; wet, loose	94.51 / 5.49	10	SS	51	ND	
SILTY SAND; brown; wet, soft	94.06 / 5.94	11	SS	42	ND	
End of Borehole	93.29 / 6.71					WL in Screen at Elev. 94.75 m in April, 2021

DRAFT

MIS-BHS 001 1166 BANK OTTAWA.GPJ GAL-MIS.GDT 4-26-21 JEM

DEPTH SCALE 1 : 50

GOLDER

LOGGED: BD

CHECKED:

RECORD OF MONITORING WELL: 21-04

PROJECT: 21458827

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 30, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	HEADSPACE VAPOUR CONCENTRATIONS	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	100.01					
ASPHALTIC CONCRETE	0.00					Flush Mount Casing
Sandy SILT, trace gravel, fine to coarse; brown; moist, loose	0.15	1	SS	23	ND	
		2	SS	19		
		3	SS	16	ND	Bentonite Seal
Lost	98.18 / 1.83	4	SS	11		
SILTY SAND, trace gravel, fine; brown; moist, loose	97.57 / 2.44	5	SS	5	ND	Silica Sand
Sandy SILT; brown; moist, soft	96.96 / 3.05	6	SS	9	ND	
Sandy SILT; grey with black staining; moist to wet, soft	96.25 / 3.76	7	SS	20		
		8	SS	15		32 mm Diam. PVC #10 Slot Screen
SILTY CLAY; brown; moist, soft	94.83 / 5.18	9	SS	26		
Sandy SILT, trace gravel, fine; brown; moist, soft	94.52 / 5.49	10	SS	27		
End of Borehole	93.76 / 6.25					WL in Screen at Elev. 94.80 m in April, 2021

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

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

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

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

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE 1 : 50

GOLDER

LOGGED: BD

CHECKED:

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31847

Report Date: 09-May-2022

Order Date: 4-May-2022

Project Description: PG6191

	Client ID:	BH2-22 SS6	-	-	-
	Sample Date:	03-May-22 09:00	-	-	-
	Sample ID:	2219423-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	-	-	-
General Inorganics					
pH	0.05 pH Units	9.46	-	-	-
Resistivity	0.10 Ohm.m	18.5	-	-	-
Anions					
Chloride	5 ug/g dry	201	-	-	-
Sulphate	5 ug/g dry	184	-	-	-

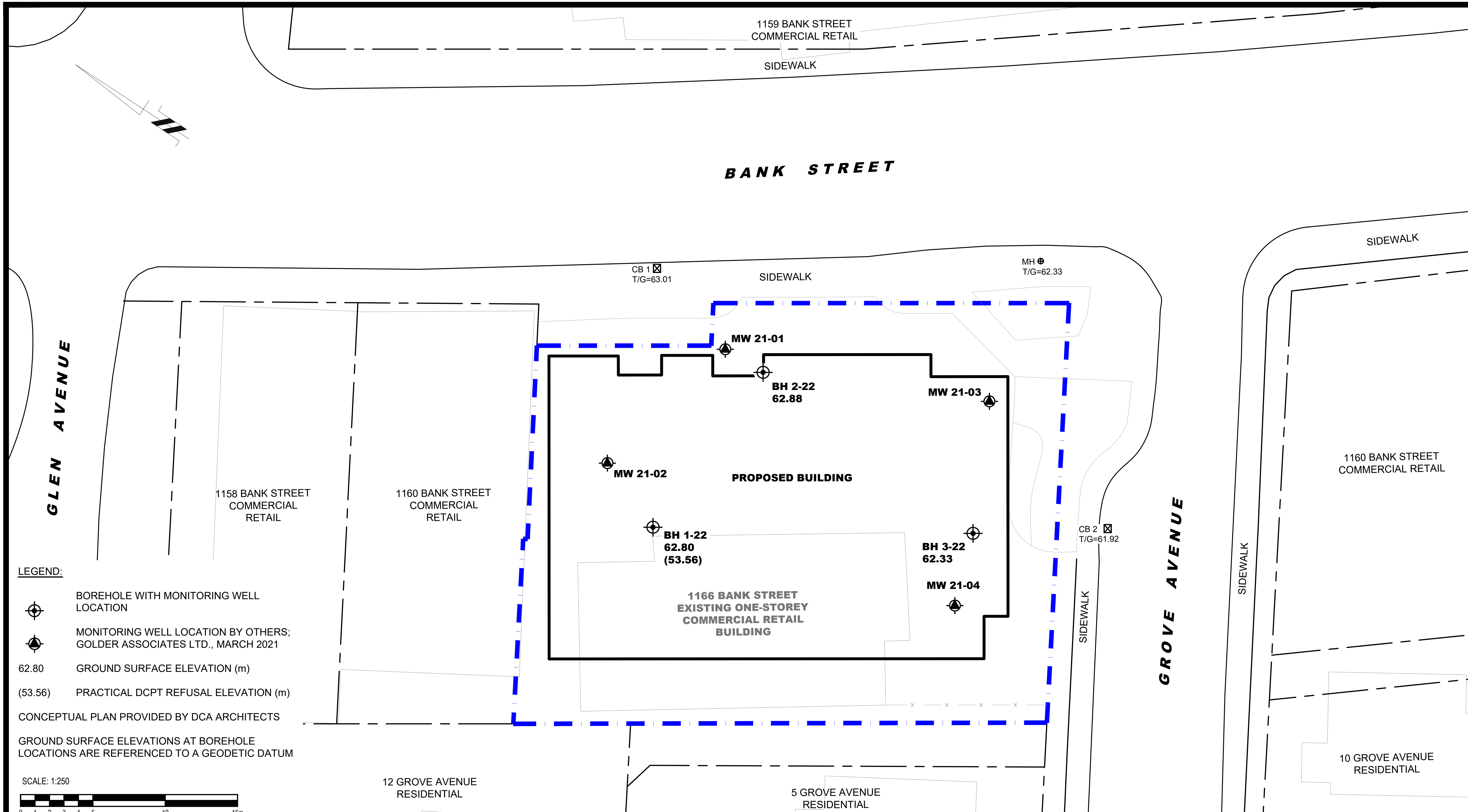
APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6191-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

AMBASSADOR REALTY INC.

GEOTECHNICAL INVESTIGATION

1166 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 05/2022
Drawn by: JM	Report No.: PG6191-1
Checked by: SD	Dwg. No.: PG6191-1
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg61xx\pg6191\pg6191-1 thlp.dwg

11x17